Prediction of Axial Load Capacity of Concrete Walls with Openings Restrained on Three Sides

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ABSTRACT

Currently, the empirical wall design equations given in major codes of practices either offer no guidelines or provide overly conservative predictions of the ultimate axial strength of concrete walls with openings restrained on three sides (TW3S walls with openings). In view of the significant shortcomings, an appropriate design equation is required for such walls. This paper presents a numerical investigation on the behaviour of TW3S walls with openings under eccentric axial loading. A finite element model using the ABAQUS software was employed to undertake parametric studies examining the influence of opening parameters, which consisted of configuration and location, on the axial load capacity of TW3S walls. In total, 101 models were developed and analysed. Having established the relationships between the opening parameters and the ultimate load capacity of TW3S walls, a more encompassing design

equation was derived. The reliability of the proposed equation was ascertained through comparisons with available test data.

KEYWORDS: concrete wall, eccentric axial loading, opening, code method, finite element analysis

1. INTRODUCTION

Cast in situ or precast reinforced concrete (RC) walls are commonly used in multi-storey buildings to withstand gravitational and lateral loadings. In many circumstances, walls are pierced with openings because of architectural requirements or functional modifications of the structures. For instance, openings may be required for the provision of doors and windows in tilt-up construction, and for services or safety reasons in core walls [1]. The presence of openings introduces geometric discontinuity into the walls, which affects the nonlinear stress distribution and may reduce the element's ultimate load capacity, stiffness and energy dissipation [2].

Walls with openings subjected to eccentric axial loads can be constructed with three types of support conditions. The hypothetical deflection and cracking characteristics at failure of such walls are illustrated in Fig. 1. Currently, axially-loaded walls can be designed using simplified design equations, provided in major codes of practice, which include, inter alia, the Eurocode 2 (EC2-04) [4], the American Concrete Institute Code (ACI318-19) [5], and the Australian Concrete Standard (AS3600-18) [6]. Nonetheless, the design of walls with openings is given little guidance in the design codes. When designing a wall with openings, engineering practitioners make the judgement to split the wall into segments of columns connected by cross beams at the openings, which requires the design of each individual segment. Despite being

reasonably acceptable, it will be more convenient if practitioners are provided with simplified equations to evaluate the ultimate capacity of the walls [7].

Numerous studies have been undertaken on the behaviour of walls with openings under one-way action (OW) and two-way action walls supported on four sides (TW4S). Seddon [8] and Zielinski et al. [9] were the pioneers of the initial experimental work on such panels. These introductory studies were the catalyst for Saheb and Desayi [10] to conduct the first systematic experimental study of axially-loaded OW and TW4S walls with openings. Since then, a number of experimental and numerical studies have been undertaken [1, 2, 7, 11-15]. Most of these studies also attempted to develop simplified design models capable of predicting the axial load capacity of in-plane vertical loaded walls.

Compared to the extensive literature on axially-loaded OW and TW4S walls with openings, only a small number of experiments [3, 16-18] have been performed on the axial load capacity of walls with openings under two-way action supported on three sides (TW3S). There is therefore insufficient data to draw quantitative conclusions about, and to derive design models for, the axial load capacity of TW3S walls with openings. Consequently, a finite element model (FE-Model) using the ABAQUS software, was employed in this research to further investigate the behaviour of TW3S walls with openings. Extensive parametric studies were undertaken to examine the effects of key parameters on the ultimate axial strength of TW3S walls with various opening configurations. These include the size of openings and the effect of varying the opening locations. In total, 101 numerical models were developed and analysed in this study. Utilising these and published test results, a simplified design formula was derived for predicting the ultimate strength of TW3S walls with openings.

2. SIMPLIFIED WALL DESIGN EQUATIONS

2.1. Code design equations

ACI318-19 [5] does not provide the methodology to account for the increase in axial load capacity of walls due to the presence of side restraints, thus the ACI318 equation was not presented in this paper. The EC2 and AS3600 design equations to predict the design axial strength of a wall are given by Eqs. (1) and (2), respectively. The symbols used in these equations and other equations throughout this paper are defined in Notation.

$$N_d = \Phi \frac{f'_c}{\gamma_c} L_w t_w \quad (1)$$

in which

$$\Phi = 1.14 \times \left(1 - 2 \frac{e + e_a}{t_w} \right) - 0.02 \times \frac{H_{we}}{t_w} \leq \left(1 - 2 \frac{e + e_a}{t_w} \right)$$

