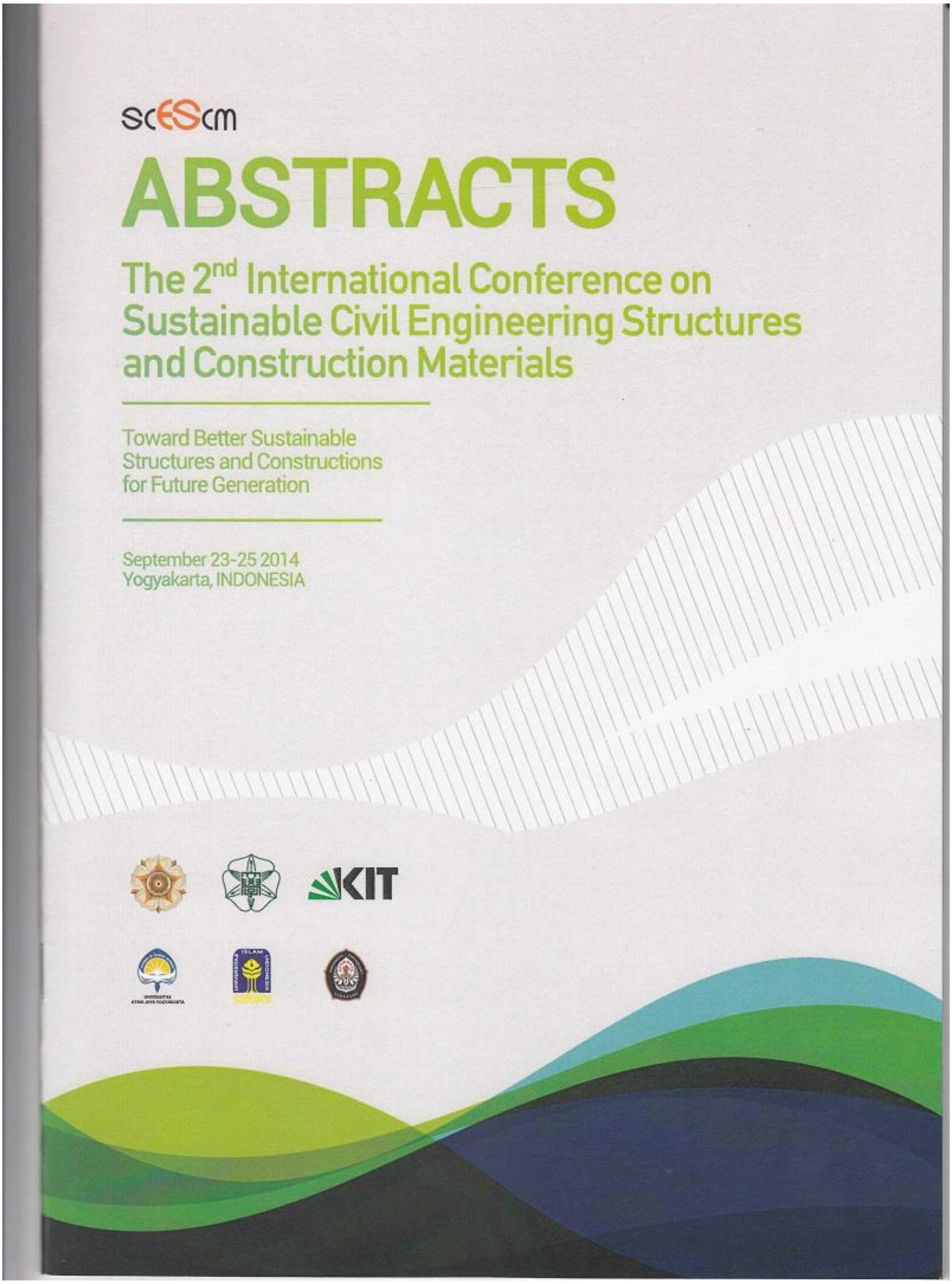

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ABSTRACTS

The 2nd International Conference on Sustainable Civil Engineering Structures and Construction Materials

Toward Better Sustainable
Structures and Constructions
for Future Generation

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LIST OF PAPER ID

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KEY 2	Toward Green Concrete for Better Sustainable Environment *Prof. Bambang Suhendro, Universitas Gadjah Mada, Indonesia*
KEY 3	Material Conditions Necessary for Strengthening Concrete Structures *Prof. Tamon Ueda, Hokkaido University, Japan*
KEY 4	Utilization of Industrial by- products in concrete *Prof. Rafaat Siddique, Thapar University, India*

