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Brief Review of Studies on Concrete Wall Panels in One and Two Way Action

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ABSTRACT: *This paper provides review of research results undertaken on reinforced concrete wall panels in one way and two way. The review also highlights two well accepted code design methods from the American (ACI) and Australia Concrete structures standards. The emphasis is on walls under axial compression only with changes in various parameters. These include the variation of panel dimensions panels (ie. Slenderness, thinness and aspect ratios), steel reinforcement, eccentricities, concrete strength and support conditions. The main purpose of this review is to compile research previous by undertaken to highlight the inadequate in certain research literature. It is envisage that this review will expose areas in wall research required so that inadequate in current methods can be rectified.*

KEY WORDS: Concrete Wall Panel, Code Methods, Empirical and Semi Empirical Formula.

1. Introduction

In the design of reinforced concrete wall panels, one of the most critical aspects that needs to be considered is on wall panel dimensions. However, the design of wall panels are currently carried out using empirical or semi-empirical methods which, amongst others, include the design code provisions. These methods involve approximations, which are not always reliable. Being empirical, their scope of application is limited. In view of this, some investigations have recently been undertaken on the applicability and the performance of such methods.

Many researchers have investigated the behaviour of reinforced concrete walls either in one-way or two-way action. Fig. 1 shows the typical example of one and two-way action on walls loaded axially. For walls in one-way action (Fig. 1(a)), with supports top and bottom only and applied axial load, many studies have been carried out. Seddon (1956) contributed to the development of the British code equation (BS8110). Similarly Leabu (1959), Oberlender (1973), Pillai and Parthasarathy (1977), Kripanarayanan (1977), Zielinski et al. (1982,83) made significant contributions to the development of the ACI equation (ACI 318). Also Saheb & Desayi (1989,1990) and Fragomeni et. al (1996) reviewed and investigated the Australian Code method (SAA 3600-94).

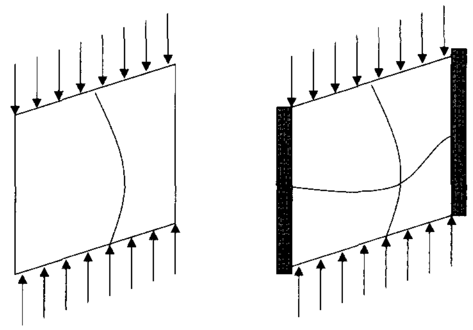
Two-way action (Fig.1 (b)) considers the buckling of concrete walls, with side supports and axial compression. Studies carried out on two-way walls are by Swartz and Rosebraugh (1974), Saheb and Desayi (1990) and recently Attard (1994), Fragomeni et al. (1996) and Maheswaran and Sanjayan (1997).

Attard (1994) has reported that wall panels with slenderness ratios exceeding 30 fail in a buckling manner as in Fig. 1 (b). Fragomeni et al. (1996) proposed modifications to the AS3600 code equation, which has incorporated the effect of high strength concrete parameters and the effect of side supports. Maheswaran and Sanjayan (1997) carried out laboratory tests on high strength concrete walls, which provided the laboratory test results for the calibration of a finite element model to predict the strength of the wall.

2. Code Equation

The Australian concrete standard (SAA 3600-94), gives two methods for the design of concrete walls. Section 11 of the code specifies a simplified equation which can be used for the design of walls when certain loading and bracing restrictions are met. The code also allows any wall to be designed as a column using the provisions of Section 10.

For the simplified design methods, the ultimate design axial



(a) One way action (b) Two way action

Fig. 1 Walls with side supports

strength per unit length, Nu, of a braced wall in compression is given by the following formula.

$$\phi N_u = \phi(t_w - 1.2e - 2e_a)0.6f_c' \quad \text{when} \quad 20 \le f_c' \le 50 \quad (1)$$

This equation applies to walls where the slenderness ratio, $H_{we}/t \le 30$(if the ultimate design axial force, $N^* \le 0.03f'_c Ag$ then $H_{we}/t \le 50$). A practice sometimes adopted in Australia is to used $H_{we}/t \le 20$ when large axial loads are encountered. The walls are required to have minimum reinforcement ratios of 0.0015 vertically, ρ_v, and 0.0025 horizontally, ρ_h.