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

where ϕ (an equivalent value) = 0.67 for EC2-04 [4] ($\phi = 1/\gamma_c = 0.67$, where γ_c is equal to 1.5 for the ultimate limit state in the case of concrete under axial load), and $\phi = 0.65$ for AS3600-18 [6]; $e_a = H_{we}/400$ for EC2-04 [4], and $e_a = H_{we}^2/(2500t_w)$ for AS3600-18 [6]. The effective height shall be taken as $H_{we} = kH_w$ in which k is determined for various support conditions as follows:

- a. For OW walls, $k = 0.85$ for EC2-04 [4] and $k = 0.75$ for AS3600-18 [6], when walls are restrained against rotation at both ends, and $k = 1.0$ for both design codes [4, 6], when walls are not restrained against rotation at one or both ends.
- b. For TW3S walls, $k = 1/[1 + (H_w/3L_w)^2]$ for both design codes [4, 6] but AS3600-18 [6] further specifies that the value of k is not less than 0.3;
- c. For TW4S walls, $k = 1/[1 + (H_w/L_w)^2]$ when $H_w \leq L_w$ and $k = L_w/2H_w$ when $H_w > L_w$ for both design codes [4, 6].

AS3600-18 [6] only allows the effects of openings in a TW4S wall to be neglected if the total area of openings does not exceed 10% of the wall area, and if the height of any

opening, not vertically above another opening, does not exceed one-third of the wall height. However, EC2-04 [4] allows for walls with various support conditions. For openings that do not comply with these conditions, the wall area between the opening and the support is designed to be supported on three sides, and the area between two openings is designed to be supported on two sides [6].

According to EC2-04 [4], an intersecting wall may be considered as a lateral restraint if:

- its total thickness is not less than $0.5t_w$, where t_w is the overall thickness of the restrained wall;
- it has the same height H_w as the restrained wall under consideration;
- its length is at least equal to $0.2H_w$, where H_w denotes the clear height of the restrained wall;
- within the length of $0.2H_w$, the intersecting wall has no openings.

However, AS3600-18 [6] only stipulates that an intersecting wall with a minimum length of $0.2H_w$ may be considered a lateral restraint.

2.2. Ho and Doh (2019) equation for TW3S solid walls

Ho and Doh [18, 19] conducted experimental tests on TW3S walls with slenderness ratios (H_w/t_w) varying from 25 to 40, and aspect ratios (H_w/L_w) from 0.83 to 2.0. These studies led to the conclusion that the AS3600-18 equation was inadequate in predicting the ultimate load capacity for TW3S walls and a reliable design equation would be needed. Consequently, Ho and Doh [19] proposed a more encompassing design equation for TW3S solid walls, which is a modified version of the AS3600 equation, as follows:

$$N_u = 4.5 f_c'^{0.45} L_w (t_w - 1.2e - 2e_a) \quad (3)$$

where $e_a = [1/(H_w/t_w)^{1.26}] \times (H_w^2/50t_w)$.

Comparisons with the available test results confirmed that the proposed equation is satisfactory and reliable.

2.3 Design equations for walls with openings

The commonly adopted method to derive an equation for estimating the ultimate strength of walls with openings, under OW and TW4S in particular, is to incorporate a nondimensional parameter arising out of the geometry of the openings into an equation used for solid wall panels, defined by a linear extrapolation using a regression analysis [7, 10, 11,13] as follows:

$$N_{uo} = (k_1 - k_2\chi)N_u \quad (4)$$

In the studies [7, 10, 11], only the influence of the size and location of the opening in the horizontal direction was considered through the opening parameter $\chi = \chi_x$, defined as:

$$\chi_x = \frac{A_{ox}}{A_x} + \frac{\eta_x}{L_w} \quad (5)$$

$$\text{where } A_{ox} = L_o t_w; A_x = L_w t_w; \eta_x = \frac{L_w}{2} - \left(\frac{\frac{L_w^2 t_w}{2} - L_o t_w \eta_{ox}}{L_w t_w - L_o t_w} \right).$$

Guan et al. [13], however, found that increasing both the length and the height of an opening has the most significant influence on the capacity. To further account for the influence of the opening height, these scholars proposed the opening parameter $\chi = \chi_{xy}$, calculated as follows:

$$\chi_{xy} = \frac{\chi_x + \lambda \chi_y}{1 + \lambda} \quad (6)$$

The opening parameter, χ_x is identical to Eq. (5). The opening parameter with respect to the influence of opening size and location in the vertical direction, χ_y , is defined as:

$$\chi_y = \frac{A_{oy}}{A_y} + \frac{\eta_y}{H_w} \quad (7)$$

$$\text{where } A_{oy} = H_o t_w; A_y = H_w t_w; \eta_y = \frac{H_w}{2} - \left(\frac{\frac{H_w^2 t_w}{2} - H_o t_w \eta_{oy}}{H_w t_w - H_o t_w} \right).$$

Further information regarding the symbols used in Eqs. (5) to (7) can be referred to Fig. 2.

