Numerical investigation of HSC column under axial and flexural loading using 3D-NLFEA

P Pradnyanita1, B Piscesa1*, M M Attard2, Faimun1, and A K Samani3

1 Civil Engineering Department, Institut Teknologi Sepuluh Nopember, 60111 Surabaya, Indonesia
2 School of Civil and Environmental Engineering, The University of New South Wales (UNSW), Sydney NSW 2052, Australia
3 B&G Consultant, Doha, Qatar

* Corresponding author: piscesa@ce.its.ac.id

Abstract. High strength concrete (HSC) has been widely used as material for Reinforced Concrete (RC) column in high rise building in the past two decades. However, ductility of HSC column was one of the main concerns in design. The ductility of HSC column is much lower compared to normal-strength concrete. To increase the ductility of HSC column, the use of adequate transverse reinforcement can be used. In this paper, numerical investigation of HSC column available in the literature is modelled using an inhouse finite element package called 3D-NLFEA. The modelled specimen consisted of HSC columns with dimension 305 m x 305 m x 1473 mm and is subjected to constant axial load and incremental displacement control which act as a shear load up to failure. From the comparisons with the available test results, it was found out that the prediction using 3D-NLFEA agrees well with the test results. This paper also discusses the yield point location of bars in the load-deflection curve, length of bar that yields, buckling of compression bar, and concrete which cracked or crushed.

1. Introduction

The use of high strength material, high-strength concrete (HSC) and bars, is widely applied for high rise building in the past two decades. The use of high strength material can reduce the structural element cost as a results of smaller member section for the same strength. By having smaller member section, the effective area of the floor increases [1]. However, HSC was found to be very brittle. For plain HSC column subjected to axial loads, sudden explosive failure most likely to occur [2] and snap-back behavior may present during the softening phase [3, 4]. Hence, in design, HSC column needs to be well confined to meet the minimum ductility demand for a Special Resisting Moment Frames (SRMF) column.

The new building codes such as ACI 318-19 [5] or SNI 2847-2019 [6] requires additional confinement equation applied for concrete with strength more than equal to 70 MPa or columns with axial load ratio (P/фA') higher than 0.3. As the axial load increases, the deformability of HSC RC column reduced (lower ductility) [7]. To meet with the minimum ductility requirements, confinement to the concrete core is required. One of the conservative way to provide confinement to the concrete core is to provide confinement rebars [8-12]. This confinement rebars around the concrete core produces confining pressure which is proportional to the core expansion [13, 14]. Experimental investigation which studies the effect of confinement bars for HSC column have been studied by many researchers.
[7, 8, 10, 15-26]. Although the responses from specific column were well studied, may other configuration still require extensive experimental validation which is extremely expensive. Furthermore, the specimen size is more likely to be limited due to the equipment availability. On the other hand, the use of numerical simulation provides more benefits and is extremely low cost.

In numerical simulation, the framework used to model the concrete constitutive model plays an important role to ensure the predicted response is sufficiently accurate. For that purpose, in this paper, the discussion is focused on the numerical simulation to investigate the load deflection curve of RC column made of HSC. There are two HSC RC columns specimen experimentally tested by Bayrak and Sheikh [8]. The RC columns are loaded under constant axial load and are loaded in the lateral direction as a shear load which will generate single curvature bending moment in column. The concrete constitutive model used is Pisciota et al. [11-14] plasticity-fracture model which was found to be in good agreement in predicting the cover spalling behaviour and load-deflection curve of RC column with HSC under concentric loading [14].

The numerical investigation is carried using an inhouse three-dimensional nonlinear finite element package called 3D-NLFEA [13]. 3D-NLFEA uses SALOME 9.30 platform [27] as the preprocessor and ParaView 5.8 [28, 29] as the postprocessor. There are two column specimens tested by [8] modelled and simulated using 3D-NLFEA. Details investigation on the behavior of RC column includes the yield point of bars, length of bar that yields, buckling of compression bar, and concrete region which cracked or crushing. It is expected that after the numerical investigation corresponds well with the available test result in the literature, extensive parametric study can be carried out to further verify the requirements of additional confining rebars clause stated in ACI 318-19 and SNI 2847-2019 which should be computed for HSC and axial load ratio greater than 0.3.