ACI318 (1989) also offers two alternatives for the design of concrete walls, a simplified method and a more accurate method using column design. For the simplified method, ACI318 gives an empirical equation for the design axial load strength of a wall as:

$$P_u = 0.55 f_c' A_g [1 - (kH/32t_w)^2] \quad (2)$$

The equation applies to walls where $H_{we}/t \le 25$ or $L/t \le 25$ whichever is less for loadbearing walls. The minimum allowable thickness is 100mm. The resultant load must be in the 'middle third' of the overall thickness of the wall. This allows for a maximum eccentricity allowance of t/6. The walls are required to have minimum reinforcement ratios of 0.0015 vertically, ρ_v and 0.0025 horizontally, ρ_h. These values can be reduced to 0.0012 and 0.002 respectively if bars are less than 16mm diameter or if mesh is used.

3. Research on Walls in One Way Action

Seddon (1956) tested several wall panels under axial load and concluded the panels with H/t (slenderness) values less than 20 failed by crushing while those with large H/t values, invariably failed by buckling. Also the author concluded that the contribution of steel to the strength of panel did not exceed the yield strength of vertical steel for single layer disposition. However, the double layer placement was more effective in increasing the strength. In eccentricity loading, the rectangular stress distribution in compression together with zero tension resistance of concrete yielded satisfactory strengths. The reduction in strength due to eccentricity at 1/6 of the wall thickness was more than 17 percent and author suggested an axial stress formula as:

$$F_a = 0.2 f_c' [1 - (H/40t_w)^3] \quad (3)$$

Oberlender (1973) tested 54 panels axially. Oberlender found the reduction in strength due to an eccentricity of 1/6 of the wall thickness (t) varied from 18 percent to 50 percent for the slenderness ratio between 8 and 28. The proposed equation for the axial load capacity of a wall is:

$$P_u = 0.60\phi f_c' L t_w [1 - (\frac{H}{30t_w})^2] \quad (4)$$

While Oberlender (1973) concentrated on testing walls with two layers of reinforcement, which help wall strength against eccentric loading, Pillai and Parthasarathy (1977) concentrated on the behaviour of panels with a central single layer of reinforcement and the proposed following equation.

$$P_u = 0.57 f_c' A_g [1 - (h/50t_w)^2] \quad (5)$$

This equation is only recommended for walls with single layered reinforcement and slenderness ratios are less than 30.

Kripanarayanan (1977) made important contributions to the modification of the empirical design equation and the design of precast (tilt-up) panels. Kripanarayanan (1977) showed that the ACI 318-71 empirical equation was made up of the product of two functions, F_1 and F_2 as shown below,

$$P_u = 0.55 f_c' A_g [1 - (H/40t_w)^2] \quad (6)$$

where $F_1 = 0.55$ is a function of eccentricity and $F_2 = [1 - H/40t_w)^2]$ is a function of slenderness.

Kripanarayanan (1977) concluded that the strength part of the design equation, F_1, gives a satisfactory estimation of short wall capacities for both plain and minimally reinforced wall elements under reasonably concentric loads. A substantial increase in wall capacity can be obtained only if the amount of vertical reinforcement is of the order of 0.75 to 1.0 percent of the gross cross-sectional area of the wall. The slenderness part of the design equation, F_2, does not give a realistic estimate of capacities for walls with pin-ended supports. So Kripanarayanan (1977) recommend a change to F_2 to include k factor(ie $F_2 = [1 - kH/40t_w)^2]$). His modified equation is given as:

$$P_u = 0.55 f_c' A_g [1 - (kH/40t_w)^2] \quad (7)$$

The work of Zielinski et al. (1982, 1983) focused on testing panels with ribs around all edges. Thin wall panels with small cross-sectional area and large relative ratios of steel were tested. In the case of thin wall panels, a revised equation should be used as given in below.

$$P_u = 0.55 f_c' A_g [1 - (h/40t_w)]^2 [1 + \rho_m (m-1)] \quad (8)$$

The equation is only applicable to panels which have reasonably concentric loading ($e \le t_w/6$) and thin wall panels comparable to those studied ($L/t < 32$ and $H/t_w < 72$).