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INV 1	Suistainable Seismic Design a Continuing Evolution *Prof. Stephen Pessiki*
INV 2	Force Prediction of Cylindrical Steel Beams on Pinned-pinned Supports under Axial Load by Vibration Technique *Prof. Priyosulistyo*
INV 3	State-of-the-Art Report on Partially-Prestressed Concrete Earthquake-Resistant Building Structures for Highly-Seismic Region *Prof. I.Gusti Putu Raka*
INV 4	Integrated Project Delivery (IPD) for maximizing design and construction considerations regarding sustainability *Prof. Barry. Jones*
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Studies on the Provisions of Confining-Reinforcement for High-Strength Concrete Column

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Abstract: It is general knowledge that the design of earthquake-resistant structures for high-strength concrete columns requires confining-reinforcement with a relatively high volumetric ratio to ensure the ductility of the structure. This implies that the mechanical behavior of high-strength concrete differs significantly from the behavior of normal-strength concrete. However, the provisions on the minimum volumetric ratio of confining-reinforcement contained in the Indonesian Concrete Code (SNI 2847-2013) is essentially derived from the test results for normal-strength concrete. This paper studies the confining-reinforcement provisions used in several standards, i.e., SNI 2847-2013, ACI-2011, NZS-2006 and CSA-2004, to determine the ductility of the concrete columns. The case study is based on the analysis of the cross-section of high-strength concrete columns, the parameters that affect the strength, and by evaluating the value of the column's cross-section curvature ductility. The study results showed that the equation for confining-reinforcement adopted in the SNI 2847-2013 is very conservative compared to other codes when applied to low axial load levels (≤0.2), but is relatively less conservative if the axial load level is greater than 0.3.

Keywords: high-strength concrete, column, confinement, reinforcement, ductility

1 INTRODUCTION

1.1 Background

Numerous comprehensive studies concerning the behavior of materials and structural components made of high-strength concrete have been conducted [Li & Park 2004, Paultre & Legeron 2008, Antonius & Imran 2012]. The resulting design equations have also been proposed and partially implemented in planning standards in each corresponding country. High-strength concrete has a brittle behavior; therefore, the structural ductility behavior becomes a major issue in the design of high-strength concrete structures, especially ductility in structural columns located in high earthquake zones.

The assemblage of lateral reinforcement as confining-reinforcement is intended to improve the ductility of concrete columns, underlining the importance of the role of reinforcement [Paultre & Legeron 2008, Subramanian 2011]. The confining-reinforcement design equations contained in the applicable Indonesian Concrete Code today is the SNI 2847-2013. The equations for square cross-sections are:

$$\frac{A_{sh}}{sh_c} = 0.3\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \qquad (1)$$

$$\frac{A_{sh}}{sh_c} = 0.09\frac{f'_c}{f_y} \qquad (2)$$

Equation (1) is used to design the structure under static loading, and equation (2) is to design structures under seismic loads. The design equations show a direct relationship between the volumetric ratio of confining-reinforcement and the concrete compressive strength. The use of high-strength concrete structures will have implications on the required increase in the confining-reinforcement volumetric ratio. To achieve the appropriate confining-reinforcement volumetric ratio for circular cross-section columns as mandated by the above design equation, a technique of spacing reduction can be applied. Meanwhile, in the case of a square cross-section column, besides a spacing reduction, one can also conduct a variation in confining-reinforcement configuration. To maintain sufficient confining-reinforcement spacing for concrete casting purposes, medium-strength to high-strength confining steel can be used [Bayrak and Sheikh 2004, Li & Park 2004, Antonius 2014]. The experimental test results also show that the ductility of high-strength concrete columns can be maintained properly if high-strength steel is used.

The research development of high-strength concrete columns has not been implemented into the design equations of the SNI 2847-2013, since the standards are derived from the research results on normal-strength concretes. The behavior of high-strength concrete therefore needs to be studied in greater depth, in particular the provision of confining-reinforcement applied to high-strength concrete columns as adopted in the SNI.