3. WALL SPECIMENS

Eight (8) experimental test results by Ho and Doh [18] were selected for this study from which a reliable model for parametric investigation could be established. The geometric and material properties of these test panels are given in Table 1(a). A single F41 mesh (4 mm diameter and 100 mm spacing) placed centrally within the panel cross-section allowed the elements to be treated as unreinforced elements, with no contribution to their capacity from the steel reinforcement [18]. The primary purpose of minimum wall reinforcement detailing was to control cracking due to shrinkage and temperature stresses. The walls were simply supported on three sides under as-built restraints and subjected to a vertical load on top of the wall with an eccentricity of $t_w/6$. Information regarding crack patterns, axial load versus lateral deflection curves and ultimate loads were collected during testing. A detailed description of the experimental program and test results can be found in the original paper [18].

4. NUMERICAL INVESTIGATION

4.1. The FE-Model

The numerical simulation provides an effective means of investigating various structural elements due to more flexibility in modifying the model as well as being cost and time-saving in comparison to labour-intensive and costly laboratory tests [20]. Among the available tools for numerical investigation, a finite element package, ABAQUS, was chosen for modelling because of its widely-known ability to perform nonlinear analyses for RC members. The

numerical simulation was developed and verified against the experimental results in the previous study [18]. Since the development of the FE-Model is not the main concern of this study but employed to investigate the behaviour of TW3S walls with openings, the FE-Model is only briefly presented for the brevity purpose.

4.1.1. Material properties and constitutive models

Concrete was modelled using the concrete damage plasticity (CDP) model. The key factors of this model are: damage variable, yield criterion, flow rule and viscous parameter. The tensile fracture and compressive plastic behaviour are completed by specifying the evolution laws of damage, which represents the stiffness degradation. The damage index varies from 0 to 1 representing no damage to complete failure. The compression and tension damages, d_c and d_t respectively, were calculated by the equations below [21-23]:

$$d_c = 1 - \frac{\sigma_c}{\sigma_{cu}} \quad (8)$$

$$d_t = 1 - \frac{\sigma_t}{\sigma_{t0}} \quad (9)$$

The concrete behaviour in compression and tension was simulated using the proposed models by Lu and Zhao [24] and Fields and Bischoff [25], respectively. The Poisson's ratio of concrete was taken as $\nu_c = 0.2$. These constitutive relations were chosen in this study because of their capability to satisfactorily predict the behaviour of axially loaded wall panels with various concrete strengths, as demonstrated in previous studies [18, 26, 27].

The CDP model allows defining the shape of the yield surface by means of the two parameters K_c and $\sigma_{b0} / \sigma_{c0}$: the first parameter describes the shape of the deviatoric plane, while the latter describes the ratio between the initial equibiaxial to the uniaxial compressive yield stress. The yield function is defined as:

$$F = \frac{1}{1-\alpha} \left(\bar{q} - 3\alpha \bar{p} + \beta \left(\tilde{\varepsilon}^{pl} \right) \left\langle \hat{\sigma}_{\max} \right\rangle - \gamma \left\langle -\hat{\sigma}_{\max} \right\rangle \right) - \bar{\sigma}_c \left(\tilde{\varepsilon}_c^{pl} \right) = 0 \quad (10)$$

$$\text{where } \alpha = \frac{(\sigma_{b0} / \sigma_{c0}) - 1}{2(\sigma_{b0} / \sigma_{c0}) - 1}; \quad 0 \leq \alpha \leq 0.5; \quad \beta = \frac{\bar{\sigma}_c \left(\tilde{\varepsilon}_c^{pl} \right)}{\bar{\sigma}_t \left(\tilde{\varepsilon}_t^{pl} \right)} (1 - \alpha) - (1 + \alpha); \quad \text{and } \gamma = \frac{3(1 - K_c)}{2K_c - 1}.$$

According to Blazejowski [28], if the response of the models is solely uniaxial, there are insignificant changes to the analytical results when the two parameters, K_c and $\sigma_{b0} / \sigma_{c0}$, are varied. The default values, $K_c = 2/3$ and $\sigma_{b0} / \sigma_{c0} = 1.16$, in the ABAQUS user's manual [29] were thus used in this study.

The CDP model assumes non-associated potential plastic flow. The flow potential follows the Drucker-Prager hyperbolic function. The flow potential eccentricity ξ and the dilation angle ψ are required to define the function:

$$G = \sqrt{(\xi \sigma_{io} \tan \psi)^2 + \bar{q}^2} - \bar{p} \tan \psi \quad (11)$$

The possible dilation angle ψ ranges from 0° to 56° . The friction angle of concrete, ϕ , can be correlated with the dilation angle of concrete, ψ , using an equation given as $\sin(\phi) = \tan(\psi)$ [30, 31]. Jiang and Wu [32] derived a formula predicting the friction angle of unconfined concrete, which provided results close to the value of approximately 37° . The dilation angle, $\psi = 31^\circ$, was thus used in the present study. This value has also been utilised by other researchers in their FE-Model investigations [31, 33, 34].

