Instability analysis of reinforced concrete walls with various support conditions

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SUMMARY

Axially loaded reinforced concrete (RC) walls can be designed using simplified design methods given in codes such as the Australian Concrete Standard (AS3600-09) and the American Concrete Institute Code (ACI318-14). The ACI318-14 equation is intended for load bearing walls supported at top and bottom only. The AS3600-09 includes effective height factors to distinguish the effect of various support conditions, and also allows for higher concrete strengths and new guidelines not provided in the previous release, AS3600-01. However, these practical equations are based on empirical models and their scope of application is still limited. Recent research has been undertaken on the applicability of more reliable and accurate wall design methods. This paper initially presents a derived numerical technique incorporated in a computer program (WASTABT), to implement the iterative analysis for concrete walls with various support conditions. The outputs from the WASTABT program are verified using the results obtained from previous experimental results. A parametric study is then conducted using the verified computer-based numerical technique to analyse axially loaded RC panels. The study focuses on the effect of varying panel properties such as wall dimensions, concrete strengths, eccentricities and reinforcement ratios, along with varying support conditions.

KEY WORDS: reinforced concrete wall; design codes; high strength concrete; cracking; buildings; restraints.

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1. INTRODUCTION

Walls can be constructed with various support conditions. Walls restrained top and bottom only, with free vertical edges, are usually encountered in tilt-up concrete structures. Such walls behave in one-way (OW) action depicted by uniaxial curvature in the direction of loading, as shown in Figure 1(a). Axially loaded walls can also behave in two-way (TW) action when restrained on three or four sides, commonly encountered in core walls of high-rise buildings. These panels generally deform along both the horizontal and vertical directions, as shown in Figure 1(b,c). The previous Australian Concrete Standard (AS3600-01) and the current American Concrete Institute Code (ACI318-14) provide wall design equations intended for load bearing walls supported at top and bottom only. These do not recognise any contribution to load capacity due to side restraints. The current Australian Standard for Concrete Structures, AS3600-09, has been extensively revised and acknowledges the increased strength effects of side restraints, providing guidelines by way of effective height factors for different support conditions. Fragomeni and Doh (2010) and Popescu *et al.* (2015) reported that while giving safe predictions, the AS3600-09 simplified method provides predicted axial load capacities that are more conservative than necessary. Moreover, Robinson *et al.* (2013) stated that due to their inherent simplicity, both AS3600-09 and ACI318-14 codes cannot correctly account for the material and geometric nonlinearities in the buckling failure of slender walls under eccentric loading leading to large safety factors adopted.

A number of research studies have been conducted on OW action walls, including Saheb and Desayi (1989), Fragomeni (1995) and Sanjayan (2000). However, Doh and Fragomeni (2005) found that none of these models provided an adequate strength prediction for slender panels ($H_w/t_w > 30$) and therefore proposed a design formula allowing for higher slenderness ratios. More recently, Hegger *et al.* (2009), Ganesan *et al.* (2010), Robinson *et al.* (2013) and Huang *et al.* (2014) have attempted to further improve design models for OW wall panels. Hegger *et al.* (2009) developed a new design method taking into account the non-linear material behaviour of

concrete and the concrete tensile strength. Nonetheless, the specific load-bearing capacity for concrete walls proposed by the authors is only valid for normal strength concrete (NSC) up to 50 MPa. Ganesan *et al.* (2010) tested 16 wall panels to study the axial strength of self-compacting concrete (SCC). Due to highly conservative predictions from available methods in the literature, the authors proposed a new method to predict the ultimate load of SCC wall panels. Robinson *et al.* (2013) proposed a new model based on the semi-empirical semi-probabilistic DAT (Design Assisted by Testing) methodology, utilizing the lumped plasticity computational model with a non-linear fibre hinge at the structural response of slender RC panels. According to Popescu *et al.* (2015), this model is more appropriate for NSC panels. Huang *et al.* (2014) devised a nonlinear theoretical model accounting for concrete cracking, tension stiffening, strain softening in compression and yielding of reinforcements, along with geometric nonlinear effects. The model was solved numerically with the use of the arc-length method. These authors also tested eight eccentrically loaded high strength concrete (HSC) panels to compare the results with their model and concluded satisfactory predictions were achieved.

Several attempts have been made in the past to investigate the strength and behaviour of RC two-way panels supported on four sides (TW4S) by Saheb and Desayi (1990), Fragomeni (1995), Sanjayan and Maheswaran (1999), and previously reviewed by Doh (2002). In addition to the aforementioned OW design equation, Doh and Fragomeni (2005) also extended the applicability of their design model to cater for TW4S walls of slenderness ratios up to 40. Nevertheless, this model predicted the test results provided previously by Sanjayan and Maheswaran (1999) with great discrepancies (Doh, 2002). Only two experimental tests on walls in TW action supported on three sides (TW3S) have been undertaken by Doh *et al.* (2008). Fragomeni and Doh (2010) compared those results with the AS3600-09 and concluded that conservative prediction is achieved. They recommended that more testing or extensive analyses be undertaken, due to limited experimental test data, to verify the current design method with TW3S support conditions and various configurations of walls. This includes the effects of

slenderness ratios (H_w/t_w), aspect ratios (H_w/L_w), eccentricities and concrete strengths, hence aiming to expand the limited scope of current codes of practice.