Saheb and Desayi (1989) tested 24 reinforced concrete panels under one-way action and authors compared the results predicted by the proposed modified method and previous methods. The proposed equations are:

$$P_u = 0.55\phi[A_g f_c' + (f_y - f_c')A_{sv}]\left[1-\left(\frac{H}{32t_w}\right)^2\right]\left[1.20-\left(\frac{H}{10L}\right)\right]$$

for h/L < 2.0 (9)

$$P_u = 0.55\phi[A_g f_c' + (f_y - f_c')A_{sv}]\left[1-\left(\frac{H}{32t_w}\right)^2\right]$$

for h/L $\geq$ 2.0 (10)

The authors concluded that: 1) the ultimate strength of the wall panel decreases linearly with increase in aspect ratio (H/L); 2) the ultimate strength of the wall panel decreases nonlinearly with increase in slenderness ratio (H/t); 3) the decrease in ultimate load is about 35 % for an increase in h/t from 9 to 27; 4) the ultimate strength of the wall panel increases almost linearly, with increase in vertical steel and 5) the effect of horizontal steel on the ultimate strength of wall panel is negligible.

Fragomeni (1996) also carried out a series of tests to study wall behaviour. In stage 1, 16 wall panels of varying slenderness ratio (H/t_w =12 to 25), varying aspect ratios (H/L = 2 to 5), and varying thinness ratio (L/t_w = 3.75 to 12.5) were tested as one way spanning walls. All walls had minimum reinforcement. Stage 2 consisted of four panels which had a single layer of reinforcement (opposed to reinforcement in the centre, in stage 1) 10 mm form the tension face (walls being 50 mm in thickness).

Fragomeni (1996) concluded that the failure mode of an axially loaded wall panel depended on the concrete strength, slenderness ratio and the amount of reinforcement used. In the testing program, the position of the minimum reinforcing mesh centrally or towards the tension face did not have an effect on the failure mode. However, the change in concrete strength from normal to high (from 40MPa to 70 MPa) did have an impact on the failure mode. Authors concluded that the direct interpolation of the current wall design equation for high strength concrete (HSC) may be a dangerous practice, particularly when only a minimum amount of reinforcement was present.

Fragomeni (1996) proposed a modification to AS3600 wall design equation to allow for the inclusion for HSC strength parameters. The following two tiered equations were proposed to include HSC parameters in the AS3600 equation.

$$\phi N_u = \phi(t_w - 1.2e - 2e_a)0.6f_c' \quad \text{when } 20\, f'_c \leq 50 \quad (11)$$

$$\phi N_u = \phi(t_w - 1.2e - 2e_a)30[1+(f_c'-50)/80] \quad \text{when } 50 \leq f'c \leq 80 \quad (12)$$

Although the concrete strength of those panels ranged from 35 MPa to 70 MPa, the researchers concluded that the equations' validity could be extended to 80 MPa, due to the conservative approach taken and the incorporation of the ϕ factor in the design.

Table 1 gives an overview of work done on walls in one way action.

4. Research on Walls in Two Way Action

Ernest (1952) tested 10 rectangular, reinforced concrete panels, which were simply supported along all edges (two-way action). Ernest (1952) concluded that in the analysis of stability problems concerning thin-shelled structures, either the Rankin-Gordon type of empirical equation or tangent-modulus instability curves may be used. The equation is given as:

$$P = \frac{f_c'}{1 + cf_c'/P_{cr}} \quad (13)$$

where $P_{cr} = \frac{kE}{(L/t)^2}$ and $k = \left(\frac{\pi^2}{12(1-\mu^2)}\right)(H/L + L/H)$ and c is

Table 1 Summary of one-way action tests panels and variables used

Research	Number of test	Concrete strength	Slenderness ratio (H/t)	Aspect ratio (H/L)	Steel ratio (ρ v)	Ecc
Seddon (1956)	-	17.5 to 28	18 to 54	1.5	0.008 single 0.004 double	0 to $t_w/3$
Leabu (1959)	Theoretical analysis	-	-	-	-	0
Oberlender (1973)	54	28 to 42	8 to 28	1 to 3.5	0.0033 single 0.0047 double	$t_w/6$
Pillai et. al. (1977)	18	16 to 31.5	16 to 31.5	5 to 30	0.0015 or 0.003	$t_w/6$
Kripanarayanan (1977)	Theoretical analysis	28	0 to 32	0 to 0.66	-	$t_w/6$
Zielinski et. al (1977)	5	33 to 37.5	72	2.25		$t_w/6$
Saheb and Desayi (1982, 1983)	24	20.2 to 25.17	12 to 27	0.67 to 2.0	0.00173 to 0.00856	$t_w/6$
Fragomeni et. al (1996)	20	36 to 60.7	12 to 25	2 to 5	0.0025 to 0.0031	$t_w/6$

3, an empirical constant derived from the tests.