1.2 Objective

This paper discusses the design equations adopted in the SNI 2847-2013 and assesses these with the confining-reinforcement design provisions as mandated in the ACI 318-11, NZS 3101-2006 and CSA 2004 standards. The objective of this study is to evaluate the feasibility of confining equations based on the SNI in the design of high-strength concrete columns. The discussion is focused on the behavior of the resulting ductility because it is very closely related to structures located in the earthquake zone. This case study is limited to columns with square cross-sections, since for this type of column the configuration in confining-reinforcement can be varied. Further, the design equations are limited for columns under static loadings only.

2 CODE PROVISIONS FOR CONFINING-REINFORCEMENT OF SQUARE SECTION

The confining-reinforcement design equation used by the SNI 2847-2013 and ACI 318-11 (2011) for square cross-sections is as seen in equation (1). The difference lies in the upper limits of the yield stress. In the SNI, a limit yield stress up to 700 MPa was set, as upper bound, while for the ACI a value up to 10,000 psi yield stress (~ 688 MPa) is allowed. The design equation in the SNI and the ACI were derived with the philosophy that the cross section of the concrete core can maintain its strength after the concrete cover spalls. The equation does not directly express the degree of ductility of the structure.

Meanwhile the minimum confining-reinforcement volumetric ratio based on the NZS 3101 2006 is as follows:

$$\frac{A_{sh}}{sh_c} = \frac{(1.3 - \rho_t m)}{3.3} \frac{A_g}{A_c} \frac{f'_c}{f_{yt}} \frac{N_o^*}{\phi f'_c A_g} - 0.006 \qquad (3)$$

To prevent buckling of longitudinal reinforcement, the volumetric ratio also must satisfy the following equation:

$$A_{tie} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s_h}{d_b} \qquad (4)$$

Where:

$$m = \frac{f_{yl}}{0.85 f'_c} \qquad (5)$$

The above NZS equation accommodates the influence of axial load levels on a structure to the volumetric confining-reinforcement ratio. This statement is also explained by Kristianto & Imran (2013). A provision in the NZS noted that the reinforcement used for confinement purposes is permitted to reach a yield stress of 800 MPa. According to Li & Park (2004), the equation above is the result of research conducted by Watson et al. (1992), and it was noted that these equations are not directly applicable to the design of high-strength concrete columns with normal- to high-strength steel.

The confining-reinforcement design equation based on the CSA-2004 for a square cross-section column is as follows:

$$\rho_s = 0.2 k_n k_p \frac{A_g}{A_c} \frac{f'_c}{f_y} \qquad (6)$$

Where k_p is the level of axial load, and k_n is the effect of the amount of longitudinal reinforcement of the section, with:

$$k_n = n_\ell / (n_\ell - 2) \qquad (7)$$

The CSA limits the yield stress of confining-reinforcement (f_y) to 500 MPa. The CSA equation actually accommodates the influence of the axial load and the amount of longitudinal reinforcement.

The design equation in the two standards is based on the required confining-reinforcement that increases significantly when the structural column is designed for strong earthquakes, since this will escalate the axial load acting on the column. It can be concluded that the necessary confining-reinforcement is highly dependent on the size of the acting axial load.

3 COMPARATIVE STUDY

This comparative study was carried out for all the design provisions as outlined in the previous chapters, and was performed on two types of confining-reinforcement configurations of columns A and B as shown in Figure 1. Material properties are as follows:

- Two cases of concrete compressive strength (f'_c), 70 and 90 MPa
- cross-sectional dimensions are 500x500 mm
- concrete cover is 40 mm

- longitudinal reinforcement diameter is 22 mm with a yield stress (f_{yl}) of 480 MPa
- the confining-reinforcement has a diameter of 12 mm, with a yield stress (f_y) ranging from 400 MPa, 600 MPa to 800 MPa

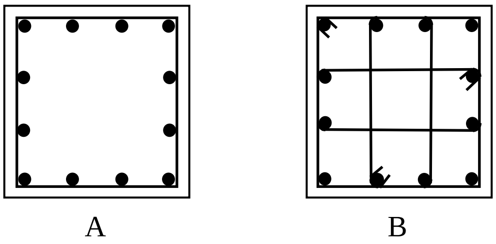

Figure 1. Sectional column

Figures 2 and 3 show the design provision comparison as a function of the minimum volumetric ratio to the axial load levels, for columns A and B. The steel has a yield stress of 400 MPa.