2. Reinforced concrete column model

There are two HSC RC column tested by Bayrak and Sheikh [8] being investigated. Specimens AS-5HT and AS-7HT. The axial load level ratio ($P_{c}/f_{c}$) for the two columns are 0.45 which is higher than the codes limitation to enable the additional clause for confinement reinforcement. It should be noted that the confinement rebar for AS-5HT and AS-7HT are 1.08 and 0.8 times the ACI requirements (without additional clause for confinement), respectively.

2.1. Geometry details

Figure 1 shows the typical specimen geometry and rebars arrangement for columns AS-5HT and AS-7HT. Specimen AS-5HT and AS-7HT have the same geometry, longitudinal bar ratio, and concrete strength. The difference between those two are located on the confining bars arrangement. The cross-sectional shape is the same, but the use of bar type and diameter is different. The column cross-section is a square with a width of 305 mm and a height of 1473 mm. The cross-section of the stub is 508 mm x 762 mm and the height are 813 mm. The concrete cover for the column and the stub is set to 20 mm. The longitudinal bar ratio is 2.58% which consisted of eight 20M bars equally on each side. The concrete compressive strength for both specimens varies slightly. The concrete compressive strength for AS-5HT and AS-7HT are 102 and 101.8 MPa, respectively.

The reinforcing cage for the stub consisted of 10 M bars with 64 mm pitch spacing. The longitudinal bars are extended up to the interface between concrete and the steel loading plate to ensure the load are transferred effectively to the concrete. The confining bar arrangement for the column consisted of one square closed tie and one diamond shaped closed tie. For specimen AS-5HT, the confining bar size for the square closed tie is 10M and for the diamond shaped closed tie is 15M. The pitch spacing is 90 mm which gives the volumetric ratios of 4.02% or equal to 1.08 times the ACI requirements for confinement. On the other hand, specimen AS-7HT have the same size for the confining bars which is 10M with the pitch spacing of 94 mm. This configuration gives the volumetric ratios of 2.72% or equal to 0.8 times the ACI requirements for confinement. To ensure the localized region occurred at the bottom of the column (when the load-deflection curve softens), a tighter pitch spacing near the loading plates are provided in the models.
2.2. Boundary conditions and loading steps
Figure 2 shows the boundary conditions, constant axial load position, and displacement control position. There are two loading steps considered. The first loading step is force controlled. The axial load given is uniformly distributed on the steel loading plate (Point 1). The applied axial load ratio is 45% and is applied within 20 load steps. Hence, for each loading steps, the applied magnitude of the axial load ratio is 2.25%. The second load step is a full displacement control and is applied at the side steel loading plate (Point 2). The load is increased monotonically while axial load is held constant. The boundary condition at the stub (Point 3) is fixed in all three principal direction while in Point 1, is fixed in x and y directions.

2.3. Modelling procedure
The geometry model and the meshing procedure is prepared using SALOME 9.3.0 [30]. The mesh size is set to 20 mm. Figure 3 shows the 3D solid model of the column using hexahedral solid element. In figure 3a, the steel plate was shown as red coloured while the concrete was shown as grey coloured. In 3D-NLFEA, the hexahedral solid element is integrated using selective integration or BBar element
method [31]. Figure 3b shows the steel rebar configurations. Inside 3D-NLFEA, the rebars element is modelled using embedded truss element inside the concrete element [32]. Perfect bond assumption is used for the interface between the rebar and concrete elements. The total solid and embedded truss elements are 54,596 and 25,958, respectively. Once the modelling in SALOME is finished, the data are extracted to the input file in 3D-NLFEA. Once the simulation completed, the output can be viewed using ParaView 5.8.0 [28, 29].

