DESIGN OF SHORT SSTT-CONFINED CIRCULAR HSC COLUMNS

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Abstract

Steel-straps tensioning technique (SSTT) has been proven to be an effective mean to confined High-strength concrete (HSC). The pre-tensioned force offers by this confining method can significantly restraint the small lateral dilation of HSC. However, most of the design guidelines only concerned with FRP-confined columns subjected to concentric compression. The direct application of these design guidelines on the SSTT-confined HSC column is being questioned due to different material and confining method adopted. Hence, a numerical study was carried out in the view of developing a simple design equation for HSC column confined with SSTT. The parameters such as SSTT-confinement ratio, load eccentricities and slenderness ratio were tested. Based on the numerical results, design equations based on regression analysis were proposed to determine the ultimate load and bending moment of SSTT-confined HSC columns.

Index Terms:*Confinement, Steel Straps, Eccentric loads, and Theoretical Model.*

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1.INTRODUCTION

Lateral confinement of concrete columns has been proven to be very effective in increasing both ultimate strength and ductility [1]. However, conventional confinement methods are less effective in confining high-strength concrete (HSC) due to smaller lateral dilation compared to normal-strength concrete (NSC) [2, 3, 4, 5]. Under-utilization of the confining materials used in confining HSC was reported due to the sudden failure of HSC before it was fully dilated under compression. This has led to the uneconomical use of such confining method for HSC structure [6].

For HSC column confined with steel-straps tensioning technique (SSTT), the low-cost steel-straps which is normally seen in the packaging industry were used (Figure 1). The steel-straps were pre-tensioned around the column by using the pneumatic tensioner prior to the loading. Different from the conventional confining method where confining effect is initiated by the dilation of concrete itself, the pre-tensioned force provides by SSTT can ensure the structure was perfectly confined even before dilation. Figure 2 shows the pneumatic tensioner used in pre-tensioning the steel-straps.

SSTT in HSC columns has several functions. They are to confine the concrete core and to restrain the longitudinal reinforcement from buckling. All of these functions contributed to the improvement of flexural strength and ductility of the columns. It should be noted that the flexural strength and moment capacity for SSTT-confined short HSC column under eccentric loads will increase with the increasing confinement volumetric ratio. However, no design equation is available to determine the actual flexural capacity of such a column.

In this paper, the development of a simple design equation for such a column is presented. A parametric study was conducted to investigate all parameters affecting the ultimate capacity of SSTT-confined HSC Columns. The proposed design equation to calculate the flexural strength and moment capacity is expected to be a very useful design aid for structural engineers in the design of SSTT-confined HSC columns.

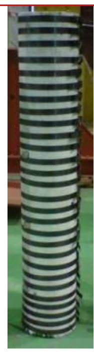

Figure 1HSC column confined with SSTT

Figure 2 Pneumatic Tensioner

2. THEORETICAL MODEL

The stress-strain model proposed by Awang[6] for SSTT-confined HSC is chosen in this study. The parameters considered in this stress-strain model are SSTT-confinement ratio, ρ_s, and unconfined concrete strength, f'_{co}, respectively. The proposed stress-strain model is as below:

$$\frac{f'_{cc}}{f_{co}} = 2.62\left(\rho_s \frac{f_y}{f_{co}}\right)^{0.4} \text{ and } \quad \rho_s = \frac{V_s}{V_c} \quad (1)$$

where

f'_{co} = unconfined concrete strength.

ρ_s = SSTT-confining volumetric ratio.