The figures show that for concrete with a compression strength of 70 and 90 MPa and for axial loading levels up to 0.2, the provision of the NZS and CSA are significantly lower than the SNI and ACI. At axial loading levels of 0.3, only the CSA provision was lower, when compared to the SNI and ACI. For an axial load level of 0.4, the confining-reinforcement provisions as mandated by the NZS and CSA are higher than the requirement in the SNI and ACI.

Additional results are shown in Figures 4 and 5. Here the yield stress of the confining-reinforcement is increased to 600 MPa. Similarly to the previous findings, for an axial load level of 0.2 the provisions for the minimum confining-reinforcement based on the NZS and CSA are also lower than the provisions mandated by the SNI and ACI. At an axial load level equal to 0.3 the same result as was observed for the confining-reinforcement with a yield stress of 400 MPa, i.e., the CSA provisions are below that of the SNI and ACI. However, the result of the NZS provision is higher than those of the SNI and ACI. But for axial load levels of 0.4, the SNI and ACI predicted are lower outcome than the other two standards, the NZS and CSA.

The utilization of high-strength confining-reinforcement steel (f_y=800 MPa) also has consequences for the minimum confining-reinforcement that should be assembled. Figures 6 and 7 show that based on the SNI and CSA standards for an axial load of 0.3 the confining-reinforcement volumetric ratio decreases, when compared to the lower yield strengths. However, the values approach the provisions of the NZS and CSA closely. At axial load levels of 0.4 the provisions of the NZS are the most conservative when compared to the SNI and ACI.

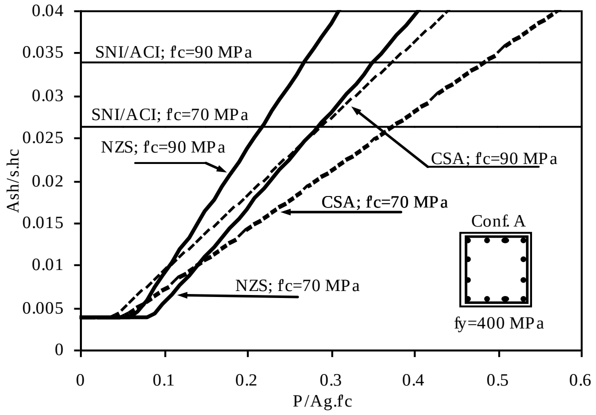

Figure 2. Comparison of minimum confining-reinforcement provisions; A configuration, f_y = 400 MPa

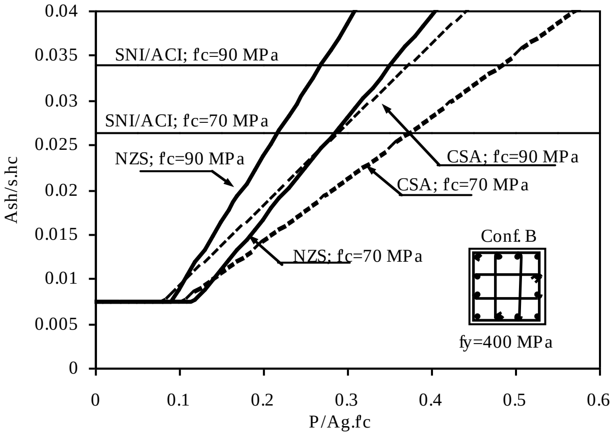

Figure 3. Comparison of minimum confining-reinforcement provisions; B configuration, f_y = 400 MPa

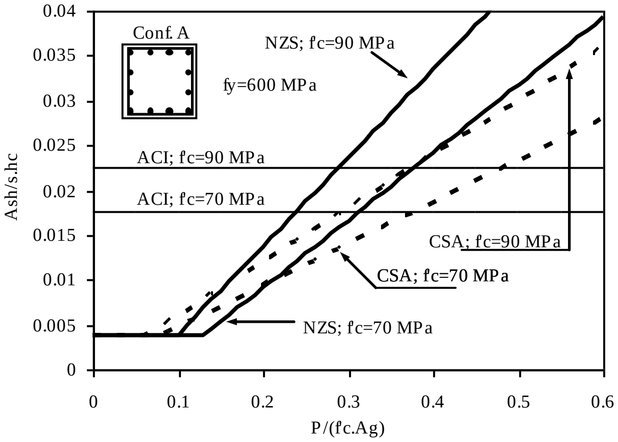

Figure 4. Comparison of minimum confining-reinforcement provisions; A configuration, f_y = 600 MPa