Swartz, et al (1974) tested 24 rectangular, reinforced concrete panels. In all cases, the mode of failure was similar to that of a simply supported slab subjected to a uniform load applied transversely. The following formula for predicting the average or membrane stress in the concrete at the onset of buckling was proposed,

$$f_{cr} = 0.425 f_c' B[-B + (4 + B^2)^{0.5}] \quad (14)$$

and $$B = \frac{\pi^2 (1/L + L)^2 (h/b)^2}{6\varepsilon_0 (1-\rho)}$$

where L = a/b, if a/b<1 and L=1, if a/b≥1. a, b and h are plate length, width and thickness, respectively.

The authors concluded that the simply supported rectangular, reinforced concrete plates subjected to uniaxial compression may fail by buckling at stress levels considerably lower than the material's compressive strength.

Saheb and Desayi (1990) tested 24 reinforced concrete panels where the reinforcement in the panel was placed in two layers symmetrically on the two faces and was fabricated form steel bars with diameter varying from 2 mm to 5 mm. Saheb and Desayi (1990) proposed two equations for predicting the ultimate strength of wall panels in two-way action. The first equation is empirical and attempts a safe prediction of ultimate load. The second proposed equation is semi-empirical and is developed from a modification of the buckling strength of thin rectangular metal plates.

First Proposed Method:

$$P_u = 0.67\phi f_c' A_g \{1 - [L/(120t_w)]^2\}\{1 + 0.12(H.L)\} \quad (15)$$

Where $Q = 0.67 f_c' \phi A_g \{1 - [L/(120t_w)]^2\}$

Second Proposed Equation is given as below.

$$P_u/(Lt_w) = (\phi c_2 f_c')/(L/t_w) \quad (16)$$

where the constant c_2 is unknown. After a few trials, as the panel contains vertical and horizontal steel, they were assumed to yield, and the Equation has been rewritten as:

$$P_u = \phi c R \quad (17)$$

where $$R = \frac{A_g f_c' + A_{sv} f_{yv}\{1 + [(A_{sh} f_{yh})/A_{sv} f_{fv})]\}}{(L/t_w)} \quad (18)$$

and C=0.8352(L/t)-0.0052(L/t)^2 for L/t < 60

Saheb and Desayi (1990) concluded that: 1) the ultimate strength of wall panel in two-way action is found to increase linearly with aspect ratio (H/L); 2) the ultimate strength of wall panel in two-way action is found to reduce nonlinearly with an increase in thinness ratio (L/t) or slenderness ratio (H/t); 3) the increase in vertical steel ratio caused a linear increase in the ultimate strength of the panel; 4) the effect of horizontal steel ratio on the ultimate strength of the wall panel was found to be insignificant. Of the two equations, the first equation was found to be in better agreement with test data, and a lower coefficient of variation resulted when all tests were considered.

Attard (1994) proposed a formula for the buckling load of a rectangular simply supported concrete walls under uniform-compression due to in-plane buckling. The concrete was assumed to be fully in compression so that the stress strain relationship of concrete could be the uniaxial stress-strain relationship. The contribution of reinforcement was ignored. Attard (1994) used the tangent modulus theory, to get a solution for the buckling of plates. The results compared well with the experiments of Swartz et al. (1974). However, the results of Saheb and Desayi (1990) did not match as the eccentricities considered were different.

Fragomeni et al. (1996) proposed a modification to AS3600 wall design equation to allow for the inclusion for HSC strength parameters and adopted the effective height factors from the German concrete code DIN 1045 and incorporated them to be used in the AS3600 equation for the wall design. The two tiered

Table 2 Summary of two-way action tests panels and variables used

Research	Number of test	Concrete strength	Slenderness ratio (H/t)	Aspect ratio (H/L)	Ecc
Ernest (1952)	10	31.487	15.13 to 75.45	0.58 to 1.04	0
Swartz et. al. (1974)	24	16.65 to 27	75 to 128.51	2.0	0
Saheb and Desayi (1990)	24	20.17 to 25.17	9 to 25	0.67 to 2.0	$t_w/6$
Attard (1994)	Theoretical analysis	-	-	-	$t_w/6$
Fragomeni et. al (1996)	4	37.3 to 73.3	12 to 15	2.0 to 3.0	$t_w/6$
Maheswaran and Sanjayan (1996)	8	65 to 90.5	40	1.33	$t_w/6$

equation for walls supported on all four sides is given as below,

$$\phi N_u = \phi 0.7 f_c^{'} (t_w - 1.2e - 2e_a) \text{ when } 20 \le f'c \le 50 \quad (19)$$

$$\phi N_u = \phi 35 (t_w - 1.2e - 2e_a)[1 + (f_c^{'} - 50)/80] \text{ when } 50 \le f'c \le 80 \quad (20)$$

where ϕ=0.6 for compressive members and $H_{we}=\beta H$, and $\beta = 1/[1+(H/L)^2]$ if H≤L or β = L/2H if H > L.