![Figure 3a](image1.png) ![Figure 3b](image2.png)

**Figure 3.** Modelling of specimen: (a) Solid element; (b) Rebar element

3. Constitutive model of material

The constitutive model for the steel loading plates is based on $J_2$ metal plasticity model with elastic behaviour. The Young’s modulus and the Poisson’s ratio for the steel loading plate is 200 GPa and 0.3, respectively. Steel rebars are modelled using embedded truss element with perfect bond assumption. The material properties of reinforcement are detailed in Table 1. The stress-strain relationship of longitudinal bars is assigned to elastic-perfectly plastic model. For the confining rebar, stress-strain behaviour are obtained directly from the test result in [8] as shown Figure 4.

| Type of Reinforcement | Longitudinal | Transverse |
|-----------------------|--------------|------------|
| Bar Size              | 20M          | 10M        | 15M        |
| Nominal Diameter (mm) | 19.5         | 11.3       | 16.0       |
| $f_y$ (MPa)           | 454          | 542        | 463        |
| $\varepsilon_y$       | 0.0023       | 0.0026     | 0.0024     |
| $\varepsilon_{sh}$    | 0.0067       | 0.0191     | 0.0207     |
| $f_u$ (MPa)           | 700          | 683        | 648        |
| $\varepsilon_u$       | 0.1290       | 0.1550     | 0.1130     |

The constitutive model for concrete material under compression is based on plasticity model proposed by [12, 33]. The failure surface for the concrete material is based on the Menetrey and Willam [34] model but it is modified in [33] to improves both the prediction of peak and residual stress for concrete under confinement. One of the important aspects in the constitutive model to simulate the behaviour of HSC RC concrete is the ability of the constitutive model to capture the cover spalling behaviour in the load-deflection curve. In [12], a Rankine failure surface was introduced to include the fracture formulation into the model. This way, the model can distinguish between the tensile splitting cracks due to compression with a pure tensile crack due to tensile stress in concrete. In addition, the
plastic dilation rate formulation was also extended to the tensile region to accelerate the dilation rate for concrete cover elements. This cover spalling behaviour is a compulsory concern in HSC column which cause premature drop of compressive load capacity of column [10].

![Figure 4. Stress strain relationship of transverse reinforcement [13]](image)

4. Shear and tip deflection

Figure 5 shows the idealization of specimen under axial load and lateral displacement control. From 3D-NLFEA, the displacement in the z-direction at the end of stub ($z_1$), the displacement in the y direction at the column-stub interface ($\delta_1$), and the reaction force ($Q_L$) applied at point 2 (see figure 3) can be obtained. The deformed angle, $\theta$ can be computed as follow:

$$\theta = \arcsin \left( \frac{z_1}{0.5d} \right) \quad (1)$$

where $d$ is the stub width.

![Figure 5. Idealization of deformed specimen](image)
The deflection in y-direction at the top of the column ($\delta_2$) which can be calculated by multiplying the length of column, $L$ and the sinus of deformed angle, $\theta$; i.e. $\delta_2 = L \sin \theta$. Figure 5 shows that the tip deflection, $\Delta_t$ is the difference between $\delta_2$ and $\delta_1$; i.e. $\Delta_t = \delta_2 - \delta_1$.

By using the reaction force of lateral displacement control, shear in column for each loading step can be formulated as:

$$V = Q \left( \frac{a}{a + b} \right)$$

In the above, $a$ is the distance between the location of hinge and the applied lateral displacement and $b$ is the distance between the location of the applied lateral displacement to the tip of the column. In addition, the bending moment can be computed:

$$M = P \delta + V (c + L)$$

where $c$ is the distance between the end of stub to the hinge location.

5. Analysis results and discussions

This section presents the analysis results from 3D-NLFEA and are compared with the test result. In FE model using 3D-NLFEA, the lateral load applies to specimen monotonically using displacement control. To attain a proper comparison result, the backbone curve from the cyclic response [8] was used. Both backbone curve for positive and negative cycles were added into the comparison.

![Figure 6. Shear in column versus tip deflection for AS-5HT](image-url)

Figure 6 and Figure 7 show the shear force as functions of the tip deflection between the FE model and the test result [8] for specimen AS-5HT and AS-7HT, respectively. From figure 6, the 3D-NLFEA peak shear load prediction are within the backbone curves (prediction: 173.14 kN, max. value: 187.88 kN, min. value: 171.87 kN), the softening response can also be captured despite with a more conservative prediction than the test result. From figure 7, the predicted peak shear load prediction is slightly above the maximum back-bone curve of the test result (prediction: 170.58 kN, max. value: 166.97 kN, min. value: 150.5 kN). The softening response was also well capture with more conservative result than the experimental counterpart.
Figure 7. Shear in column versus tip deflection for AS-7HT

Figure 8 and figure 9 shows the deformed shape of the RC column along with the hardening and fracturing parameter. There are two points observation. The first point shows the condition before the peak shear load was reached while the second point shows the condition beyond the peak shear load (around 80% of the peak shear load).