In view of this need, the authors have undertaken an extensive numerical analysis, incorporated into a computer program (WASTABT) written in the Matlab programming package, to establish the behaviour of NSC and HSC walls under various support conditions with broader applicability. This method is a modified version of the technique developed by El-Metwally *et al.* (1990), Fragomeni and Mendis (1997) for OW walls and Sanjayan and Manickarajah (1995) for TW4S walls. Also, the authors have proposed a model based on Sanjayan and Manickarajah's method (1995) to account for the buckling analysis of TW3S RC panels. The outputs from the WASTABT program are compared to existing test results obtained from previous studies to verify the material and geometric modelling techniques adopted. The verified WASTABT program is then used to conduct a parametric study to analyse the behaviour of RC wall panels. Owing to limited research on the behaviour of TW action panels, this study focuses more on the effect of varying panel properties such as slenderness ratios ($10 \leq H_w/t_w \leq 60$), aspect ratios ($0.4 \leq H_w/L_w \leq 2$), concrete strengths ($32 \text{ MPa} \leq f'_c \leq 100 \text{ MPa}$), reinforcement ratios ($0.0015 \leq \rho_v \leq 0.01$) and eccentricities ($t_w/20 \leq e \leq t_w/3$) for panels supported on three and four sides.

2. SIMPLIFIED WALL DESIGN METHOD

2.1 The Australian Concrete Standard (AS3600-09)

For the simplified design method, the ultimate design axial strength per unit length of a braced wall is given by the following formula:

$$\phi N_u = \phi(t_w - 1.2e - 2e_a)0.6f'_c \quad (1)$$

where $\phi = 0.6$ is the capacity reduction factor, t_w is the wall thickness (mm), e is the load eccentricity (mm) which has a minimum of $0.05t_w$, f'_c (MPa) is compressive concrete strength

($20 \leq f'_c \leq 100$ MPa), and $e_a = H_{we}^2 / (2500t_w)$ is the additional eccentricity due to the deformation of the wall. The effective height as stipulated in Clause 11.4 shall be taken as $H_{we} = kH_w$ in which the factor k is determined for various support conditions as follows: for OW walls, $k = 0.75$, when walls are restrained against rotation at both ends, and $k = 1$, when walls are not restrained against rotation at one or both ends; for TW3S walls, $k = 1 / (1 + (H_w / 3L_w)^2) \geq 0.3$, but less than obtained from OW walls; for TW4S walls, $k = 1 / (1 + (H_w / L_w)^2)$ when $H_w \leq L_w$ or $k = L_w / 2H_w$ when $H_w > L_w$. H_w is the floor-to-floor unsupported height and L_w is horizontal length.

Provided the ratio of effective height to thickness (H_{we} / t_w) does not exceed 30, the Equation (1) is applicable to wall design. The walls are required to have minimum reinforcement ratios ρ_v and ρ_h of 0.0015 and 0.0025 respectively.

2.2 The American Concrete Institute Code (ACI318-14)

ACI318-14 gives the equation for the design axial load strength of a wall as:

$$\phi N_u = 0.55\phi f'_c L_w t_w \left[1 - (kH_w / 32t_w)^2 \right] \quad (2)$$

in which $\phi = 0.65$, $k = 0.8$ when walls are restrained against rotation at one or both ends (top, bottom, or both), and $k = 1$ when walls are unrestrained against rotation at both ends.

The equation was derived empirically from axial load tests on NSC walls in OW and generally applies to walls where $H_w / t_w \leq 25$. The resultant load must be in the 'middle third' of the overall thickness of the wall for this formula to be valid. This means a maximum eccentricity $t_w / 6$ is allowed. The minimum vertical and horizontal reinforcement ratios required to use Equation (2) are 0.0015 and 0.0025 respectively. If deformed bars with a diameter of less than 16 mm (or wire mesh) are used then these ratios can be reduced to 0.0012 and 0.0020 respectively.

3. NUMERICAL TECHNIQUE: WASTABT PROGRAM DEVELOPMENT

The two principal theoretical components of the program are moment-curvature calculation and evaluation of instability load.

3.1 Moment-Curvature calculation

In the derivation of a moment-curvature relationship, the requirements of strain compatibility and equilibrium of forces must be satisfied. Dividing the rectangular cross-section into a reasonable number of horizontal elements of equal depth parallel to the neutral axis (Figure 2(a)), the strain at the middle of each element, ε can be obtained. The following assumptions are made in the analysis: plane sections before bending remain plane after bending; and torsional resistances shear deformation, Poisson's effects and slip between reinforcement and concrete are neglected (Sanjayan and Manickarajah, 1995). With these assumptions, for a definite curvature κ , and an assumed strain at the extreme compressive fibre of the section ε_{cm} , a linear strain profile along the section can be determined as shown in Figure 2(b). From the stress-strain relationship of material, the value of stress σ corresponding to the strain ε can be obtained; thus the stress distribution in concrete section can be derived (Figure 2(c)). Also, the strain across the bar diameter is uniform and equal to the strain at the bar centroid, the strain in the bar can be calculated. From the stress-strain relationship of reinforcing steel, the stress in the reinforcement is obtained. The axial force, N , and the bending, M , corresponding to assumed strain profile of the section, can then be determined:

$$N = \sum_{i=1}^n \sigma_i A_i + \sum_{i=1}^{n_{st}} \sigma_{si} A_{si} \quad (3-1)$$

$$M = \sum_{i=1}^n \sigma_i A_i y_i + \sum_{i=1}^{n_{st}} \sigma_{si} A_{si} y_{si} \quad (3-2)$$

where A_i is the element area, A_{si} is the area of steel at layer i , σ_i is the stress at the centroid of element i , σ_{si} is the steel stress at layer i , y_i is the distance from the element centroid to the section centroid, y_{si} is the distance from the centroid of the steel layer i to the centroid of this section.

Comparing this calculated force, N , with the applied axial force, the imbalance axial force ΔN can be obtained. If ΔN is within a specified allowable tolerance, a point on the moment versus curvature curve is obtained. Otherwise, the assumed strain value at the extreme compressive fibre is modified, and the procedure for obtaining N and M is repeated. Repeating the produce for various values of curvature, κ , the full moment-curvature relationship for the specified axial force can be obtained.