Defining a minute value (close to zero) for the viscosity parameter μ can help to avoid the convergence difficulties, without compromising results [29]. In this study, a small viscosity parameter $\mu = 10^{-4}$ was used. Similar approaches have also been used in previous studies [21, 34].

A summary of the CDP model parameters used for the numerical analyses is presented in Table 2. Background details of these parameters can be further found in the ABAQUS user's manual [29].

For the steel reinforcement, the stress-strain curve was assumed to be the elastic-perfectly plastic material and identical in compression and tension. The elastic modulus and the Poisson's ratio for steel were taken to be $E_s = 210$ GPa and $\nu_s = 0.3$, respectively. The stress-strain curve of the steel restraints was assumed to be elastic material with the same E_s and ν_s as those of the steel reinforcement.

4.1.2. Numerical analysis and nonlinear solution

Eight-node brick elements with reduced integration and hourglass control (C3D8R elements) were used to model the concrete wall and the restraints. Discrete truss elements in linear order (T3D2 elements) were used to model the steel reinforcement. The steel reinforcement was used for other than strength purposes and the placement of minimum reinforcement at the mid-depth of the walls has little effect on axial load capacity [18]. For the sake of simplicity, the reinforcement was simulated as embedded in the concrete region with a perfect bond, and initial imperfection of the steel reinforcement was not considered. For sophisticated modelling techniques of initial imperfection for steel bars, readers can refer to the available publication [35]. The rough friction formulation was implemented in order to prevent the wall from losing contact with the restraints, so that boundary conditions could be correctly applied. In accordance with the hard contact simulation used, both the RC wall and the restraints were assumed to be impenetrable rigid bodies.

The modified Riks method is usually used to predict the unstable, geometrically nonlinear collapse of a structure – it uses an arc-length method to determine the response of the loaded structure, where there are significant changes in the structure's stiffness [36]. In this study, the

modified Riks method was employed to analyse all the numerical models for the purpose of finding the ultimate strength of the walls. The load was applied in increments using the modified Riks method available in the ABAQUS library, through the reference point on the top restraint [37]. The simulations are done with the initial arc length increment of 0.05. Automatic stability was used to avoid divergence solutions, and the Nlgeom was activated to account for the geometric nonlinearity. The analysis was terminated when the nominated node displacement was achieved. Ho and Doh [18] conducted mesh convergence studies to arrive at appropriate meshes that provided a relatively accurate solution with low computational time. A typical mesh sensitivity study for the TW3S-NF wall panel is presented, for perusal, in Table 3. The geometric and material properties of the panel is given in Table 1(b). To investigate the mesh sensitivity in the concrete section, the sizes of mesh for other parts, consisting of reinforcement and restraints, were maintained. The global mesh size of 20 for H_w and L_w and the local mesh size of 3 for t_w provided good results in terms of accuracy and computational time. The aspect ratio of the structured elements was close to 1:1.5:1.5 (thickness:length:height). Aspect ratios between 1.0 and 4.0 are considered acceptable [15, 38]. Fig. 3 shows the finite element idealisation and meshing. Similar meshing strategies for RC members with openings were deployed in previous research as well [14, 15, 20, 21, 39-41].

4.2. Model verification

The effectiveness of the numerical simulation was demonstrated through a comparison with the testing results in the study by Ho and Doh [18] with respect to crack patterns, load-deflection profiles, and ultimate loads. In the numerical models, the crack patterns were visualised through the maximum principal plastic strains. Fig. 4 shows typical examples of the crack patterns reasonably well predicted by the numerical simulations. The comparison of the load-deflection response between the numerical and experimental results is shown in Fig. 5.

The numerical load-deflection paths are typified by linear portions, with the linearity of the curves up to 40% to 55% of the ultimate loads, followed by an increasing non-linear curve. The trends of the load-deflection responses in the numerical models and experimental tests were similar. The gradients of the load-deflection responses predicted by the FE-Model, however, were steeper than those of the experimental results. The ultimate loads predicted by the FE-Model are presented in Table 1(a). The ultimate load predictions ($N_{u,FEM}$) were satisfactory but slightly overestimated. The results indicated that the ratios of the FE analysis to experimental failure loads ($N_{u,FEM}/N_{u,EXP}$) varied from 0.89 to 1.23, with a mean of 1.06, and a standard deviation of 0.11. These discrepancies are expected due to the experimental and human errors combined with the idealistic nature of the FE-Model. In testing, imperfections often exist such as possible dimensional variations, and the variations in restraint or loading conditions.