The peak strain ε'_{cc} is calculated as below:

$$\frac{\varepsilon'_{cc}}{\varepsilon'_{co}} = 11.60\left(\rho_s \frac{f_y}{f_{co}}\right) \quad (2)$$

where

ε'_{co}= concrete axial strain for unconfined concrete strength f'_{co}

The capacity of SSTT-confined HSC sections can be calculated if the stress-strain model for such a concrete is known. However, strain gradient exists for column subjected to eccentricity loading and it is general practice to assume that this effect is negligible. The axial load, P and bending moment, Mx is found based on the equations below:

$$P = \int_{\lambda_c=R-x_n}^{R} \sigma_c b d\lambda_c + \sum_{i=1}^{n}(\sigma_{si} - \sigma_c) A_{si} \quad (3)$$

$$Mx = \int_{\lambda_c=R-x_n}^{R} \sigma_c b\lambda_c d\lambda_c + \sum_{i=1}^{n}(\sigma_{si} - \sigma_c) A_{si}(R - d_{si}) \quad (4)$$

Where R is radius, $b$is the length of the segmented layer from the location of λ_c, σ_{si} is the steel stress within that particular layer, and $A_{si}$is the cross-sectional area of the steel.

The reference column used in this study is circular with diameter D = 150 mm. The concrete characteristic cube strength is 60 MPa. 4 steel bars are distributed around the column evenly. The characteristic yield strength of the steel is 460 MPa with elastic modulus,E_s= 200 GPa.

This analysis equally divided the column section into 50 layers with each thickness of 3 mm. In order to ensure accuracy, the calculation is stopped when difference between the resultant load and assumed load exceeded 10^{-6} N. This theoretical model assumed the columns should have deflected in half-sine shape. Checking on the force equilibrium is only needed at the mid-height of the column where failure normally takes place. However, the model is limited to the modeling of column subjected to equal eccentricities. The present method of analysis has been used in several past studies and the reliability of this method has been proven [7, 8, 9] Nevertheless some of the existing design codesare based on this method of analysis [10, 11].

It is assumed that the deflected shape of columns can be closely approximated using a half-sine shape. It is easily expressed mathematically as[7, 8, 9]:

$$\delta = -\delta_{mid} \sin\left(\frac{\pi}{l} x\right) \quad (5)$$

whereδ_{mid} is the lateral displacement at the critical section and x is the distance from the origin. By differentiate Equation 5 twice, the equation for curvature is obtained as below.

$$\emptyset = \delta_{mid}\frac{\pi^2}{l^2} \sin\left(\frac{\pi}{l} x\right) \quad (6)$$

When x is occurred at the mid-height of the column, hence

$$\delta_{mid} = \frac{l^2}{\pi^2}\emptyset_{mid} \quad (7)$$

The moment of acting on the critical section thus can be found. However, this moment and stress at the critical section have to be in equilibrium state. Hence, strain value for outmost compression segment has to be assumed to check the axial load and moment for each value of $\emptyset_{mid}$. The correct $\delta_{mid}$is found when the axial force divided by moment is equal to the $\delta_{mid}$for a particular $\emptyset_{mid}$. The complete load-deflection curve can then be drawn for incremental value of $\emptyset_{mid}$.