Numerous concrete stress-strain relationship models are available for calculating the moment-curvature characteristics of RC wall section. The models developed by Saenz (1964) and Lu and Zhao (2010) were chosen for the analysis of the wall panels investigated in this paper because they adequately predict the behaviours of unconfined concrete, which is generally the case for wall panels having central reinforcement or reinforcement in both faces with limited confinement, of normal and high strength concrete under uniaxial compression, respectively. For lightly RC members like wall panels, the tension stiffening effect needs to be considered because of its ability to provide a large proportion of the postcracking stiffness (Gilbert, 2007). El-Metwally *et al.* (1990) also took into account the contribution of concrete in tension in their analysis. The models from previous studies (Sanjayan and Manickarajah, 1995; Fragomeni and Mendis, 1997), however, neglected the tensile capacity of concrete. In this study, the applicable concrete model in tension, proposed by Fields and Bischoff (2004), is included for more accurate prediction.

3.1.1 Concrete in compression

The initial modulus of elasticity of concrete E_c is computed by the following equations as given in AS3600-09:

$$E_c = \rho^{1.5} (0.043 \sqrt{f'_c}) \text{ MPa} \quad \text{where} \quad f'_c \leq 40 \text{ MPa, or} \quad (4-1)$$

$$E_c = \rho^{1.5} (0.024 \sqrt{f'_c} + 0.12) \text{ MPa} \quad \text{where} \quad f'_c > 40 \text{ MPa} \quad (4-2)$$

in which ρ is the concrete density and may be taken as 2400 kg/m³. These empirical equations were also used to simulate the concrete walls' properties by Lima *et al.* (2016).

The stress-strain relationship proposed by Saenz (1964) is used to construct the uniaxial compressive stress-strain curve for concrete with compressive strength less than 50 MPa:

$$\sigma_c = \frac{E_c \varepsilon}{1 + (R + R_E - 2) \left(\frac{\varepsilon}{\varepsilon_0} \right) - (2R - 1) \left(\frac{\varepsilon}{\varepsilon_0} \right)^2 + R \left(\frac{\varepsilon}{\varepsilon_0} \right)^3} \quad (5)$$

where $R = \frac{R_E (R_\sigma - 1)}{(R_\sigma - 1)^2} - \frac{1}{R_\sigma}$, $R_E = \frac{E_c}{E_0}$, $\varepsilon_0 = 1.491 \times 10^{-5} f'_c + 0.00195$, $E_0 = \frac{f'_c}{\varepsilon_0}$ and $R_\sigma = 4$, $R_\varepsilon = 4$

as used by Hu and Schnobrich (1989).

For HSC, the stress-strain relationship proposed by Lu and Zhao (2010) was used. The model is applicable for concrete with uniaxial compressive strength up to 140 MPa. The expressions that define the model are as follows:

$$\sigma_c = f'_c \left[\frac{(E_c / E_0)(\varepsilon / \varepsilon_0) - (\varepsilon / \varepsilon_0)^2}{1 + (E_c / E_0 - 2)(\varepsilon / \varepsilon_0)} \right] \quad \text{for } 0 \leq \varepsilon \leq \varepsilon_L \quad (6-1)$$

$$\sigma_c = f'_c \left[1 + 0.25 \left(\frac{\varepsilon / \varepsilon_0 - 1}{\varepsilon_L / \varepsilon_0 - 1} \right)^{1.5} \right]^{-1} \quad \text{for } \varepsilon > \varepsilon_L \quad (6-2)$$

where $\varepsilon_0 = 700 f'_c{}^{0.31} \times 10^{-6}$, $E_0 = f'_c / \varepsilon_0$ and $\varepsilon_L = \varepsilon_0 \left[(0.1 E_c / E_0 + 0.8) + \sqrt{(0.1 E_c / E_0 + 0.8)^2 - 0.8} \right]$

The value of ε_L corresponds to stress level of $0.8 f'_c$, on the descending branch of the stress-strain curve.

3.1.2 Concrete in tension

Huang *et al.* (2104) utilized the model proposed by Fields and Bischoff (2004) to model the tension-stiffening effect in concrete wall panels. It was found that this model was capable of describing the constitutive relationship of concrete in tension. The model takes the form of a

descending exponential relation between stress and strain in tension after the peak stress in tension and is defined by the following expressions:

$$\sigma_t = E_c \varepsilon \quad \text{for } 0 \leq \varepsilon \leq \varepsilon_{cr} \quad (7-1)$$

$$\sigma_t = e^{-0.8(\varepsilon - \varepsilon_{cr}) \times 10^3} E_c \varepsilon_{cr} \quad \text{for } \varepsilon_{cr} < \varepsilon \quad (7-2)$$

in which $\varepsilon_{cr} = f'_{ct,f} / E_c$, where ε_{cr} is the cracking strain, $f'_{ct,f}$ is the characteristic flexural tensile strength of concrete and $f'_{ct,f} = 0.6\sqrt{f'_c}$ is given in AS3600-09.

3.1.3 Steel Stress-Strain Model

The elastic-perfectly plastic model is adopted for steel reinforcement, and its constitutive law is determined by the simple bilinear model:

$$\sigma_s = E_s \varepsilon_s \quad \text{for } |\varepsilon_s| \leq \varepsilon_y \quad (8-1)$$

$$\sigma_s = E_s \varepsilon_y \quad \text{for } \varepsilon_y < \varepsilon_s \quad (8-2)$$

$$\sigma_s = -E_s \varepsilon_y \quad \text{for } \varepsilon_s < -\varepsilon_y \quad (8-3)$$

The elastic modulus of steel, E_s , is taken as 206,000 MPa.

3.2 Instability load evaluation

The evaluation of instability load is based on the maximum “effective height” the wall section can attain for a particular applied axial load before becoming unstable. As the compressive force increases, the instability effective height of the wall section diminishes. Lighter loads allow walls to reach greater effective heights before buckling occurs, whereas heavier loads require shorter panels to prevent buckling.