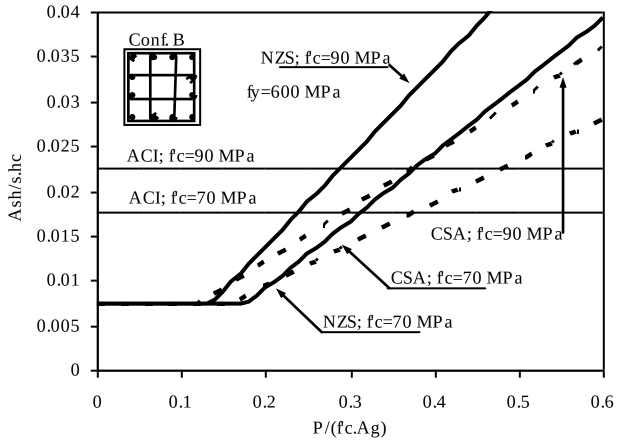

Figure 5. Comparison of minimum confining-reinforcement provisions; B configuration, f_y = 600 MPa

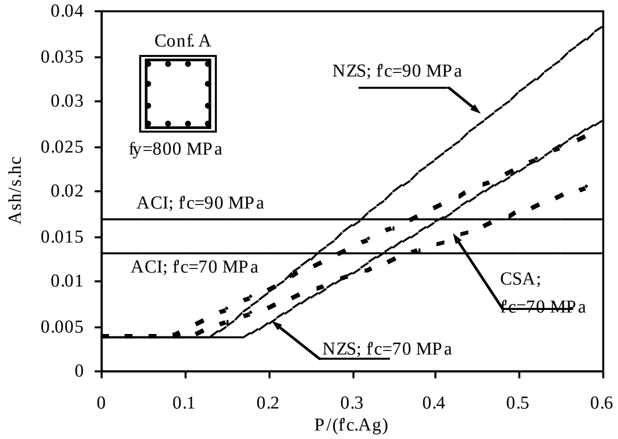

Figure 6. Comparison of minimum confining-reinforcement provisions; A configuration, f_y = 800 MPa

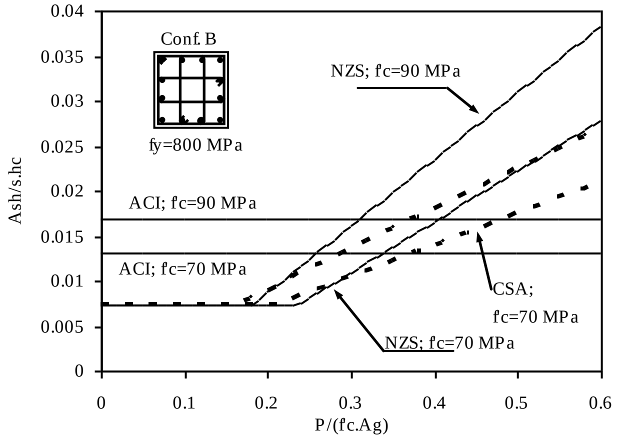

Figure 7. Comparison of minimum confining-reinforcement provisions; B configuration, f_y = 800 MPa

The result of the comparison indicates that for minimum confining-reinforcement steel for moderate axial load levels of 0.2 to 0.3, the SNI and ACI are very conservative. However, for the higher axial load levels, the provisions for the confining-reinforcement are below than the NZS and CSA. These findings implicate that the SNI and ACI standards are less profitable for low to moderate axial load levels.

4. COLUMN DUCTILITY BEHAVIOUR

At further stages, the influence of minimum confining-reinforcement designed based on the above-mentioned standards to the ductility behavior is evaluated. The evaluation is based on the moment-curvature cross-section behavior of the column. For this study, a concrete compressive strength of 70 MPa was taken. The specifications and strengths of the material remained unchanged, and the reinforcement configurations were as shown in Figure 1. The level of applied axial load is set to the highest, equal to 0.4. This was favored since at this load level the provisions from the SNI and ACI provisions are lower that of than the NZS and CSA. The high-strength concrete confinement model was based on the model as proposed by Antonius (2011); the stress-strain model is showed in Figure 8.