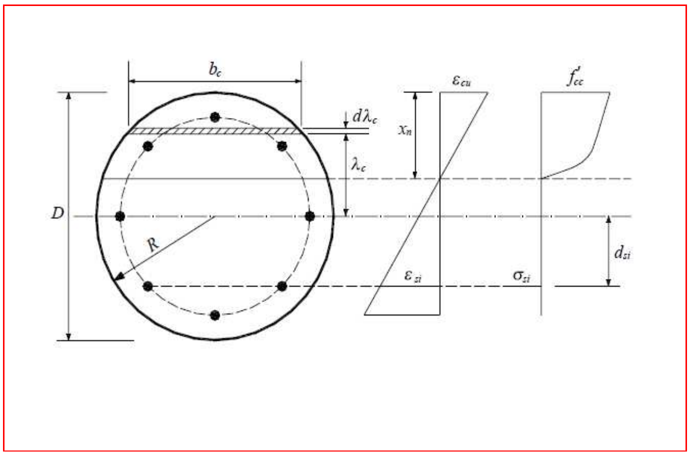

Figure 3 Typical train and stresses diagram over the circular confined column

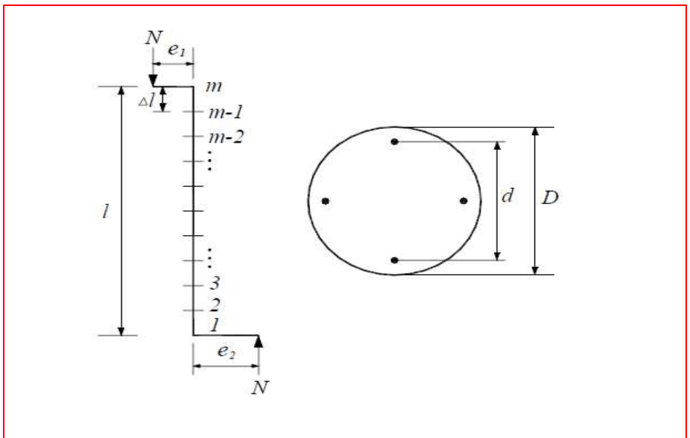

Figure 4 Schematic of theoretical model

3.MOMENT-CURVATURE AND LOAD-MOMENT INTERACTION CURVES FOR SSTT-CONFINED HSC COLUMNS

A series of theoretical moment-curvature curves is generated based on the stress-strain model discussed on the previous section. Assumptions were made in generating the moment-curvature curves:

(i) Linear strain is assumed across the column section.

(ii) Tensile strength of concrete is assumed negligible.

(iii) The ultimate unconfined concrete strain is 0.004

(iv) The initial tangent modulus of concrete, E_c, is equivalent to $4230\sqrt{f_{cu}}$.

Figure 5 shows a series of moment-curvature curves for different axial load. It should be noted that the axial load level is defined by $P/Af'_{cc}$where P is the axial load, A is the cross-sectional area of the column and f'_{cc} is the confined concrete strength. In Figure 5, it is clearly seen that the ductility of moment-curvature curves reduced gradually as the axial load increases. In low axial load level, the curves resembled to the elasto-plastic shape. For higher axial load level, the maximum moment degrades rapidly.

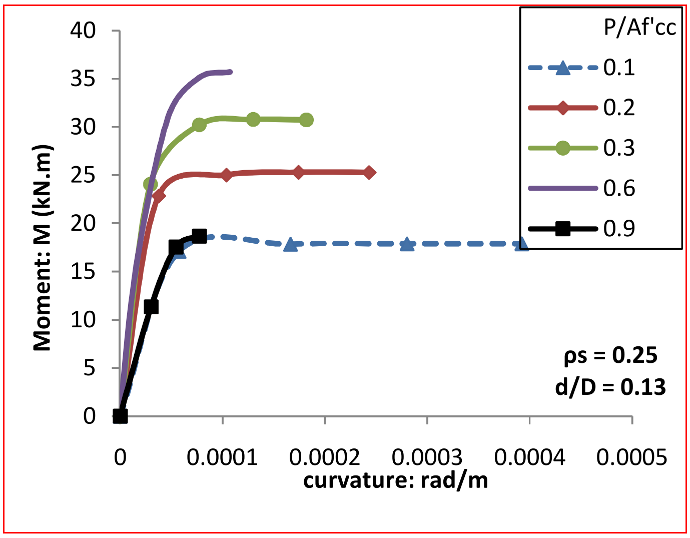

Figure 5 Moment-curvature curves for SSTT-confinement ratio, ρ_s= 0.25 based on Awang's model

Figure 6 shows the corresponding load-moment interaction curve. It is obvious from Figure 6 that moment increases from zero to approximately 3.5 x 10^7 N. mm. After this level, the moment decreases as the axial load increases. The maximum moment occurred when the axial load is approximately 1 x 10^6 N, which represented the balanced axial load level.

