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OVERVIEW OF EMPIRICAL EQUATION PREDICTION FOR ULTIMATE AXIAL LOAD OF PRECAST LIGHTWEIGHT FOAMED CONCRETE SANDWICH PANEL (PLFP)

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ABSTRACT

In the absence of analytical theory, empirical equation is useful in estimating the ultimate load carrying capacity of structural component. Empirical approach means the collection of data on which to base a theory or derive a conclusion in science. It is part of the scientific method. The empirical method is often contrasts with the precision of the experimental method where data are derived from an experiment. This paper review the development of empirical equation from solid reforced panel to sandwich panel. The previous developed empirical equations are be able to predict an adequate ultimate strength of PLFP panel under axial loading due to the safety factor reduction. Series of experiment and Finite Element ANALYSIS (FEA) were carried out to produce sufficient data to analyze the previous developed empirical equation to predict the ultimate load carrying capacity. From findings, a new empirical equation is in need to predict the ultimate axial load of sandwich panel in order to get accurate prediction.

Keywords: ultimate load carrying capacity, empirical equation, precast sandwich panel.

INTRODUCTION

Precast sandwich panel presents a series of possibilities for the solution of housing problems especially in low and medium cost housing sector [1-5], Sandwich panels have all the desirable characteristics of a normal precast concrete wall panel such as durability, economical, fire resistance, large vertical spaces between supports, and can be used as shear walls, bearing walls, and retaining walls. It can be located to accommodate building expansion need. In addition, the insulation property provides superior energy performance compared to other wall systems [1].

Researchers studied the sandwich panels since few decades ago with different materials until recently to determine its structural behaviours and ultimate load carrying capacity. These researchers contributed to the development of empirical equations and suggested modification for various conditions [5-7].

However, there is very little knowledge on the PLFP panel structural properties. The information and study of lightweight sandwich elements as load bearing walls is still very limited. As such, the purpose of this paper aims to investigate the ultimate load carrying capacity prediction of PLFP panel with double shear truss connectors by previous developed equation.

PREVIOUS DEVELOPED EMPIRICAL EQUATIONS FOR SOLID WALL

According to American concrete institute (ACI) [8] and British Standard 8110 (BS 8110) codes [9], the design of concrete wall method falls into two categories. The concrete wall can be designed under empirical design formula or treating the wall as column in design procedure.

ACI 318-89, BS8110 codes and previous researchers proposed expressions for various walls structures to determine its structural behaviour in term of its ultimate strength capacity.

Empirical Equation from ACI 318-89

In ACI 318-89 [8], an empirical equation provided to calculate the design axial strength of solid walls with rectangular cross section. According to it, walls subjected to axial load or combined flexure and axial load shall be designed as compression member.

$$P_u = 0.55\Phi f_{cu} A_c \left[1 - \left(\frac{kH}{32t}\right)^2\right] \quad (1)$$

Where;

P_u = the ultimate strength of panel
Φ = the strength of reduction factor (0.7 for reinforced member)
f_{cu} = the compressive strength of concrete
A_c = the gross area of section
k = 0.8 for wall brace top and bottom against lateral translation and restrained against rotation at one or both ends.
H = the vertical distance between supports; or height of panel
t = the overall thickness of member

In addition, in ACI318-89 also stated a design strength equation for design axial load strength equation for reinforced column and wall member with hoops and ties as transverse reinforcement. The equation is as stated in equation 2.

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$$P_u = 0.85\Phi[0.85 f_{cu}(A_c - A_{sc})] \quad (2)$$

Where;

A_{sc} = total area of longitudinal reinforcement

f_y = the tensile strength of the steel

Empirical Equation from BS8110

Other than ACI318, BS8110 [9] also provided an equation to determine the design axial load capacity for short braced concrete walls. The equation is as below:

$$P_u = 0.35 f_{cu} A_c + 0.67 f_y A_{sc} \quad (3)$$

It also stated that for lightweight concrete, when the slenderness ratio of a wall is more than ten, the wall will be considered as slender.