3.2.1 Numerical method of OW action walls

In the analysis, the wall is treated as a beam-column of unit width (vertical strip), subjected to an axial force and two equivalent end moments, which account for the eccentric loading and secondary effects (Figure 3). Due to the symmetry of the system, the analysis is performed on

the half-height of the wall, which is divided into “ n ” equal segments of length Δx (Figure 4). The wall section is given a preliminary deflected shape (sinusoidal deflection shape) with an initial midspan deflection Y_0 . The value of deflection at each segment, Y_i , is then calculated by the equation $Y_i = Y_0 \sin[\pi(i/2n)]$. The bending moment at each segment is calculated using $M_i = NY_i + M_{max}$, where M_{max} is the end moment on the wall. The values of curvature, κ_i , corresponding to M_i are determined from the moment versus curvature relationship. The double integration procedure suggested by Newmark (1943) is then used to calculate the deflection at each segment using

$$Y_i = \sum_{j=1}^i \left[\sum_{p=n}^j \kappa_p \Delta x \right] \Delta x = \Delta x^2 \alpha_i \quad (9)$$

in which $\alpha_i = \sum_{j=1}^i \sum_{p=n}^j \kappa_p$. Using the calculated deflection at midspan, α_n , a new value for Δx is consequently calculated by the equation $\Delta x^{new} = \sqrt{Y_0 / \alpha_n}$. New values for the deflection at each segment can then be calculated using $Y_i^{new} = (\Delta x)^2 \alpha_i$.

The new deflection values replace the old ones and the iteration continues until an updated value for Δx is obtained. If the difference between successive Δx values falls outside the acceptable tolerance, the iteration is repeated until the difference falls within the specified tolerance. Once the tolerance is achieved, a point on the height-midspan deflection curve is obtained ($H_w = 2n\Delta x$). The initial midspan deflection value, Y_0 , is incremented and the process continued until the ultimate moment capacity of the wall section is reached. The peak value on the height-deflection curve gives the instability height of the wall section for the particular axial load (instability load or failure load).

3.2.2 Numerical method of TW action walls

In the analysis, the wall is simulated as a series of vertical strips (columns) and horizontal strips (beams) parallel to the edges of the panel as shown in Figure 5. It is postulated that once

the columns are prone to buckle because of the applied vertical loads, the beams shall prevent the columns from buckling. The deflected columns exert horizontal forces on the beams, and the beams, consequently, counteract the columns. The critical load is obtained by satisfying the equilibrium conditions between external loads and internal resistances at each grid point (intersecting point of beam and column).

For TW4S panels, one quarter is considered for the analysis, since the wall geometry and the loadings are symmetric about two perpendicular axes parallel to the wall edges. The total number of column strips is $2m$, numbered as 0 at the side edge to m at the centre. The total number of beam strips is $2n$, numbered as 0 from the top edge to n at the centre. The widths of each column and beam strip are $\Delta x = b/2m$ and $\Delta y = a/2n$, respectively. As stated by Sanjayan and Manickarajah (1995), if the height of wall is greater than its width, the buckling strength of walls depends on the width rather than the height. This stems from the previous simplified method stipulated in AS3600-94 considering the effective height to be the lesser value of the distance between horizontal restraints or the distance between vertical restraints.

For TW3S panels, it is hypothesized that the wall is a half of the fictitious TW4S wall (as shown in Figure 6). Therefore, a half-height of the TW3S wall is considered for the analysis. The number of column strips adds up to a total of $2m$ and the number of beam strips adds up to a number of $2n$. The widths of each column and beam strip is therefore $\Delta x = b/m$ and $\Delta y = a/2n$, respectively. For TW action walls, the value of deflection at each segment, $Y_{i,j}$, is calculated by the assumed sinusoidal deflection shape with an initial midspan deflection $Y_{m,n}$:

$$Y_{i,j} = Y_{m,n} \sin\left(\frac{\pi i}{2m}\right) \sin\left(\frac{\pi j}{2n}\right) \quad (10)$$

The internal forces exerted on each grid point along the beams due to the deflection of the wall are calculated. The following equations are presented for the j^{th} beam. From finite difference:

$$\kappa_{i,j} = - \left(\frac{d^2 Y}{dx^2} \right)_{i,j} = - \frac{(Y_{i+1,j} - 2Y_{i,j} + Y_{i-1,j})}{(\Delta x)^2} \quad (11)$$

The curvature at mid beam $\kappa_{m+1,j}$ is determined by:

$$\kappa_{m+1,j} = \frac{(2Y_{m+1,j} - 2Y_{m,j})}{(\Delta x)^2} \quad (12)$$

The force at grid point (i, j) , $F_{i,j}$, is given by the following algorithm:

$$F_{i,j} = iF_{0,j} - \sum_{k=0}^{i-1} (i-k)F_{k,j} - (M_{(i+1),j} / \Delta x) \quad (13)$$

where at point 0, $\kappa_{0,j} = 0$ hence $M_{0,j} = 0$ (simply supported edge) and at point i , $M_{i,j}$ corresponding to $\kappa_{i,j}$ is obtained from the M - κ curve.

The derived internal resistances, $F_{i,j}$, acting on beams are exerted on the columns in the direction opposing the deflection, as shown in Figure 7. Therefore, the moment on the columns is a combination of moment due to equivalent axial load, equal end moment and moment due to the internal resistance $F_{i,j}$ which is given by Equation (13). The following iterative procedure is performed along the column strips. Moment at j^{th} point:

$$M_{i,j} = PY_{i,j} + M_{\max} + \sum_{k=1}^j (j-k)F_{i,k}\Delta y - j\Delta y F_{i,0} \quad (14)$$

The curvature $\kappa_{i,j}$ corresponding to $M_{i,j}$ is then obtained from M - κ curve. The new deflection, $Y_{i,j}$, at each station are obtained by numerically integrating the curvature and the slope.

$$Y_{i,j} = \sum_{q=1}^j \left[\sum_{p=n}^q \kappa_{i,q} \Delta y \right] \Delta y = \alpha_{i,j} \Delta y^2 \quad (15)$$

Having determined the midspan deflection of the middle column, a new value for Δy can be obtained as given in equation $\Delta y^{new} = \sqrt{Y_{m,n} / \alpha_{m,n}}$. This new value of Δy shall replace the previous one and is then used in Equation (14). The iteration proceeds until the difference between successive Δy values falls within the acceptable tolerance. Subsequently, a point on the height-midspan deflection curve is computed ($a = 2n\Delta y$). The whole process is then repeated to

find other points on the height-deflection curve. The initial midspan deflection, $Y_{m,n}$, is increased by a certain increment and the process is stopped when the moment at any section exceeds the ultimate moment capacity of the column strip. The peak value on the height-deflection curve yields the instability height of the wall section, for the particular axial load or failure load.

