DESIGN OF SHORT SSTT-CONFINED CIRCULAR HSC COLUMNS

Abdullah Zawawi Awang¹, Wahid Omar², Ma Chau Khun³, Maybelle Liang⁴

¹Senior Lecturer, Faculty of Civil Engineering, Universiti Teknologi Malaysia, Malaysia, abdullahzawawi@utm.my
²Professor, Faculty of Civil Engineering, Universiti Teknologi Malaysia, Malaysia, drwahid@utm.my
³Ph.D Candidate, Faculty of Civil Engineering, Universiti Teknologi Malaysia, Malaysia, machaukhun@gmail.com
⁴Ph.D Candidate, Faculty of Civil Engineering, Universiti Teknologi Malaysia, Malaysia, maybelle_liang@hotmail.com

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 restrain 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.

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 1 HSC column confined with SSTT
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, \( \rho_s \), and unconfined concrete strength, \( f'_{\text{co}} \), respectively. The proposed stress-strain model is as below:

\[
\frac{f'_{\epsilon}}{f'_{\text{co}}} = 2.62 \left( \rho_s \frac{f'_{\epsilon}}{f'_{\text{co}}} \right)^{0.4} \quad \text{and} \quad \rho_s = \frac{V_s}{V_c} (1)
\]

where

\( f'_{\epsilon} \) = unconfined concrete strength.

\( \rho_s \) = SSTT-confining volumetric ratio.

The peak strain \( \varepsilon'_{\epsilon} \) is calculated as below:

\[
\frac{\varepsilon'_{\epsilon}}{\varepsilon'_{\text{co}}} = 11.60 \left( \rho_s \frac{f'_{\epsilon}}{f'_{\text{co}}} \right) (2)
\]

where

\( \varepsilon'_{\epsilon} \) = concrete axial strain for unconfined concrete strength \( f'_{\epsilon} \).

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 columns subjected to eccentricity loading and it is general practice to assume that this effect is negligible. The axial load, \( P \) and bending moment, \( M_x \) is found based on the equations below:

\[
P = \int_{\lambda_c=R-x_n}^{R} \sigma_c b d\lambda_c + \sum_{i=1}^{n} (\sigma_{\text{st}} - \sigma_c) A_{\text{st}} (3)
\]

\[
M_x = \int_{\lambda_c=R-x_n}^{R} \sigma_c b \lambda_c d\lambda_c + \sum_{i=1}^{n} (\sigma_{\text{st}} - \sigma_c) A_{\text{st}} (R - d_{\text{st}}) (4)
\]

Where \( R \) is radius, \( b \) is the length of the segmented layer from the location of \( \lambda_c, \sigma_{\text{st}} \) is the steel stress within that particular layer, and \( A_{\text{st}} \) 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 codes are 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]:

\[
\delta = -\delta_{\text{mid}} \sin \left( \frac{\pi}{L} x \right) (5)
\]

where \( \delta_{\text{mid}} \) 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.

\[
\phi = \delta_{\text{mid}} \frac{\pi^2}{L^2} \sin \left( \frac{\pi}{L} x \right) (6)
\]

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

\[
\delta_{\text{mid}} = \frac{i^2}{\pi^2} \phi_{\text{mid}} (7)
\]

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 \( \phi_{\text{mid}} \). The correct \( \delta_{\text{mid}} \) is found when the axial force divided by moment is equal to the \( \delta_{\text{mid}} \) for a particular \( \phi_{\text{mid}} \). The complete load-deflection curve can then be drawn for incremental value of \( \phi_{\text{mid}} \).
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 $4230\sqrt{f_{ct}}$.

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 6 shows the corresponding load-moment interaction curve. It is obvious from Figure 6 that moment increases from zero to approximately $3.5 \times 10^7$ N.mm. After this level, the moment decreases as the axial load increases. The maximum moment occurred when the axial load is approximately $1 \times 10^6$ N, which represented the balanced axial load level.
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, \(\alpha\), as the ratio of the uniform stress over the stress block to the compressive strength of SSTT-confined HSC and the block depth factor, \(\beta\), 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 \(\rho_s = 0.5\) 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 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:

\[
P = \alpha_s f'_{cc} A + \sigma_c A_{sc} - \sigma_s A_{st} \quad \text{(8)}
\]

\[
M = \alpha_s f'_{cc} A \left( \frac{D}{2} - \frac{\beta x_n}{2} \right) + (\sigma_c A_{sc} - \sigma_s A_{st}) \left( \frac{D}{2} - d' \right) \quad \text{(9)}
\]

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, \(\sigma_c\) and \(\sigma_s\) 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, \(x_c\) is the depth of compression zone is calculated by \(\beta x_n\), \(D\) is the diameter of column and \(y\) is the distance between the centroid of compression zone and the centroid of the column.