Figure 8 and Figure 9a shows the hardening parameter contour before the peak shear load. At this stage (point 1 in the load deflection curve), the damage in the concrete still localized at the extreme concrete fibre under compression. However, at the softening stage (point 2), the concrete that softens (showed as a hardening parameter value over than unity) are spread not only in the extreme concrete fibre under compression but also to the sides of the concrete cover. Furthermore, bulging appearances on the compression zone (see figure 8b and figure 9b) are clearly shown as a result of high dilation rate for concrete in the compression-tension region.
Figure 8c and figure 9c shows the fracturing parameter before the peak shear load. As shown in figure 8c and figure 9c, the fracturing region occurred in the tension region. It should be noted that the fracturing parameter is associated with the pure uniaxial tensile stress. At the softening phase, the fracturing region spreads in the tension region (figure 8d and figure 9d). The localised cracks can be shown by looking at the strain magnitude, but it is not shown in the discussion here.

Figure 9. The Deformed Shapes of Specimen AS-7HT with: (a) Hardening contour at point 1; (b) Hardening contour at point 2; (c) Fracturing contour at point 1; and (d) Fracturing contour at point

Figure 10. Stress rebar contour of specimen: (a)AS-5HT; (b) AS-7HT

In [8], after the concrete cover spalls, the confining bar was found to be yielded. The failure of tested specimens was signed by buckling of longitudinal bars due to the absence of concrete covers and perimeter ties. Figure 10 shows the simulation of bars that are yield. At peak shear load, the tensile stress in the transverse bars are almost near it yield stress. There are variations in the confining bar stress as function of the column depth which caused by strain gradient. As shown in figure 10, in the tension region, the confining bar stress was found to be the lowest. The length of the plastic hinges, which is associated with the length of the longitudinal bar that are yield, are 470 mm for AS-5HT and 450 mm for AS-7HT. Furthermore, the buckling of longitudinal bars can be observed clearly in Figure 11 which present the condition in the end of simulation before all specimens fail. The absence of concrete cover
and transverse reinforcement is affected to high compressive stress in longitudinal bars and the occurrence of buckling.

Figure 11. Buckling of longitudinal bars

Figure 11 shows buckling of the longitudinal bars. As shown in figure 11, the longitudinal bars that are in the extreme concrete fibre under compression were found to be curved which clearly indicates that the bar are buckled. The confining bars is also deformed outward which indicates that the concrete inside the ties are dilated outward. This buckling of bar phenomena is important and should be incorporated in the model by 2nd order effect analysis option being enabled.

6. Conclusions
This paper presents finite element analysis of high-strength RC column under constant axial load and shear force that generate a single curvature column. The numerical investigations are executed using the 3D-NLFEA program which incorporates the plasticity-fracture model [14]. Two specimens are adopted from the available literature [8]. The different of each specimen is emphasized on the volumetric ratio of the transverse reinforcement. The comparison of specimen behaviour from the experiment and 3D-NLFEA result are presented in the form of shear load versus the tip deflection diagram. The prediction using 3D-NLFEA agrees well with the test result. The peak shear prediction was within the range for AS-5HT and slightly higher for AS-7HT. The prediction of the load-deflection behaviour during the softening phase was found to be conservative compared to the test results.

The cover that spalls were shown clearly in the deformed shaped at the softening region. The hardening parameters propagates from the extreme concrete fibre in compression to the side concrete cover. By using the 2nd order analysis, the buckling of bars can also be captured, and the P-Delta effect behaviour of the whole RC column can also be considered. Further research should be carried out with varying input data such as higher concrete strength, different level of axial load, and the use of high-yield strength of the bars. Furthermore, simplified model such as fiber-based model [35] can be improved from parametric studies carried out in the future research.

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