4. WASTABT VERSUS EXPERIMENTAL RESULTS

The effectiveness of the WASTABT is demonstrated through a comparison with the experimental data in terms of the ultimate failure loads. The OW and TW test results from previous research studies are used to verify the numerical outcomes. The dimensions, material properties and ultimate failure loads of these wall panels are summarized in Table 1.

For the analysis, 10 x 10, 10 x 10 and 20 x 10 meshes were used for a quarter of OW, a quarter of TW4S and a half of TW3S panels respectively (shown in Figure 8 for TAHS3 as an illustration). Figure 9 shows the typical comparisons of actual deflection profiles from experimental testing versus WASTABT output results. The ultimate load predictions are satisfactory as shown in Figures 10-11 and Table 2. The accuracy of code equations and WASTABT was evaluated using the following statistical indicators: the mean; the standard deviation (St Dev) which measures the amount of variation from the mean; the coefficient of variation (CoV) which shows the extent of variation and the coefficient of determination (R^2) that indicates how well the data fit a model within a 95% confidence interval. The ratios of the WASTABT load prediction to the experimental failure load varied from 0.75 to 1.24, with an overall mean of 0.94 and a standard deviation of 0.12 for OW panels. For TW panels, the ratio varied from 0.67 to 1.13, with an overall mean of 0.89 and a standard deviation of 0.14. Discrepancies between experimental and numerical results can be attributed to experimental and human errors combined with the idealistic nature of the numerical techniques. In testing, imperfections frequently exist such as possible dimensional variations, material irregularities,

concrete voids, changes in reinforcement location, and variations in restraint or loading conditions.

It can be seen in Figure 10 and Table 2 that, in contrast to the numerical model, the predicted ultimate strengths using the AS3600-09 and the ACI318-14 for OW walls were generally significantly lower than the test results, and further these formulae even failed to adequately predict the ultimate capacity of panels with high slenderness ratios. The ratios of predicted to actual strength ranged between 0.21 and 1.0, with an overall mean of 0.56 and a standard deviation of 0.23 for the AS3600-09, and between 0.29 and 2.00, with an overall mean of 0.71 and a standard deviation of 0.37 for the ACI318-14 (compared to the mean of 0.94 and standard deviation of 0.12 for WASTABT prediction). The AS3600 equation, nevertheless, provided less conservative predictions in a number of cases for TW action walls (as shown in Figure 11). The AS3600-09/test ultimate strength ratios varied from 0.54 to 1.70 with an overall mean of 0.88, and a standard deviation of 0.29 (compared to the mean of 0.89 and standard deviation of 0.14 for WASTABT prediction). This significant difference in standard deviation illustrates the variability of results using the AS3600 equation and reinforces the use of the WASTABT method which has low standard deviation.

Note that less conservative results were obtained when using the proposed model (545.8 and 685.6 kN) in comparison with the AS3600-09 equation (312.8 and 502.0 kN) for TW3S, although this is based on few experimental tests (TSN0 and TSH0 with ultimate capacities of 502.2 and 809.3 kN, respectively). Due to limited available test data in the literature, further verification of the proposed model is inevitably required for TW3S panels, once further testing is undertaken.

5. PARAMETRIC STUDY USING WASTABT

Since acceptable agreement between the experimental and WASTABT results has been observed, the numerical program was employed to conduct a parametric study. It was decided that TW panels would be the focus, by virtue of the fact that little treatment has been given in

research TW panel behaviours compared to OW panel. A comprehensive parametric study for OW panels using WASTABT was undertaken previously by Fragomeni and Mendis (1997).

In this study varying of slenderness ratios (H_w/t_w), aspect ratios (H_w/L_w), concrete strengths (f'_c), eccentricities (e), and reinforcement ratios (ρ_v) was considered. The thicknesses of walls were varied from 100 to 300 mm. Four different concrete strengths ($f'_c = 32, 50, 80$ and 100 MPa) were investigated. Steel reinforcement was assumed to have a yield stress of 450 MPa with a diameter of 12 mm. The axial strength ratio ($N_u/f'_c L_w t_w$), helping normalise results and identifying the effect particular variables (such as concrete strength, slenderness ratio) have, was used to compare the axial load capacity of the wall panels. The following observations can be made:

5.1 TW4S wall panels

5.1.1 Variation of slenderness ratios

To investigate the effects of slenderness ratio (H_w/t_w), which were varied between 10 and 60, axial load eccentricity was kept constant at $e = t_w/6$ along with $H_w/L_w = 1$ and $\rho_v = 0.31\%$. Figure 12 plots the variation of the axial strength ratio ($N_u/f'_c L_w t_w$) obtained from WASTABT with respect to various slenderness ratios for four different concrete strengths. It can be seen that the axial strength ratios significantly decreased with an increase in slenderness ratios, revealing the significant influence of the second order moment. In stocky walls ($H_w/t_w < 20$), crushing or material failure mode prevails. For panels with slenderness ratio around 20, an abrupt change in behaviour can be indisputably identified. That is for H_w/t_w beyond 20, the superimposed effects of the geometric and material nonlinearities trigger the acceleration of panel failure. In effect, as slenderness ratio increases, the more predominant collapse state of the panel is governed by buckling.