Maheswaran and Sanjayan (1996) carried out a series of tests on walls loaded axially, supported on all four sides. The testing procedure was different to those carried out earlier, as the loading was carried out using a series of jacks, which provided constant loading, as opposed to constant displacements obtained in conventional tests in laboratories using uniaxial testing machines. These results were used as a basis to calibrate a non-linear finite element model in this project. The calculated results using previous researchers' equations were compared with the experimental results. It was found that the equations proposed by earlier researchers grossly over-estimate the failure load. Also the current code of practices available for calculating in-plane loads severely underestimate the failure load.

The details of walls tested in two way action are shown in Table 2.

5. Limitation

Table 3 shows the summary of the limitations for each research. Most studies focused on the behaviour of walls in one-way action. This type of research in particular contributed to the development of national code equations. Hence, the current practices available to calculate failure loads do not consider the contributions from side supports. As a consequence research work of walls in two-way action is yet to be incorporated in design codes.

6. Conclusion and Summary

A review of research undertaken on reinforced concrete walls has been undertaken. The review focused on axially loaded walls in one-way and two-way action. It was concluded that, in comparison to research on one-way walls, walls in two way action has recovered limited attention. Also only a few of studies focused on high strength concrete panels and walls with high slenderness ratios (H/t>30).

The authors have highlighted certain areas where more information is required and consequently more test in these area are needed. In particular, the high strength in two-way action and slenderness ratios between 15 to 40 should be investigated. Also the impact of reinforcement on axial load capacity needs to be studied. Then, the inadequacies which exist in the major national design codes and empirical or semi-empirical equations presented herein, may then be rectified.

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Table 3 Summary of limitation

Research	Limitation
Seddon (1956)	Eccentricity varied only up to $t_w/3$
Leabu (1959)	Theoretical analysis
Oberlender (1973)	Low rate of slenderness ratio (H/t) and no reinforcement ratio
Pillai, et. al (1977)	Low rate of slenderness ratio (H/t) and single reinforcement
Kripanarayanan (1977)	Low rate of slenderness ratio (H/t), concrete strength not varied
Zielinski (1977)	Very high rate of slenderness ratio (H/t), limited to dimension of the few models tested
Saheb and Desayi (1982, 1983)	Normal strength of concrete of approximated 22MPa
Ernest (1952)	Not concerned with eccentricity
Swartz et. al. (1974)	Not concerned eccentricity, very high rate of slenderness ratio
Saheb and Desayi (1990)	Do not included high strength concrete
Attard (1994)	Theoretical analysis
Fragomeni et. al (1996)	Only few models were tested in two-way action
Maheswaran and Sanjayan (1996)	Very high slenderness, very low rate of reinforcement were used

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Appendix

The following symbols are used in this paper.

ϕ=Capacity reduction factor
β=Factor for walls supported on four sides
ε_0=Average ultimate strain of concrete at 28 days
ρ=Total reinforcement ratio
ρ_h=Proportion of horizontal reinforcement in wall panel
ρ_v=Proportion of vertical reinforcement in wall panel
ρ_m=$fy/f'c$, the yield strength ratio of steel to concrete
μ=Poisson's ratio
A=Gross cross-sectional area of wall panel in plan (=Lt)
A_{sh}, A_{sv}=Areas of horizontal and vertical steel in the panel
C_1, C_2=Constants
E=Elastic modulus of concrete
e_a=An additional eccentricity due to deflections in the wall $e_a=(H_{we})^2/(2500t_w)$
f'_c=Cylinder strength of concrete
F_a=Axial stress
F_y=Yield strength of steel
f_{ys}, f_{yv}=Yield strength of horizontal and vertical steel respectively
H=Height of wall panel
H_{we}=The effective height of a wall
L=Length of wall panel
P_u=Ultimate load
T_w=Thickness of wall panel
W=Spacing of vertical ribs.