Empirical Equation from Eurocode 2

Eurocode 2 [10] also provided an equation to determine the axial resistance of lightly reinforced concrete wall. The equation calculates the axial resistance under ultimate limit states. The equation stated as below is for rectangular cross section wall with uniaxial eccentricity, e in the direction of overall depth as shown in Figure-1.

$$N_{Rd} = \emptyset(\eta f_{cu} bh) \quad (4)$$

$$\emptyset = 1.14\left(\frac{1-2e_{tot}}{h}\right) - 0.02\left(\frac{l_o}{h}\right) \leq \left(\frac{1-2e_{tot}}{h}\right) \quad (5)$$

Where;

N_{Rd} =axial resistance of wall

$\emptyset$ =Factor taking into account curvature, including second order effects and normal effects of creep

η =effective length factor, 1.0 for compressive strength of concrete at 28 days ≤ 50MPa.

b =overall width of the cross section

h =overall depth of the cross section

e = the eccentricity of loading in the direction h_w, $e \leq \frac{h_w}{6}$

h_w =overall depth of the cross section of foamed cross section

h_p =overall depth of the cross section of polystyrene

l_o =effective length of the wall

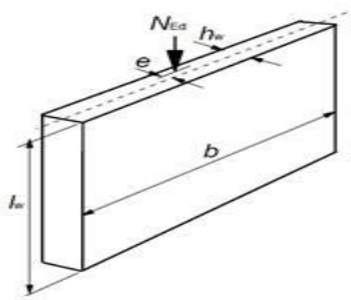

Figure-1. Notation for wall [10].

Empirical Equation from Previous Researchers

Since half decades ago, researchers started to study on precast concrete walls toward its structural properties. These researchers contributed to the development of empirical equations and suggested modification for various conditions.

Leabu [11] was the earliest researcher who studied the problem of solid precast concrete wall panels. From the finding, it was suggested an expression to determine the ultimate load carrying capacity in concrete wall panel under pure axial load. The suggested expression included the effect of slenderness ratio on the ultimate load carrying capacity.

$$P_u = 0.2\Phi f_{cu} A_c \left[1 - \left(\frac{H}{40t}\right)^2\right] \quad (6)$$

Obelender *et al.* [12] carried out an experimental programme on solid RC walls with slenderness ratios $\frac{H}{t}$ varying from 8 to 28. 54 wall panels with two layers of symmetric reinforcement and separately placed within the wall thickness were tested under uniformly distributed axial and eccentric loads. The eccentricity was applied at $\frac{1}{6}$ of the wall thickness. From the result, it was observed that under axial and eccentric loading, panels with $\frac{H}{t}$ values less than 20 failed in crushing while those with larger values of $\frac{H}{t}$ failed due to buckling. The result also showed that for load of eccentricities $\frac{t}{6}$, the reduction in wall strength was between 18 to 50 percentages as the slenderness ratio increased from 8 to 28. From this experimental programme, Oberlender (1973) proposed an expression for the ultimate axial load.

$$P_u = 0.6\Phi f_{cu} A_c \left[1 - \left(\frac{H}{30t}\right)^2\right] \quad (7)$$

Pillai and Parthasarathy [13] studied and concentrated on the behaviour of panels with a central single layer of reinforcement. 18 wall panels tested under uniformly eccentric loads ($\frac{1}{6}$ of the wall thickness) with simple support at top and bottom only. The slenderness ratios varied from 5 to 30, aspect ratios from 0.57 to 3.0 and thickness from 40 to 80mm. It was reported that the steel ratio had very little influence on the ultimate load carrying capacity of those wall panels. The panels with lower $\frac{H}{t}$ ratios generally failed by cracking and splitting near the edges while those with $\frac{H}{t} > 20$ failed because of horizontal cracks at mid height and the tension face. An expression was proposed to determine the ultimate load carrying capacity for wall panels with slenderness ratio $\frac{H}{t} > 30$.

$$P_u = 0.57\Phi f_{cu} A_c \left[1 - \left(\frac{H}{50t}\right)^2\right] \quad (8)$$

Krispanarayanan [14] suggested *K* factor for the slenderness part of the design equation to estimate the capacity for walls with pin-ended supports. The modification of the empirical equation was accepted by the ACI code and incorporated in the current ACI318 code (Equation 1).