In general, the numerical outcomes showed close correlation with the experimental data and observations, and thus confirmed the effectiveness of the modelling techniques in simulating the responses of TW3S walls with openings subjected to eccentric axial loading. The verified numerical model was subsequently employed to investigate the influences of varying the opening configuration and opening location on the axial load capacity of TW3S walls.

4.3 Parametric study

Concrete walls of a 100-mm-thick cross-section with a height of 3000 mm ($H_w/t_w = 30$) and three aspect ratios (H_w/L_w) of 0.5, 1.0 and 2.0 were used with various opening configurations. In the analysis, the concrete strength was assumed to be 50 MPa. Each concrete wall model had a single layer of steel reinforcement centrally placed which was assumed to contain reinforcement bars with a diameter of 12 mm and yield stress of 450 MPa with $\rho_v = \rho_h = 0.31\%$. The axial load eccentricity was kept constant at $e = t_w/6$. Details of the wall dimensions including the opening configurations are shown in Tables 4 and 5.

4.3.1. Effect of opening configuration

The effect of the opening configuration on the ultimate strength of the walls was analysed through three stages: 1) varying opening size (varying the opening height and length simultaneously); 2) varying opening height (H_o); and 3) varying opening length (L_o). All the wall models had a centrally located opening, either square or rectangular in shape.

The influence of varying the opening size is illustrated in Fig. 6 which shows the axial strength ratio ($N_{uo}/f'_c L_w t_w$) versus the dimensionless size parameter (A_o/A_w), where $A_o = H_o \times L_o$ and $A_w = H_w \times L_w$. The parameter A_o/A_w varied from 0.05 to 0.4 for walls with $H_w/L_w = 0.5$ (increasing the opening size from 949 mm $\times$ 949 mm to 2683 mm $\times$ 2683 mm) and for walls with $H_w/L_w = 1.0$ (increasing the opening size from 671 mm $\times$ 671 mm to 1897 mm $\times$ 1897 mm), and from 0.05 to 0.25 for walls with $H_w/L_w = 2.0$ (increasing the opening size from 474 mm $\times$ 474 mm to 1061 mm $\times$ 1061 mm). As can be seen in Fig. 6, the axial strength ratio appeared indirectly proportional to the opening size. The axial strength ratio decreased linearly with the increase in opening size, indicated by good values of the correlation coefficient (R^2). Observation of the trendlines further highlighted that increases in the size of the opening had the greatest impact on the decreased axial strength ratio of walls with $H_w/L_w = 2.0$ (with the gradient value of 0.548), the second greatest impact on walls with $H_w/L_w = 1.0$, and the least impact on walls with $H_w/L_w = 0.5$ (with the gradient values of 0.325 and 0.163, respectively). According to Popescu et al. [2], a wall with openings can be idealised as a hybrid system with the column elements behaving as a parallel system connected in series with the beam elements. These scholars stated that a parallel system with brittle elements will collapse if one of the brittle components fails. As opening size increases for TW3S walls with a centrally located opening, not only are perceived advantages gained from having the side restraint offset as a consequence of the TW behaviour being interrupted by the opening, the column element at the

free end tends to exhibit more OW behaviour. The aspect ratio (H_w/L_w) of the column element at the free end of the walls with $H_w/L_w = 0.5$ and 1.0 increased from 1.19 to 1.81 and from 2.58 to 5.44 , respectively, when A_o/A_w varied from 0.05 to 0.4 . Noting that the aspect ratio of the column element at the free end of the walls with $H_w/L_w = 2$ rapidly increased from 5.85 to 13.67 when A_o/A_w varied from 0.05 to 0.25 . As discussed by Ho and Doh [18], for OW walls the ultimate strength shows a decreasing trend with an increase in aspect ratio. Altogether, these geometric characteristics account for the accelerated failure of high-aspect-ratio walls with a centrally located opening. A further inspection of the numerical results revealed that when the opening dimension increased from 5% to 40% of the wall dimension, approximately 33.4% and 52.2% of wall strength was reduced for models with $H_w/L_w = 0.5$ and 1.0 , respectively. Likewise, for the wall models with $H_w/L_w = 2.0$ the strength reduction was 44.3% as the opening dimension increased from 5% to 25% of the wall dimension. In general, increasing the opening size dramatically decreases the ultimate load carrying capacity of axially loaded TW3S walls.