5.1.2 Variation of aspect ratios

To investigate the effects of aspect ratio, which were varied between 0.857 and 2.0, all panels were subjected to a constant eccentricity of $t_w/6$, with the reinforcement ratio being equal to 0.31%. Figures 13(a,b) were produced using constant H_w/t_w values equal to 40 and 60 respectively. The axial strength ratios ($N_u/f'_c L_w t_w$) were found to increase nonlinearly with increase in aspect ratios. This increase is due to the more significant contribution of the side restraints and the resulting two-way action of the panels. In varying H_w/L_w from 0.857 to 2.0, an average increase of 67.5% in axial strength ratio was obtained for H_w/t_w of 40. Comparatively, the average increase was 80.5% for H_w/t_w of 60.

5.1.3 Variation of eccentricities

The effects of load eccentricities, varied from $t_w/20$ to $t_w/3$, were investigated. The reinforcement ratio was equal to 0.0031 regardless of panel thickness and H_w/t_w varied from 20 to 60. Figure 14 indicates that axial load capacities are sensitive to changes in eccentricity. In general, as eccentricities increase, axial load capacities dramatically decline as expected. Intriguingly, for panels loaded with eccentricity of $t_w/3$, the failure mode was dominated by buckling for all slenderness ratios, which was not the case for the other eccentrically loaded panels of $t_w/6$, $t_w/10$ and $t_w/20$ appearing to be still be dominated by material failure at $H_w/t_w = 20$. Further, for panels with varying eccentricities and concrete strengths, it can be seen that the axial strength ratios descended nonlinearly with increasing slenderness ratios. Another interesting trend observed for eccentricity of $t_w/3$ is that its deceleration in axial strength ratios was less intense as H_w/t_w increased in comparison with the other eccentricities.

5.1.4 Variation of concrete strengths

Figure 14 also provides an observation of concrete strength. Table 3 was reproduced using the results from this figure. Table 3 presents the percentage ultimate strength increments due to concrete strengths with varying eccentricities. The advantage of HSC on wall strength is evident. When increasing the concrete strengths from 32 to 50 MPa (56.3% increase), 50 to 80

MPa (60% increase) and from 80 to 100 MPa (25% increase), wall strengths increased by about 28 to 55%, 22 to 54%, and 10 to 23%, respectively. However, the results indicate that the percentage increase in wall strength does not correspond to the same percentage increase in concrete strength. This is in contrast to the linear relationship suggested by the code simplified wall design equations. As discussed by Mendis (2003), HSC is structurally a distinct material and rules relevant to NSC are not always conservative once applied to HSC due to the variations in fracture modes, microstructure and the differences brought about by various additives. Further, Table 3 also shows that concrete strengths have a significant effect on wall strength at lower slenderness ratios, but that effect diminishes as the walls become more slender. Distinctively, for concrete strengths of 80 and 100 MPa with varying eccentricities approaching a slenderness ratio of 60, the effect of HSC on the ultimate wall strength increments almost vanished. It is apparently reflected by the roughly unchangeable load ratios of 1.23 for the concrete strength increment from 50 to 80 MPa and 1.10 for the increment from 80 to 100 MPa.

5.1.5 Variation of reinforcements

To investigate the RC sections for reinforcement effects, reinforcement was placed centrally in single layer or symmetrically in double layers. Using a constant $H_w/L_w = 1$ and $e = t_w/6$ for all the panels, the reinforcement ratio ρ_v was increased from 0.15 to 1% with results presented in Figure 15. Figure 15 gives the axial strength ratios versus reinforcement ratio for four different concrete strengths with H_w/t_w varied from 20 to 40. In the case of reinforcement placed in both faces of the wall section, the clear cover to reinforcement in each face was 20 mm.

For singly reinforced sections, it was found that the axial strength ratios of the walls were almost the same with increasing the amount of reinforcement. There is insufficient effective depth to the tension steel for centrally placed reinforcement; as such the contribution of steel reinforcement to load capacity showed negligible influence.

For doubly reinforced sections, the load carrying capacity of walls showed significant increases in strength when increasing the reinforcement content. Taking the $f'_c = 32$ MPa wall

panel for example, the increased axial strength ratio appeared to be directly proportional to the increase in reinforcement ratio. Wall strengths increased by roughly 18.0% for $H_w/t_w = 20$ and 8.8% for $H_w/t_w = 30$ when increasing ρ_v from 0.0015 to 0.01. In spite of this, the contributions of increased reinforcement ratio were less obvious under the effects of increasing the concrete strength and the slenderness ratio. In particular, the effect of double-layered reinforcement ratios was a moderate wall strength increase when HSC was used. Finally, in cases of walls with H_w/t_w of 40, the increased reinforcement ratios contributed minimally to the increase in wall strengths, being slightly higher than the results of singly reinforced sections.

5.2 TW3S wall panels

As aforementioned, due to limited amount of experimental test data, the proposed model for TW3S panels needs to be further validated. As a result, only certain parameters were investigated in this section to highlight the capability of the proposed model as compared to the Australian and American Standards in predicting the behaviours of this type of wall. All the model panels were subjected to loads with eccentricity of $t_w/6$ and with the centrally placed reinforcement ratio being equal to 0.0031. Two slenderness ratios of 30 and 40 along with a concrete strength of 32 MPa were studied with variation in aspect ratios (H_w/L_w) from 0.4 to 2. Results of this analysis are presented in Figure 16. The conservative nature of the code formulae is evident. In particular, the code equations give negative values, indicating zero load-bearing capacity, for OW walls with H_w/t_w ratios greater than 30, which is clearly not the case according to the experimental test results presented in the Section 4 of this paper. In addition, it was found that the Australian simplified wall method also predicted zero strength for TW3S and TW4S walls of $H_w/t_w = 40$ once the aspect ratios were varied, particularly when $H_w/L_w \leq 1.5$ for TW3S and $H_w/L_w \leq 0.5$ for TW4S.