Saheb and Dasayi [6] conducted a series of experimental work programme on 24 reinforced concrete wall panels. They investigated the influence of the slenderness ratios (from 17 to 32), aspect ratio (0.67 to 2.0) vertical and horizontal reinforcement ratios on the ultimate load carrying capacity of the wall panels. They proposed two equations for the ultimate load carrying capacity for solid reinforced walls with aspect ratio, $\frac{H}{L} < 2.0$ and $\frac{H}{L} \leq 2.0$ for wall with $\frac{H}{t} < 32$ and $e \leq \frac{t}{6}$. They found that, the ultimate load carrying capacity of the wall panels decreases linearly with an increase in aspect ratio. The reduction in ultimate load is about 16.6% for an increase in $\frac{H}{L}$ from 0.67 to 2.0.

For aspect ratio $\frac{H}{L} < 2.0$:

$$P_u = 0.55\Phi[f_{cu}A_c + (f_y - f_c)A_{sc}]\left[1 - \left(\frac{H}{32t}\right)^2\right]\left[1.20 - \left(\frac{h}{10L}\right)\right] \quad (9)$$

For aspect ratio $\frac{H}{L} \geq 2.0$:

$$P_u = 0.55\Phi[f_{cu}A_c + (f_y - f_{cu})A_{sc}]\left[1 - \left(\frac{H}{32t}\right)^2\right] \quad (10)$$

The limitations of these two equations may only be used for slenderness ratio $\frac{H}{t} \leq 32$ and eccentricity of $\frac{t}{6}$.

Jeung [15] conducted experimental and theoretical studies of normal and high strength concrete wall panels. He introduced an effective length factors into the expression for walls in both one and two way action.

$$\emptyset N_u = \emptyset 2.0 f_{cu}^{0.7}[t_w - 1.2e - 2e_a] \quad (11)$$

Where;

$\emptyset$ =0.6 for compression member

N_u =design axial strength per unit length of wall (N/mm)

e = eccentricity of the load measured at right angles to plane of the wall (mm)

e_a =an additional eccentricity due to deflections in the wall and $e_a = \frac{H^2}{2500t}$ (mm)

Previous Developed Empirical Equations for Precast Sandwich Wall Elements

Benayoune [5] suggested an expression to determine ultimate load carrying capacity for precast concrete sandwich panel which subjected to axial load. He developed the expression by using experimental and FEA results, in the equation, the effect of reinforcement used in precast concrete sandwich panel was included. This equation increases the slenderness function and incorporates the contribution of the steel reinforcement to better fit the FEA and experimental results. However, the equation developed by him was only applicable for panel with slenderness ratio lower than 25, and therefore the expression developed can't be used for taller panel with higher slenderness ratio.

$$P_u = 0.4 f_{cu} A_c \left[1 - \left(\frac{kH}{40t}\right)^2\right] + 0.67 f_y A_{sc} \quad (12)$$

The latest researcher who studied the behaviour of precast lightweight foam concrete panel with single shear connector was Mohamad [16]. Based on the study, an empirical expression for the ultimate load capacity was suggested. The proposed equation was modified from the previous research equations, ACI318 [8] and BS 8110 [9] by incorporating the contribution of the steel area and by introducing the eccentricity of $\left(t - \frac{t}{20}\right)$. The multiplying factor for steel is reduced to 0.6 because steel's contribution on the strength of panel is generally very small. The eccentricity is included in the equation due to imperfection during testing. The proposed equation is as below:

$$P_u = 0.4 f_{cu} A_c \left[1 - \left(\frac{kH}{40\left(t - \frac{t}{20}\right)}\right)^2\right] + 0.6 f_y A_{sc} \quad (13)$$

Comparison of Results from FEA and Empirical Equations

Upon the completion of FEA, validated FEA results were used to develop an empirical equation to predict the ultimate load carrying capacity of PLFP panel. Panels were simulated with and without initial curvature namely perfect and imperfect model to predict the structural behaviour of panel. From findings, panel with initial curvature was able to predict the ultimate load carrying capacity with adequate accuracy, and therefore it would be used as empirical data for the development of empirical equation. Panels with 100 mm thickness and various heights from 1,800 mm to 4,000 mm were used to predict the ultimate load carrying capacity; the comparison of results from FEA and previous developed equations were plotted in Figures-2 & Figure 3. Reduction factor (safety factor) was considered in empirical equations from previous studies, and therefore the strength prediction was only considered as design load which was much lower that ultimate load carrying capacity as predicted by FEA (Figure-2). In order to use their equations to predict the ultimate load carrying capacity, reduction factor was

excluded to calculate the ultimate strength of panel as seen in Figure-3. From the comparison of FEA results with previous developed equations, it could be seen that the empirical equation from Eurocode 2 [10], Benayoune [2] and Mohamad [16] gave the most conservative ultimate load carrying capacity values for PLFP panel with different slenderness ratio.