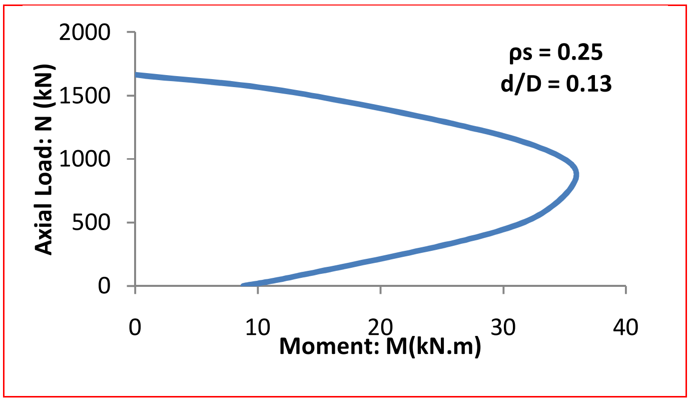

Figure 6 Load – moment interaction curve

4. PROPOSED DESIGN EQUATIONS

4.1 Equivalent Stress Block

In the design of reinforced concrete (RC) members, the stress profile of concrete in compression is generally simplified using an equivalent stress block, over which the stresses are uniformly distributed. This equivalent stress block can be described by two factors, the magnitude of stresses and the depth of the stress block. One criterion in defining these two factors is that the resulting equivalent stress block based on these two factors must resist the same axial force and bending moment as the original stress profile. Due to the existence of SSTT confinement, the use of the conventional equivalent stress blocks as proposed by the current design guidelines are no longer suitable. Hence, it is necessary to develop an appropriate stress block factors for SSTT-confined HSC. Similar to the conventional equivalent stress block as proposed by the current design guidelines, this paper adopted the mean stress factor, α_1 as the ratio of the uniform stress over the stress block to the compressive strength of SSTT-confined HSC and the block depth factor, β_1 as the ratio of the depth of the stress block to that of the neutral axis.

The stress distributions over the compression zone are examined for different neutral axis positions in order to find the appropriate equivalent stress block factors. The maximum SSTT-confinement ratio ρ_s= 0.5 is adopted based on the recommendation in the previous section. The stress block parameters are determined simultaneously from the axial load and moment equilibrium conditions to match with the equations proposed by Warner et al [12] as below:

$$P = \alpha_1 \beta_1 f'_{cc}A + \boldsymbol{\sigma}_{sc}A_{sc} - \boldsymbol{\sigma}_{st}A_{st} \quad (8)$$

$$M = \alpha_1 \beta_1 f'_{cc}A\left(\frac{D}{2} - \frac{\beta X_n}{2}\right) + (\boldsymbol{\sigma}_{sc}A_{sc} - \boldsymbol{\sigma}_{st}A_{st})\left(\frac{D}{2} - d'\right) \quad (9)$$

where D is the total height of the column, x_n is the depth of neutral axis, d' is the effective depth calculated as the distance between the outmost compression fiber and the center of tensile reinforcement. $\boldsymbol{\sigma}_{sc}$ and $\boldsymbol{\sigma}_{st}$ represent the stress for compressive reinforcement and tensile reinforcement, respectively.

For circular columns, the shape of the compression zone is a segment of a circle as demonstrated in Figure 7. In Figure 7, x_n is the depth of compression zone is calculated by $\beta_1 x_n$. D is the diameter of column and y is the distance between the centroid of compression zone and the centroid of the column.