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

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

The moment of this area can be expressed as follow:

\[
Ay = A \left( \frac{D}{2} - \frac{\beta x_n}{2} \right) \quad \text{(11)}
\]

Where \(\theta_{rad}\) is the angle expressed in radius. While \(\theta\) can be calculated using the following formula:

\[
\theta = \arccos \left( \frac{R - \beta x_n}{R} \right) \quad \text{for} \quad \beta x_n \leq D/2 \quad \text{(12)}
\]

\[
\theta = \Pi - \arccos \left( \frac{\beta x_n - R}{R} \right) \quad \text{for} \quad \beta x_n \geq D/2 \quad \text{(13)}
\]

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

\[
C_c = \alpha_s f'_{cc} A \quad \text{(14)}
\]

And the moment contributed by the concrete is given as:

\[
M_c = \alpha_s \beta f'_{cc} Ay \quad \text{(15)}
\]

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

\[
\varepsilon_c = \varepsilon_c (1 - \frac{d_n}{x_n}) \quad \text{(16)}
\]

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

\[
\sigma_s = E_s \varepsilon_c ; \varepsilon_s \leq \varepsilon_y \quad \text{(17)}
\]

\[
\sigma_s = f_s ; \varepsilon_s \geq \varepsilon_y \quad \text{(18)}
\]
where \( \varepsilon_{sy} \), \( f_{sy} \), 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:

\[
C_s = \sigma_{sc} A_{sc}
\]  

(19)

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

\[
\varepsilon_{st} = \varepsilon_u (\frac{d'}{X_n} - 1)
\]

(20)

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

\[
\sigma_{st} = \begin{cases} 
E_s \varepsilon_{st} & \varepsilon_{st} \leq \varepsilon_{sy} \\
 f_{sy} & \varepsilon_{st} \geq \varepsilon_{sy}
\end{cases}
\]

(21)

(22)

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

\[
T = \varepsilon_{st} A_{st}
\]

(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 \( \beta_1 = 0.9 \)

\[
\beta_1 = 0.9
\]

(24)

Once the \( \beta_1 \) is fixed, \( \alpha_1 \) 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 \( \beta_1 = 0.9 \). Based on the Figure 9, the simple linear equation can be suggested as follow:

\[
\alpha_1 = 0.195 \rho_s + 0.85
\]

(25)

The performance of Equation 24 and 25 in evaluating the capacity of SSTT-confined HSC sections 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.

\[
N_c = 0.9(0.195 \rho_s + 0.85)f'_c A + \sigma_{sc} A_{sc} - \sigma_{st} A_{st}
\]

(26)

\[
M_c = 0.9(0.195 \rho_s + 0.85)f'_c c A \left( \frac{D}{2} - \frac{BX_n}{2} \right) + (\sigma_{sc} A_{sc} - \sigma_{st} A_{st}) \left( \frac{D}{2} - d' \right)
\]

(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.
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.

REFERENCES

BIOGRAPHIES
Dr. Abdullah Zawawi Awang
Senior Lecturer in UniversitiTeknologi Malaysia (UTM), Johor, Malaysia. PhD in Structural Engineering in UTM (2013), Masters in Structural Engineering in University of Sheffield and Bachelor in Civil Engineering, Glasgow.

Prof. Ir. Dr. Wahid Omar
Vice Chancellor of UTM, Johor, Malaysia. PhD in Structural Engineering in University of Birmingham, UK (1998), Master in Bridge Engineering in University of Surrey, UK (1989) and Bachelor in Civil Engineering in University of Strathclyde, UK (1986).

Ma Chau Khun
PhD Candidate in Civil Engineering (Structural Engineering), UTM, Johor, Malaysia. Research area in Simulation of External Confinement in Slender Column.

Maybelle Liang
PhD Candidate in Civil Engineering, UTM, Johor, Malaysia. Bachelor in Civil Engineering, UTM Skudai (2011).