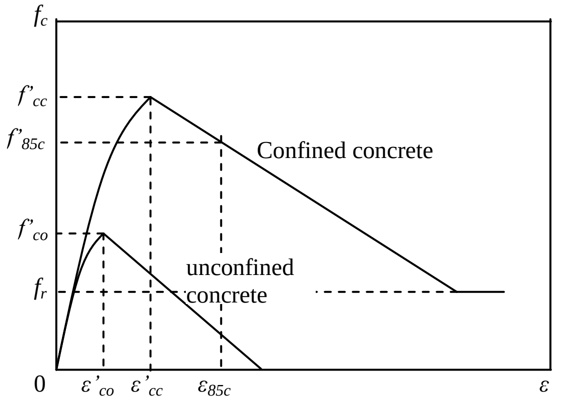

Figure 8. Stress-strain model of confined high-strength concrete [Antonius, 2011]

From the figure above, the following mathematical expressions were derived.

$$f_c = \frac{f'_{cc}\left(\frac{\varepsilon_c}{\varepsilon'_{cc}}\right).r}{r-1+\left(\frac{\varepsilon_c}{\varepsilon'_{cc}}\right)^r} \quad ; \ \varepsilon_c \le \varepsilon'_c \tag{8}$$

$$f_c = f'_{cc} - \left(\varepsilon_c - \varepsilon'_{cc}\right)\frac{0.15.f'_{cc}}{\left(\varepsilon_{85c} - \varepsilon'_{cc}\right)} ; \ \varepsilon_c > \varepsilon'_c \tag{9}$$

$$r = \frac{E_c}{E_c - \left(f'_{cc} / \varepsilon'_{cc}\right)} \tag{10}$$

$$E_c = 3400\sqrt{f'_c} + 4800 \tag{11}$$

$$K = \frac{f'_{cc}}{f'_{co}} = 1 + 3.7\left(\frac{f_{lat.}}{f'_{co}}\right)^{0.9} \qquad (12)$$

$$\varepsilon'_{cc} = \varepsilon'_{co}\left[1.94(K-1)+1\right] \qquad (13)$$

$$\varepsilon_{co}' = 0{,}0004 \,.\, (f'_{co})^{0.45} \qquad (14)$$

Stress of the confining-reinforcement at peak response for a square section is:

$$f_s = E_s\left\{0{,}0004.\ln\left[\frac{\left(\frac{s}{D_c}\right)}{\rho_s\sqrt{f'_c}}\right] + 0{,}002\right\} \le f_y \qquad (15)$$

$$\varepsilon_{85c} = \varepsilon_{cc}' + 10^{-5}.\, e^{3.7K} \qquad (16)$$

$$f_r = 0{,}25\, f'_{cc} \qquad (17)$$

The ductile column behavior refers to the definition as expressed by Li & Park (2004). After the spalling of the concrete cover, the moment in the column increases, exceeding or at least equaling the moment at the first peak. Alternately, a relatively flat curve will result (condition 1). On the other hand, a less ductile column is characterized by a reduction in moment capacity, subsequent to cover spalling. It can also be said that the moment is lower than the first peak (condition 2). A more detailed description of the column's ductility behavior is shown in Figure 9.

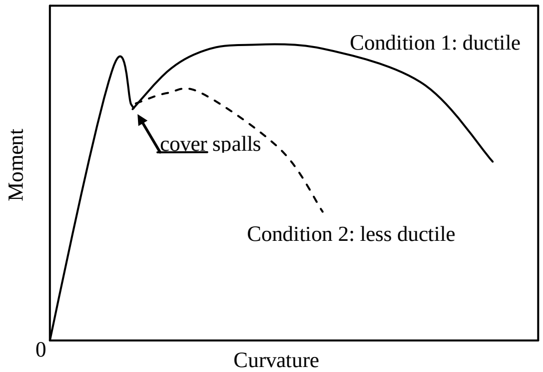

Figure 9. Definition ductile columns

4.1 The Influence of Axial Load Levels

Figure 10 shows the moment-curvature behavior of a columns with the A configuration. The confining-reinforcement steel yield stress varies from 400, 600 to 800 MPa. At the relatively low axial load levels of 0.2 it is shown that the curve is relatively flat after cover spalling. The opposite is seen for high axial load levels of 0.4.

The moment-curvature behavior for column B is based on the minimum confining-reinforcement design of the SNI and ACI. It is shown that the moment declines after cover spalling. This phenomenon is true for both normal and high-strength confining-reinforcement steel (Figure 11).