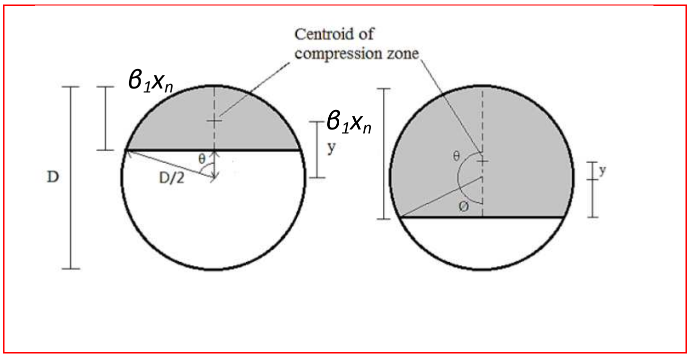

Figure 7 Compression zones of circular columns under eccentric loadings as Warner et al [12]

The area of the compression zone A can be calculated as follow:

$$A = D^2\left(\frac{\theta_{rad} - \sin\theta\cos\theta}{4}\right) \quad (10)$$

The moment of this area can be expressed as follow:

$$Ay = A\left(\frac{D}{2} - \frac{\beta X_n}{2}\right) \quad (11)$$

Where θ_{rad} is the angle expressed in radius. While θ can be calculated using the following formula:

$$\theta = \arccos\left(\frac{R - \beta_1 x_n}{R}\right) \; ; \text{for } \beta_1 x_n \leq D/2 \quad (12)$$

$$\theta = \Pi - \arccos\left(\frac{\beta_1 x_n - R}{R}\right) \; ; \text{for } \beta_1 x_n \geq D/2 \quad (13)$$

Therefore, the concrete compressive load, C_c for circular column can be expressed as:

$$C_c = \alpha_1 \beta_1 f'_{cc} A \quad (14)$$

And the moment contributed by the concrete is given as:

$$M_c = \alpha_1 \beta_1 f'_{cc} Ay \quad (15)$$

For compressive force of compression reinforcement, the strain of the steel can be calculated using the triangles method as:

$$\varepsilon_c = \varepsilon_u\left(1 - \frac{d_0}{x_n}\right) \quad (16)$$

Hence, the stress in the longitudinal steel can be calculated as:

$$\boldsymbol{\sigma}_{sc} = E_s\varepsilon_{sc} ; \varepsilon_{sc} \leq \varepsilon_{sy} \quad (17)$$

$$\boldsymbol{\sigma}_{sc} = f_{sy} ; \varepsilon_{sc} \geq \varepsilon_{sy} \quad (18)$$

whereε_{sy}, $f_{sy}$and $E_s$are the yield strain, yield stress and the elastic modulus of steel. Therefore, the compression force of the compressive steel can be calculated as:

$$C_s = \boldsymbol{\sigma}_{sc}A_{sc} \qquad (19)$$

In which $A_{sc}$is the area of compressive steel. Similarly, the strain and stress in the tension steel can be calculated as:

$$\varepsilon_{st} = \varepsilon_u\,(\frac{d'}{x_n} - 1) \qquad (20)$$

Hence, the stress in the tension longitudinal steel can be calculated as:

$$\boldsymbol{\sigma}_{st}= E_s\varepsilon_{st};\varepsilon_{st}\leq \varepsilon_{sy}(21)$$
$$\boldsymbol{\sigma}_{st}= f_{sy};\varepsilon_{st}\geq \varepsilon_{sy}(22)$$

Therefore, the tensile force in the steel is given as:

$$T = \varepsilon_{st}A_{st} \qquad (23)$$

Figure 8 shows the variations of the stress block factors against the SSTT-confinement ratio. It can be seen that the mean stress factor decreases as the strength enhancement ratio increases and as the block depth factor varies only slightly against the strength enhancement ratio. For simplicity, it is suggested that $\beta_1 = 0.9$