The influence of increasing the opening length while maintaining a constant height in Stage 2 was studied by plotting the axial strength ratio ($N_{uo}/f'_c L_w t_w$) against the length parameter (L_o/L_w) (see Fig. 7). The parameter L_o/L_w varied from 0.158 to 0.447 (increasing the opening length from 949 to 2683 mm) for walls with $H_w/L_w = 0.5$, from 0.224 to 0.632 (increasing the opening length from 671 mm to 1897 mm) for walls with $H_w/L_w = 1.0$, and from 0.316 to 0.707 (increasing the opening length from 474 mm to 1061 mm) for walls with $H_w/L_w = 2.0$. The height parameter H_o/H_w was equal to 0.316 , 0.224 and 0.158 for $H_w/L_w = 0.5$, 1.0 and 2.0 , respectively. The variation of the axial strength ratio against L_o/L_w appeared almost perfectly linear, validated by the R^2 values. The effect of accelerated failure by increasing L_o/L_w , indicated by the slope of the trendlines, for walls with various aspect ratios can be explained by reasons similar to increasing A_o/A_w . In addition, once L_o/L_w increased by the aforementioned values, the ultimate load capacity of the wall models with $H_w/L_w = 0.5$, 1.0 and 2.0 decreased

by 27.2%, 44.8% and 36.9%, respectively. This confirms that increasing the length of the opening considerably reduces the ultimate load carrying capacity of axially loaded TW3S walls.

The influence of increasing the opening height while the length remained constant was then examined in Stage 3. Fig. 8 plots the axial strength ratio ($N_{uo}/f'_c L_w t_w$) against the height parameter (H_o/H_w). The parameter H_o/H_w varied from 0.316 to 0.894 (increasing the opening length from 949 to 2683 mm) for walls with $H_w/L_w = 0.5$, from 0.224 to 0.632 (increasing the opening length from 671 mm to 1897 mm) for walls with $H_w/L_w = 1.0$, and from 0.158 to 0.447 (increasing the opening length from 474 mm to 1061 mm) for walls with $H_w/L_w = 2.0$. The length parameter L_o/L_w was equal to 0.158, 0.224 and 0.316 for $H_w/L_w = 0.5$, 1.0 and 2.0, respectively. A near ideal linear relationship existed between the axial strength ratio and H_o/H_w , verified by the R^2 values. As evidenced by the steepness of the lines, increasing H_o/H_w had the most significant impact on the decreased axial strength ratio of walls with $H_w/L_w = 2.0$ and the least effect on the decreasing axial strength ratio of walls with $H_w/L_w = 0.5$. This may be attributed to the following reasons. The failure of a solid wall with $H_w/L_w = 2.0$ is due to the dominance of the TW behaviour, characterised by the propagation of diagonal cracking only from the restrained corners. By introducing the central opening in the wall and gradually increasing the height of the opening, a hybrid system of beam and column elements is created, altering the failure mechanism of the wall because of the column element at the free end being subjected to OW failure. In contrast, the portion located between the opening and the free end of the wall with $H_w/L_w = 0.5$ is susceptible to OW failure prior to the presence of the opening. Consequently, varying H_o/H_w does not significantly affect the failure mechanism of the wall. The results further revealed that increasing H_o/H_w had little effect on the ultimate load capacity, particularly in relation to the influence of A_o/A_w and L_o/L_w . Only 4.9%, 13.4% and 10.4%

1 reductions in axial capacity occurred for wall models with $H_w/L_w = 0.5, 1.0$ and 2.0 ,
2 respectively, when the parameter H_o/H_w increased by the aforementioned values.

4 **4.3.2. Effect of opening location**

5 Figs. 9 to 11 present the changes of axial strength ratio ($N_{uo}/f'_c L_w t_w$) of walls with three
6 different aspect ratios ($H_w/L_w = 0.5, 1.0$ and 2.0) when varying the opening location. The
7 openings were square, and the ratios of opening size to total area of wall (A_o/A_w) were selected
8 as 5%, 10% and 15%. Two locations of the centre of the openings in the vertical direction were
9 investigated, consisting of the centre located on the central horizontal axis of the walls and the
10 centre located 500 mm above the central horizontal axis of the walls. The openings were moved
11 sideways as indicated by the distance from the centre of the openings to the restrained edge of
12 the walls (η_{ox}) in Figs. 9 to 11. The numerical outcomes demonstrated that varying the opening
13 location produced different behavioural patterns in axial strength ratio for walls with various
14 aspect ratios. This is possibly attributable to the complex combined effects of altering the TW
15 behaviour gained from the side restraint, together with the OW behaviour of the column
16 element at the free end. In the case of the centre located on the central horizontal axis of the
17 walls, moving the opening location in the horizontal direction slightly affected the load-bearing
18 capacity of walls with $H_w/L_w = 0.5$ (see Fig. 9), with differences in the strength ratio of up to
19 8.5%, 6.8% and 5.9% when $A_o/A_w = 5\%, 10\%$ and 15% , respectively. For walls with $H_w/L_w =$
20 1.0 and 2.0 , the results generally showed that, as the distance between the vertical restraint and
21 the opening location increased, the ultimate strength of the wall panel increased (see Figs. 10
22 and 11). A similar phenomenon was revealed by the results obtained from the experimental
23 test results [16, 18]. Regarding walls with $H_w/L_w = 1.0$, the differences in the strength ratio
24 were up to 9.7%, 14.5% and 15.4% when $A_o/A_w = 5\%, 10\%$ and 15% , respectively. Concerning