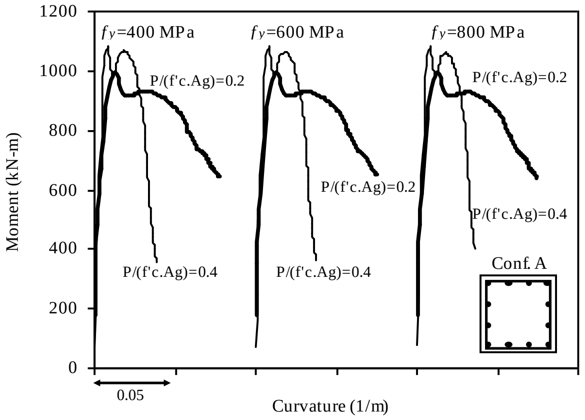

Figure 10. Behavior of Moment-curvature configuration A, the variation f_y

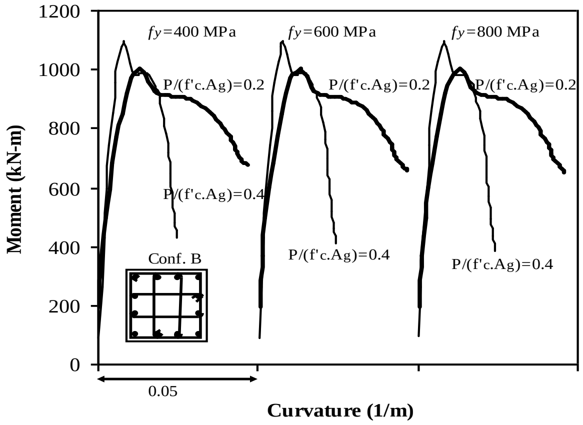

Figure 11. Behavior of Moment-curvature configuration B, the variation f_y

4.2 Evaluation on the Moment-curvature Behavior based on each Standard

The column ductility behaviors based on each standard are demonstrated by their moment-curvature curves and are shown in Figure 12, 13 and 14. Generally, the column ductility as provided by the provisions of the NZS and CSA are better when compared to the SNI and CSA, although the graphs also suggested that the reinforcing provision confinement adopted in the NZS is the most conservative. Observing the column with the A reinforcement configuration, it can be seen that the moment as predicted by the NZS provision always increases significantly after cover spalling. The increase in moment even exceeded the first peak.

The ductility behavior evaluation of column B is based on the minimum volumetric ratio, resulting in a maximum spacing which is far below the spacing of column A. The observation of curves suggested that after cover spalling all standards tend to result in a less ductile behavior.