In using WASTABT, a more logical and rational behaviour of wall panels has been observed. As evidenced by the figures, when the values of aspect ratio become smaller and smaller, the axial strengths ratios of TW3S and TW4S panels converge towards the values of

axial strength ratio of OW action walls. This implies that the behaviours of TW3S and TW4S panels gradually approach the behaviours of OW panels. The reason is because when the panels are extremely long, the effect of the side restraints consequently becomes insignificant or the panels shall buckle locally somewhere between the restraints as in the behaviour of OW panels do likewise.

6. CONCLUSION

The numerical analysis, accounting for both material and geometrical nonlinearities, was incorporated into the computer program WASTABT, which can be used to predict the axial load behaviour of wall panels with various support conditions. This analysis gives more reliable and generally less conservative results than the AS3600-09 and ACI318-14 empirical equations, which was validated via a comparative study with a number of existing experimental results from various researchers. It should be highlighted that the proposed analytical method for TW3S panels is capable of estimating the load capacities, and while the results were in good agreement with the limited test data, further validation of this is desired.

A parametric study was subsequently undertaken to investigate the effects of varying panel properties for TW4S wall panels. The study showed that:

- $N_u/f'_c L_w t_w$ reduces with an increase in the slenderness ratio (H_w/t_w) of the walls. The decrease in strength is more pronounced for H_w/t_w beyond 20 as a consequence of combination of both geometrically and materially induced failure.
- $N_u/f'_c L_w t_w$ increases nonlinearly with an increase in aspect ratio.
- $N_u/f'_c L_w t_w$ is also sensitive to changes in eccentricity. As eccentricities increase, axial load capacities dramatically decline.
- Considerable strength increases occur when two layers of reinforcement placed symmetrically in each face are used in comparison with single layer of reinforcement placed centrally. However, the increase in wall strength does not appear to be significant for HSC walls with high slenderness ratios.

The effects of various aspect ratios for walls with different support conditions were also investigated. In using WASTABT, a more logical and rational behaviour of wall panels has been observed compared to the code of standards. For TW3S and TW4S panels, the study indicated that side restraints greatly increase ultimate strength of walls. In spite of this, if wall panels are rather long, the effects of lateral supports shall gradually diminish causing TW3S and TW4S panels to approach the behaviours of OW ones.

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Table 1. Dimensions, concrete strengths and experimental failure loads of RC panels with various support conditions.

Author	Support condition	Panel designation	H_w (mm)	L_w (mm)	t_w (mm)	f'_c (MPa)	$\frac{H_w}{t_w}$	$\frac{H_w}{L_w}$	N_u (kN)
Saheb and Desayi (1989)	OW	WAR-1	600	900	50	17.9	12.0	0.67	484.0
		WAR-2	600	600	50	17.9	12.0	1.00	315.0
		WAR-3	600	400	50	17.9	12.0	1.50	198.0
		WAR-4	600	300	50	17.9	12.0	2.00	147.0
		WSR-2	600	400	50	17.3	12.0	1.50	254.0
		WSR-3	900	600	50	17.3	18.0	1.50	299.0
Saheb and Desayi (1990)	TW4S	WAR-1(P)	600	900	50	17.9	12.0	0.67	556.0
		WAR-2(P)	600	600	50	17.9	12.0	1.00	413.5
		WAR-3(P)	600	400	50	17.9	12.0	1.50	284.9
		WAR-4(P)	600	300	50	17.9	12.0	2.00	235.2
		WSR-4(P)	1350	900	50	17.3	27.0	1.50	534.0
		WSTV-2(P)	600	900	50	20.1	12.0	0.67	597.8
		WSTV-5(P)	1200	900	50	18.3	24.0	1.33	498.2
Fragomeni (1995)	OW	3a	1000	200	40	37.1	25.0	5.00	100.0
		4b	1000	300	40	54.0	25.0	3.33	217.0
		5a	1000	500	40	35.7	25.0	2.00	201.0
		5b	1000	500	40	59.7	25.0	2.00	269.0
	TW4S	11a	600	200	40	37.3	15.0	3.00	198.2
		11b	600	200	40	68.7	15.0	3.00	296.2
		12a	420	210	35	37.3	12.0	2.00	208.8
		12b	420	210	35	73.3	12.0	2.00	271.2
Sanjayan and Maheswaran (1999)	TW4S	1	2000	1500	50	65.0	40.0	1.33	524.4
		4	2000	1500	50	96.0	40.0	1.33	560.5
		6	2000	1500	50	65.0	40.0	1.33	1510.2
		7	2000	1500	50	82.5	40.0	1.33	647.3
		8	2000	1500	50	83.0	40.0	1.33	1532.6
Sanjayan (2000)	OW	1	2000	1500	50	58.5	40.0	1.33	238.0
		2	2000	1500	50	59.0	40.0	1.33	202.0
		3	2000	1500	50	59.0	40.0	1.33	212.0
Doh (2002)	OW	OWNS3	1400	1400	40	52.0	35.0	1.00	427.0
		OWNS4	1600	1600	40	51.0	40.0	1.00	442.0
		OWHS2	1200	1200	40	78.2	30.0	1.00	483.0
		OWHS3	1400	1400	40	63.0	35.0	1.00	442.0
		OWHS4	1600	1600	40	75.9	40.0	1.00	456.0
	TW4S	TWNS2	1200	1200	40	37.0	30.0	1.00	735.8
		TWNS3	1400	1400	40	51.0	35.0	1.00	1177.2
		TWNS4	1600	1600	40	45.8	40.0	1.00	1177.2
		TWHS2	1200	1200	40	64.8	30.0	1.00	1177.2
		TWHS3	1400	1400	40	60.1	35.0	1.00	1250.8
		TWHS4	1600	1600	40	70.2	40.0	1.00	1648.1
		TAHS1	1600	1400	40	77.8	40.0	1.14	1618.7
		TAHS2	1400	1000	40	77.8	35.0	1.40	1118.3