walls with $H_w/L_w = 2.0$, the differences in the strength ratio were up to 16.3%, 14.6% and 17.9% when $A_o/A_w = 5\%$, 10% and 15%, respectively.

Figs. 9 to 11 also depict that for walls with various aspect ratios, positioning the centre of the openings above the central horizontal axis of the walls while moving the opening sideways resulted in behavioural patterns in axial strength ratio being almost similar to those of walls with the centre of the opening located on the central horizontal axis. However, the values of axial strength ratio were slightly higher than those of walls with the centre of the opening located on the central horizontal axis.

5. PROPOSED ULTIMATE LOAD PREDICTION EQUATION FOR TW3S WALLS WITH OPENINGS

5.1. The proposed equation

To develop the ultimate load equation for TW3S walls with openings, Eq. (4) was used in conjunction with Eq. (3) and χ is as defined by Eq. (6). The proposed equation is as follows:

$$N_{uo} = (k_1 - k_2 \chi_{xy}) N_u \quad (12)$$

The set of constants k_1 and k_2 was derived considering the following points. The global load-bearing capacity of walls with openings offset from the central axes and towards the free end is higher than that of walls with openings at the wall central axes, as presented in Section 4.3.2. However, in practice, it is conservative and safe to ignore the capacity increase when the opening is moved towards the free end because uncertainties such as local buckling failure may occur in the slender column element at the free end. Further, because of the nature of the approach based on a decreasing function, it was decided to only use the numerical results of walls with openings at the wall central axes and of walls with openings offset from the central axes and towards the restraint in the regression analysis. For a wall with an opening offset from the central axes and towards the free end, its ultimate load capacity is assumed the same as the

ultimate load capacity of a wall with an opening at the wall central axes. To carry out the analysis, the weighting ratio λ , varied at an increment of 0.01, was used to determine the best fit for $N_{uo,FEM}/N_{u,FEM}$ versus χ_{xy} . $N_{uo,FEM}$ is the numerical result of wall models with openings and $N_{u,FEM}$ is the numerical result of identical solid wall counterparts. It was found that $\lambda = 0.14$ yielded the highest R^2 value of 0.85 with the corresponding $k_1 = 0.944$ and $k_2 = 0.759$. This analysis is presented in Fig. 12.

5.2. The performance and limitations of the proposed equation

Table 6 compares the failure loads predicted by Eq. (12), using the derived k_1 and k_2 , with available experimental results [3, 16, 18] (the details of which can be found in Tables 1(a) and (b)). Given that the EC2 equation can be used to predict the ultimate capacity of TW3S walls with openings, the failure loads predicted by Eq. (1) are also presented in Table 6. In general, the proposed equation was found to perform satisfactorily against the test data. The ratio of the predicted strength to actual strength varied from 0.65 to 1.07, with an overall mean of 0.83 and a standard deviation of 0.12. This indicates very good predictions, with safe estimation of experimental load. If an appropriate strength reduction factor, ϕ , in the design (e.g. $\phi = 0.65$ in AS3600-18 [6]), Eq. (12) will provide very safe and conservative design values for all the test panels. When using the EC2 equation, however, the ratios of predicted strength to actual strength ranged between 0.27 and 0.68, with an overall mean of 0.43 and a standard deviation of 0.14, indicating highly conservative prediction of failure load.

Although the comparative study of predicted to test results indicates that the proposed equation is satisfactory and reliable, the following are the limitations on the applicability of the proposed equation:

- The equation is only applicable to rectangular openings. Different shapes such as circular, diamond, triangular and even irregular shapes are required further investigations.
- The equation is proposed considering a single opening in walls, therefore for walls with multiple openings more investigations are required prior to using the design equation.