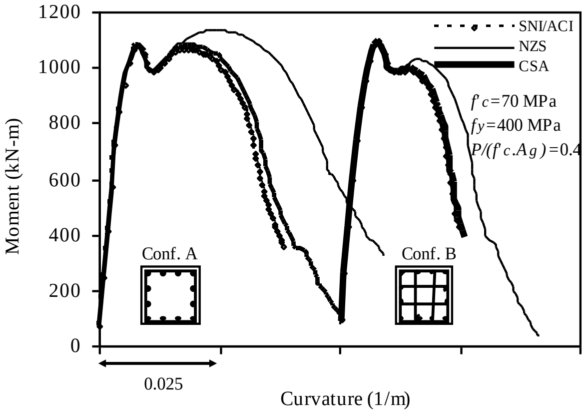

Figure 12. Comparison of moment-curvature of each standard, f_y=400 MPa

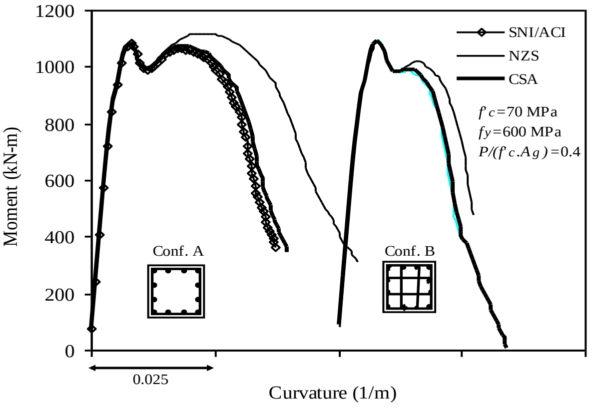

Figure 13. Comparison of moment-curvature of each standard, f_y=600 MPa

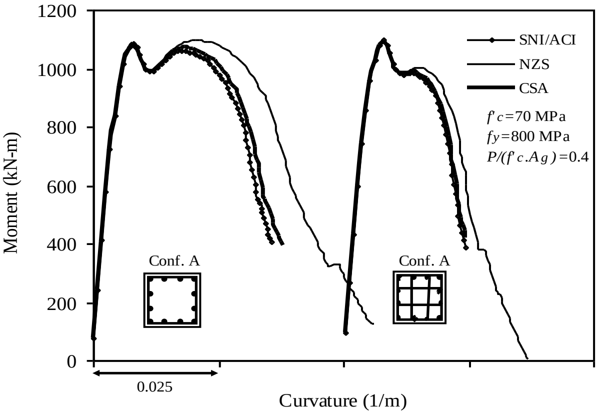

Figure 14. Comparison of moment-curvature of each standard, f_y=800 MPa

Furthermore, high-strength confining-reinforcement steel is used to improve the ductility of high-strength concrete columns. The confining-reinforcement for column B is reduced to a minimum so that the volumetric ratio of the assembled confining-reinforcement will be higher than what is required (resulting in an approximately similar spacing as column A). It was found that the ductility of the column increased significantly (Figure 15).

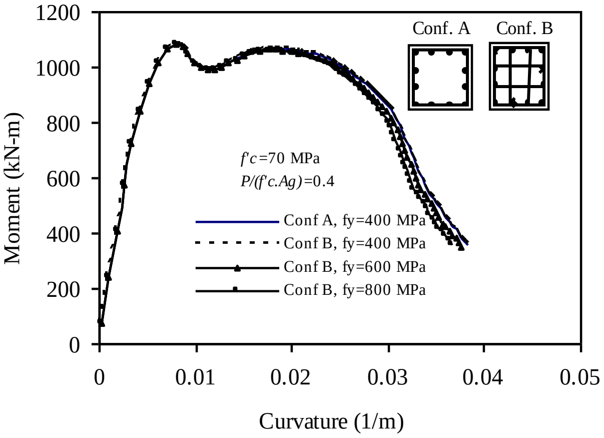

Figure 15. Improved ductility of the column configuration B

5 CONCLUSION AND RECOMENDATION

5.1 Conclusion

From the result of the studies as discussed, the following conclusions can be drawn:

1. The provisions for minimum confining-reinforcement based on the SNI and ACI does not consider variability in axial load levels so that the ductility behavior remains unchanged, despite a change in the earthquake magnitude.
2. Since the provisions of confining-reinforcement of the SNI and ACI code do not take into account the effects of axial load levels, the outcome will be underestimated if the structure is located in a strong earthquake-zone.
3. The use of high-strength confinement steel is one solution to maintain a column's ductility.
4. The ductility of columns can be improved by simulating the design parameter (i.e., the configuration); utilizing the confining-reinforcement volumetric ratio, and optimizing the spacing as well as utilizing the use of high-strength steel.

5.2 Recommendation

Accommodating the level of axial load on the minimum confining-reinforcement provisions into the SNI is highly recommended. However, for this purpose, it is necessary to develop a comprehensive research on the behavior of columns with variations in axial load levels, both analytically and experimentally.

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NOTATIONS

A_{sh} = area of confining reinforcement
b_c = width of core column
E_c = modulus elasticity of concrete
ε_c = strain of concrete
ε'_{co} = peak strain of unconfined concrete
ε'_{cc} = strain of confined concrete at peak response
ε_{85c} = strain of confined concrete at 85% of confined concrete peak stress
f'_{co} = peak stress of unconfined concrete
f_c = stress of concrete
f'_c = compressive strength of concrete cylinder 150/300 at 28 days
f'_{cc} = peak stress of confined concrete
f_{lat}. = lateral stress
f_y = yield stress of confining-reinforcement/ steel
f_{yl} = yield stress of longitudinal reinforcement
f_r = residual stress of confined concrete
f_s = stress of confining-reinforcement at peak response
h_c = length of core column
K = strength enhancement of confined concrete
s = spacing of confining-reinforcement (centre to centre)
ρ_s = volumetric ratio of confining-reinforcement
ρ = ratio of longitudinal reinforcement
n_l = number of longitudinal reinforcement