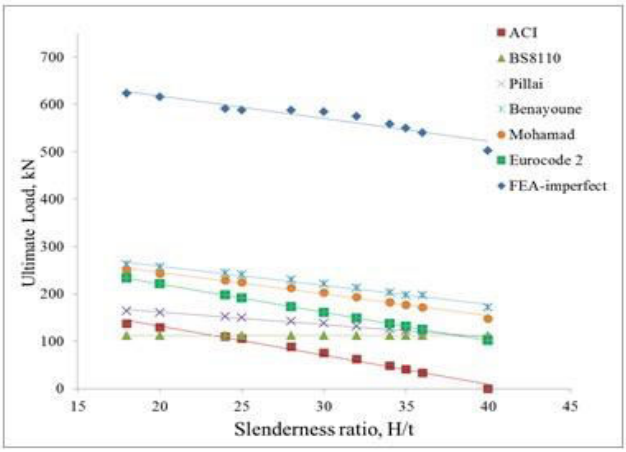

Figure-2. Comparisons of FEA and empirical values from empirical equation (safety factor included) for ultimate load of PLFP panels.

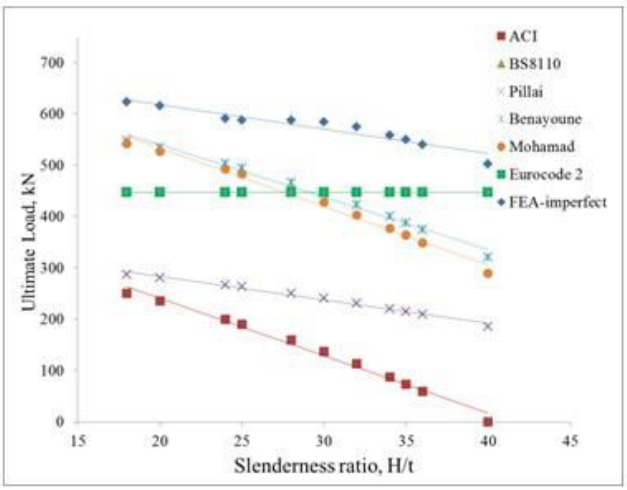

Figure-3. Comparisons of FEA and empirical values from empirical equations (safety factor excluded) for ultimate load of PLFP panels.

Three most accurate prediction equations were selected for modification and proposed as a new empirical equation. Figure-4 shows the ultimate load versus slenderness ratio curves for the three selected equations with FEA results. All the curves from equations by Benayoune [2], Mohamad [16] and FEA recorded decrease ultimate load with increase slenderness ratio; however, Eurocode 2's equations show a straight curve due to the exclusion of entire safety and reduction factor [10]. The basic formula is $f_{cd}bh$, and therefore the result is the ultimate strength under perfect condition.

The graphs (Figures-2 & 3) drawn from previous developed equations predicted the design load of the panel. In this study, the aim was to predict the ultimate load of PLFP panel, and therefore a new empirical equation was modified from previous empirical equation.

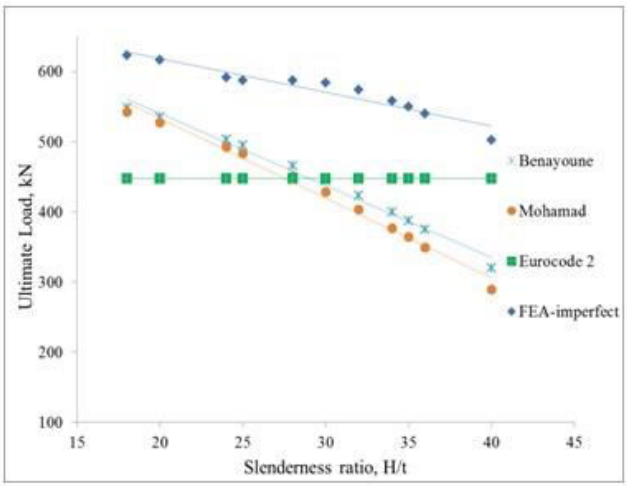

Figure-4. Comparisons of FEA and three closest empirical predictions for ultimate load carrying capacity of PLFP panels.