$$\beta_1 = 0.9 \qquad (24)$$

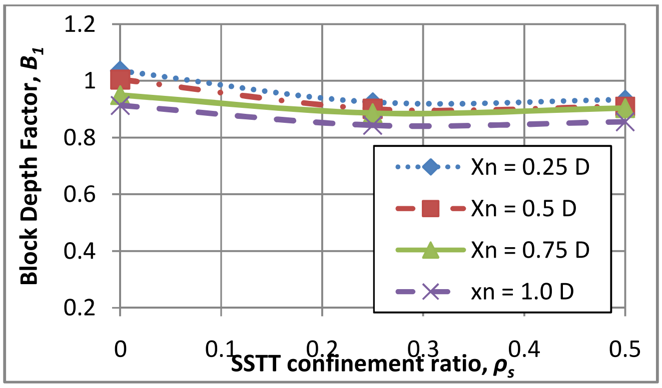

Figure 8 Block depth factor, β_1 for SSTT-confined HSC sections

Once the β_1 is fixed, α_1 can be calculated according to the criterion of equivalent stress block as discussed above. Figure 9 shows the calculated value of mean stress factor, when β_1 = 0.9. Based on the Figure 9, the simple linear equation can be suggested as follow:

$$\alpha_1 = 0.195\rho_s + 0.85 \qquad (25)$$

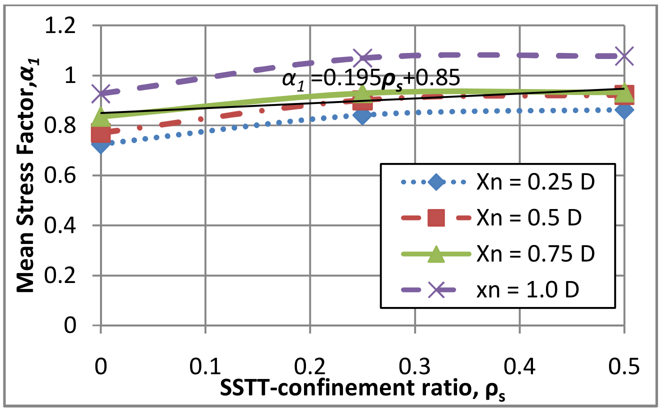

Figure 9 Mean stress factor, α1for SSTT-confined HSC sections

The performance of Equation 24 and 25 in evaluating the capacity of SSTT-confined HSC column will be evaluated in later section.

4.2 Results for SSTT-Confined HSC Columns

Using the stress block factors proposed above, design equations based on simplified section analysis method are presented herein.

$$N_u=0.9(0.195\rho_s + 0.85)f'_{cc}A + \boldsymbol{\sigma}_{sc}A_{sc}-\boldsymbol{\sigma}_{st}A_{st} \qquad (26)$$

$$M_u = 0.9(0.195\rho_s + 0.85)f'ccA\left(\frac{D}{2} - \frac{\beta X_n}{2}\right)+(\boldsymbol{\sigma}_{sc}A_{sc}\text{-}\ \boldsymbol{\sigma}_{st}A_{st})\left(\frac{D}{2} - d'\right)(27)$$

Figure 10 compares the load-moment interaction curves predicted using the proposed design method with those produced using the theoretical model. It can be seen that for short SSTT-confined column, the proposed equation gives an excellent agreement.

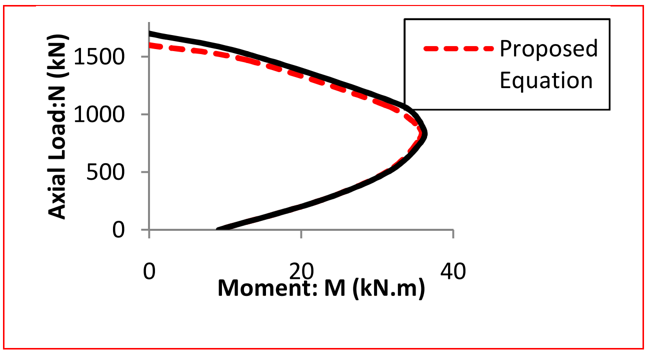

Figure10 Load-Moment interaction curves for SSTT-confined HSC column

CONCLUSIONS

This paper deals with the development of design equations for short SSTT-confined HSC columns. The proposed design equation is dedicated to SSTT-confined circular column. Based on the results and discussion, it can be concluded that the proposed design equations are simple but accurate in predicting the ultimate load and moment of short SSTT-confined HSC.

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