		TAHS3	1600	1200	40	73.8	40.0	1.33	1265.5
		TAHS4	1600	1000	40	77.8	40.0	1.60	1442.1
Doh <i>et al.</i> (2008)	TW3S	TSN0	1200	1200	40	50.1	30.0	1.00	502.2
		TSH0	1200	1200	40	80.4	30.0	1.00	809.3
Robinson <i>et al.</i> (2013)	OW	10	2800	500	100	52.4	28.0	5.60	683.0
		11	3000	500	100	51.6	30.0	6.00	582.0
		12	3000	500	100	51.6	30.0	6.00	597.0
		13	3000	500	100	51.6	30.0	6.00	572.0
		14	3000	500	100	51.6	30.0	6.00	568.0
		15	3000	500	100	52.4	30.0	6.00	322.0
		16	3000	500	100	52.4	30.0	6.00	336.0
Huang <i>et al.</i> (2014)	OW	ST3	2700	460	100	81.4	27.0	5.87	1274.0
		ST4	2700	460	100	81.4	27.0	5.87	297.0
		ST5	2700	460	130	81.4	20.8	5.87	1427.0
		ST6	2700	460	160	81.4	16.9	5.87	1882.0
		ST7	2700	460	100	81.4	27.0	5.87	846.0
		ST8	2700	460	100	81.4	27.0	5.87	839.0

Table 2. Statistical summary for comparison of average ratio between theoretically and experimentally determined capacities for OW and TW walls.

Model	OW walls				TW walls			
	Mean	St Dev	CoV (%)	R ²	Mean	St Dev	CoV (%)	R ²
AS3600-09	0.56	0.23	40.4	0.85	0.88	0.29	32.6	0.82
ACI318-14	0.71	0.37	51.9	0.78	0.52	0.24	45.8	0.05
WASTABT	0.94	0.12	13.1	0.98	0.89	0.14	15.2	0.88

Table 3. Percentage ultimate strength increment due to concrete strength ($H_w/L_w = 1$, $\rho_v = 0.31\%$).

Concrete strength (MPa)	$\frac{H_w}{t_w}$	Ultimate load increment (%)							
		Failure load (kN)				Load ratio			
		$t_w/20$	$t_w/10$	$t_w/6$	$t_w/3$				
32 MPa	20	44529.0	39510.7	33434.8	17625.6				
	30	23794.6	21003.8	16972.8	8943.3				
	40	13635.4	11774.9	9400.3	5226.5				
	60	5197.9	4510.8	3688.3	2320.7				
$N_u(50 \text{ MPa})/N_u(32 \text{ MPa})$									
50 MPa	20	69003.0	60282.0	50990.0	25474.0	1.55	1.53	1.53	1.45
	30	36006.0	31492.5	24939.0	12138.0	1.51	1.50	1.47	1.36
	40	19603.1	16688.5	13005.0	6885.0	1.44	1.42	1.38	1.32
	60	6885.0	5910.0	4794.0	2970.8	1.32	1.31	1.30	1.28
$N_u(80 \text{ MPa})/N_u(50 \text{ MPa})$									
80 MPa	20	106365.6	91922.4	77724.0	34516.8	1.54	1.52	1.52	1.35
	30	52224.0	48880.0	34190.4	15504.0	1.45	1.43	1.37	1.28
	40	26089.6	21817.8	16707.6	8629.2	1.33	1.31	1.28	1.25
	60	8441.5	7242.0	5875.2	3635.3	1.23	1.23	1.23	1.22
$N_u(100 \text{ MPa})/N_u(80 \text{ MPa})$									
100 MPa	20	130815.0	111873.6	94645.8	39305.7	1.23	1.22	1.22	1.14
	30	61710.0	52326.0	39066.0	17340.0	1.18	1.17	1.14	1.12
	40	29376.0	24342.3	18513.0	9562.5	1.13	1.12	1.11	1.11
	60	9246.3	7930.5	6446.4	3993.3	1.10	1.10	1.10	1.10

Figure 1. Walls with various support conditions (Doh *et al.*, 2008).

Figure 2. Moment-curvature analysis of a wall section (Doh, 2002).

Figure 3. Equivalent end moment and centric load (Fragomeni and Mendis, 1997).

Figure 4. Half-height wall with assumed deflection (Fragomeni and Mendis, 1997).

Figure 5. Strip layout in RC panels (Sanjayan and Manickarajah, 1995).

Figure 6. Proposed TW3S wall model as half of the fictitious TW4S wall.

Figure 7. Modelling of wall in beam and column cases (Sanjayan and Manickarajah, 1995).

Figure 8. Mesh generation of TAHS3 (All dimensions are in mm).

(a) Experiment (Doh, 2002)

(b) Experiment (Doh *et al.*, 2008)

(c) Experiment (Doh, 2002)

(a) WASTABT

(b) WASTABT

(c) WASTABT

Figure 9. Deflected shape of walls (a) OWS4; (b) TSN0; and (c) TAHS3.

Figure 10. Theoretical predictions of code equations and WASTABT versus test results for OW walls.

Figure 11. Theoretical predictions of code equations and WASTABT versus test results for TW walls.

Figure 12. Axial strength ratio versus slenderness ratio of RC wall ($H_w/L_w = 1$, $e = t_w/6$, $\rho_v = 0.31\%$).

Figure 13. Axial strength ratio versus aspect ratio of RC wall ($e = t_w/6$, $\rho_v = 0.31\%$).

(a) $e = t_w/3$

(b) $e = t_w/6$

(c) $e = t_w/10$

(d) $e = t_w/20$

Figure 14. Influence of eccentricity on axial load.

(a) $f'_c = 32$ MPa

(b) $f'_c = 50$ MPa

(c) $f'_c = 80$ MPa

(d) $f'_c = 100$ MPa

Figure 15. Axial strength ratio versus vertical reinforcement ratio for RC wall.

(a) $H_w/t_w = 30$

(b) $H_w/t_w = 40$

Figure 16. Axial strength ratio versus aspect ratio for RC wall.