In order to assess the ultimate load carrying capacity predictability of the equation, the upper and lower limit of boundaries is set at ± 20% as seen in Figure-5, 6 & 7. From the finding, these equations were inadequate to predict the ultimate strength of PLFP panel and the predicted result exceeded the upper boundary of 20%.

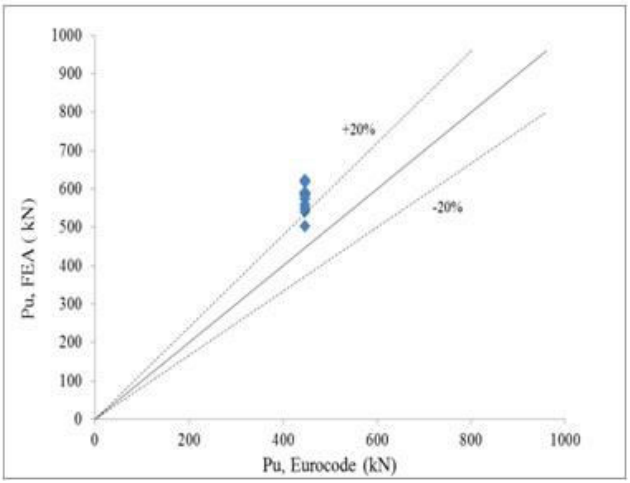

Figure-5. Percentage difference between ultimate load carrying capacity from FEA and equation 4 .

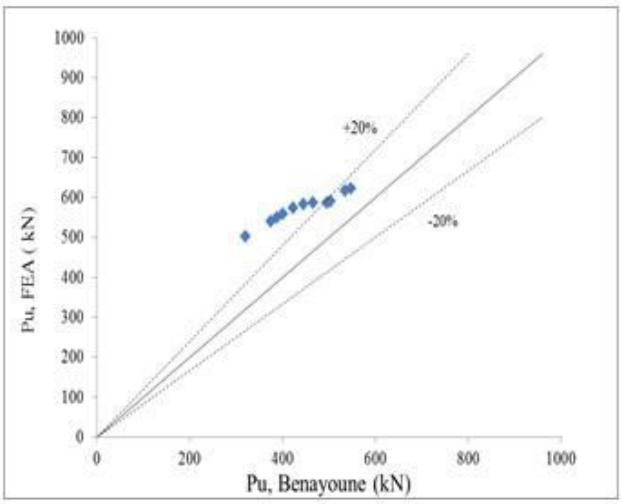

Figure-6. Percentage difference between ultimate load carrying capacity from FEA and equation 12.

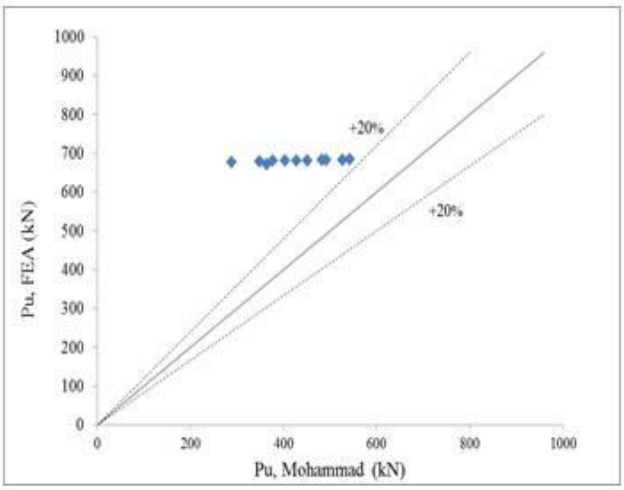

Figure-7. Percentage difference between ultimate load carrying capacity from FEA and equation 13.

CONCLUSIONS

This study discussed previously developed empirical equations and their accuracy levels to predict the ultimate load carrying capacity of PLFP panel.

It was found that equation from [10] gives the most conservative ultimate strength for PLFP panel with various slenderness ratios, $\frac{H}{t}$; however, this equation was originally designed for plain concrete wall; it ignored the effect of steel area on the ultimate strength. Therefore there is an urgent need to develop a new empirical equation to predict the ultimate load carrying capacity of PLFP panel under axial loading.

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