6. CONCLUSIONS

This study has numerically investigated the influences of the configuration and position of openings on the axial load capacity of TW3S walls. Results of the analysis have shown the following:

- Increasing the length and the height of the opening (A_o/A_w) in equal proportion dramatically decreased the axial strength ratio ($N_{uo}/f'_c L_w t_w$).
- Increasing only the length of the opening (L_o/L_w) also considerably decreased $N_{uo}/f'_c L_w t_w$.
- Increasing only the height of the opening (H_o/H_w) had little effect on $N_{uo}/f'_c L_w t_w$, particularly in relation to the influence of A_o/A_w and L_o/L_w .
- The responses of $N_{uo}/f'_c L_w t_w$ to variations in A_o/A_w , L_o/L_w and H_o/H_w , were approximately linear.
- Increases in the opening size, length and height had greater impacts on the decreased axial strength ratio for walls with high aspect ratios.
- Varying the opening location sideways with the centre of the openings located on the central horizontal axis of the walls produced different behavioural patterns in axial strength ratio for walls with various aspect ratios. For walls with low aspect ratios, moving the opening location in the horizontal direction slightly affected $N_{uo}/f'_c L_w t_w$. For

walls with high aspect ratios, however, the results generally showed that, as the distance between the vertical restraint and the opening location increased, $N_{uo}/f'_c L_{wtw}$ increased.

- For walls with various aspect ratios, positioning the centre of the openings above the central horizontal axis of the walls while moving the opening sideways resulted in behavioural patterns in axial strength ratio being almost similar to those of walls with the centre of the opening located on the central horizontal axis. However, the values of $N_{uo}/f'_c L_{wtw}$ were slightly higher than those of walls with the centre of the opening located on the central horizontal axis.

A new design equation for estimating the ultimate capacity of TW3S walls with openings was also proposed because available models for such walls are lacking. A comparative study of predicted to test results indicated that the proposed equation is satisfactory and reasonably conservative.

Despite the conduct of extensive numerical analysis on TW3S walls with openings and derivation of a reliable design equation, further work on such walls is still deemed necessary. More experimental and numerical studies are required to investigate the effects of various concrete strength, slenderness ratio, load eccentricity, opening shape, and multiple openings. This would help to further validate and refine the proposed equation, improving its applicability and establishing more accurate design aids for this important class of concrete structures.

DECLARATION OF COMPETING INTEREST

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

1 NOTATION

A_w, A_o	areas of the wall panel and opening(s), respectively
A_x, A_{ox}	horizontal cross-sectional area of the solid wall and opening, respectively
A_{oy}, A_y	vertical cross-sectional area of the solid wall and opening, respectively
d_c, d_t	compressive and tensile damage parameters, respectively
e	eccentricity to account for eccentricity of the load application
e_a	eccentricity to account for slenderness and ‘out of straightness’
E_s	elastic modulus of steel
f_c	cylinder compressive strength of concrete
F	yield function
G	flow potential
H_w, H_o	height of the wall and opening, respectively
H_{we}	effective height of the wall ($H_{we} = kH_w$)
k	effective height factor
k_1, k_2	intersection and slope of opening function, respectively
K_c	ratio of the second stress invariant
L_w, L_o	horizontal length of the wall and opening, respectively
N_d	design axial strength
N_u, N_{uo}	ultimate axial strength of a solid wall and a wall with opening(s), respectively
$\bar{p}$	hydrostatic pressure stress
$\bar{q}$	Misses equivalent effective stress
t_w	thickness of the wall
α	reduction factor applied to the deflection to allow for the influence of axial load
χ_{xy}	dimensionless parameter for an opening in both the horizontal and vertical planes

χ_x, χ_y	opening parameters with respect to the influence of opening size and location in the horizontal and vertical directions, respectively
$\tilde{\varepsilon}_c^{pl}$	compressive damaged plastic strain
$\tilde{\varepsilon}_t^{pl}$	tensile damaged plastic strain
Φ	specific load-bearing capacity for plain concrete walls
ϕ	capacity reduction factor or friction angle of concrete
γ_c	partial safety factor
η_x, η_y	distance of the centre of gravity of wall with opening(s) to the wall centre in the horizontal and vertical directions, respectively
η_{ox}, η_{oy}	distance of opening centre to the edge in the horizontal and vertical directions, respectively
λ	weighting ratio indicating the percentage of χ_x in relation to χ_y
μ	viscous parameter
ν_c	Poisson's ratio for concrete
ν_s	Poisson's ratio for steel
ρ_h, ρ_v	horizontal and vertical reinforcement ratios in the wall, respectively
σ_c, σ_t	normal stress of concrete in compression and tension, respectively
σ_{b0}	initial equibiaxial compressive yield stress
σ_{c0}	initial uniaxial compressive yield stress
σ_{cu}	compressive stress of concrete at peak point
σ_{t0}	initial uniaxial tensile yield stress
$\hat{\sigma}_{max}$	maximum principal effective stress
ψ	dilation angle
ξ	flow potential eccentricity

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