FLEXURAL-STRENGTH OF REINFORCED CONCRETE COLUMNS WITH MIXED ULTRA HIGH AND NORMAL STRENGTH STEEL BARS FOR THE LONGITUDINAL REINFORCEMENT

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ABSTRACT

to give columns a high degree of protection against premature yielding under severe earthquakes, a mixed use of ultra high and normal strength steel bars for the longitudinal reinforcement is being adopted as an alternative. The flexural strength beyond the elastic limit of such kind of columns, however, could not be adequately predicted by use of the code's methods (SN-1992, NZS3101:1982, ACI318-89). Instead, two methods of Maeder et al (1984) and Sheikh and Yeh (1992), mentioned as Method A and Method B in this paper are proposed with a modification in determining the minimum usable compressive strength of concrete. As shown in the experimental results these proposed methods give a satisfied result.

INTRODUCTION

Normally, a column cross section has uniform normal strength of steel for all of their longitudinal bars reinforcement. However, to give columns a high degree of protection against premature yielding under severe earthquakes, a mixed use of ultra high and normal strength steel bars for the longitudinal reinforcement can be used as an alternative [Watanabe et al (1990), Satyarno et al (1993)]. For such kind of columns, the conducted laboratory tests showed that their flexural strength beyond the elastic limit could not be adequately predicted by use of the code's methods (SN-1992, NZS3101:1982, ACI318-89). Instead, two methods of Maeder et al (1984) and Sheikh and Yeh (1992), mentioned as Method A and Method B in this paper are proposed with a modification in determining the maximum usable compressive strength of concrete.

METHOD A

If a maximum usable strain at the extreme fiber of compressive concrete is prescribed, the compressive force per unit width, $C_c$, in a column section can be found by assuming an equivalent rectangular stress block, that is

$$C_c = \alpha \cdot \beta \cdot \gamma \cdot c$$

where

$\alpha$ = ratio of compressive concrete stress to control concrete cylinder strength, $f_c$, in equivalent rectangular concrete stress block,

$\beta$ = ratio of equivalent rectangular concrete stress block depth to neutral axis depth,

$\gamma$ = neutral axis depth.

The compressive force per unit width, $C_c$, in Eq. 1 acts at a distance $\beta c/2$ from the extreme fiber of the compressed concrete as shown in Fig. 1. Twisters $\alpha$ and $\beta$ in Eq. 1 are found by integrating the area under stress-strain curve using Eq. 3 and from the first moment of area about neutral axis calculated using Eq. 5.

$$\int_0^{f_c} d \varepsilon = \alpha \cdot \beta \cdot \gamma \cdot \frac{f_c \cdot d \varepsilon}{\varepsilon_{cm}}$$

therefore:

$$\alpha \cdot \beta \cdot \gamma = \frac{f_c}{\varepsilon_{cm}}$$

$$\int_0^{\varepsilon_{cm}} f_c \cdot d \varepsilon$$

Fig. 1. Equivalent rectangular concrete stress block.

First moment of area about neutral axis:

$$\int_0^{\varepsilon_{cm}} f_c \cdot d \varepsilon = \left(1 - \frac{\beta}{2}\right) \varepsilon_{cm} \int_0^{\varepsilon_{cm}} f_c \cdot d \varepsilon$$

therefore:

$$\beta = 2 - \frac{2}{\varepsilon_{cm}} \int_0^{\varepsilon_{cm}} f_c \cdot d \varepsilon$$

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In Eqs. 2 to 5, the concrete stress, $f_c$, is determined from a stress-strain model. To avoid difficulties in solving Eqs. 3 and 5, Mander et al. (1984) provided graphs to find $\alpha$ and $\beta$ using their concrete stress-strain model as shown in Figs. 2 and 3.

The main variables in determination of $\alpha$ and $\beta$ are $f_{c0}$, $E_c$ and $K$, where $f_{ce}$ and $f_{c0}$, are confined and unconfined concrete strength, respectively, and $\varepsilon_{ce}$ is strain at peak stress of confined concrete, $\varepsilon_{c}$, see Mander et al. (1984) or Satya-pra (1993) for detail explanation. For application of Method A the following equation is proposed to calculate $f_{ce}$ [Satya-pra (1993)]:

$$\varepsilon_{ce} = 0.00165 R_{shd} + (0.001 + \varepsilon_{c0})$$

where $\varepsilon_{ce}$ = strain at peak stress of confined concrete in Mander et al. (1984) concrete stress-strain model,

$R_{shd}$ = area ratio of ultra high strength steel to total area of longitudinal reinforcement.

**METHOD B**

In this method the stress-strain model of concrete proposed by Seikh et al. (1992) is used. The model, as shown in Fig. 4, is expressed as follows:

$$f_{c} = \begin{cases} 
K_{s} f_{cp} \left( \frac{\varepsilon_{c}}{\varepsilon_{cp}} \right) & \varepsilon_{c} < \varepsilon_{cp} \\
K_{s} f_{cp} [1 - Z(\varepsilon_{c} - \varepsilon_{cp})] & \varepsilon_{c} \geq \varepsilon_{cp}
\end{cases}$$

for $\varepsilon_{c} < \varepsilon_{cp}$:

$$f_{c} = K_{s} f_{cp} \left[ 1 - Z(\varepsilon_{c} - \varepsilon_{cp}) \right] \geq 0.3 K_{s} f_{cp}$$

(9)

for $\varepsilon_{c} \geq \varepsilon_{cp}$:

$$f_{c} = f_{cp}$$

(8)

Variables in Eqs. 7 to 9 are found from the following equations:

$$f_{ce} = K_{s} f_{cp}$$

(10)

$$f_{cp} = K_{p} f_{c}$$

(11)

$$K_{s} = 1 + \frac{B^{2}}{10.58 K_{p}} \left[ 1 - 5 \left( \frac{e_{c}}{B} \right)^{2} \left( 1 - \frac{5}{20} \right) \frac{e_{c}}{B} f_{c} \right]$$

(12)

$$e_{cp} = 0.55 K_{p} f_{c} e_{c}$$

(13)

$$e_{cp} = 0.0022 K_{s}$$

(14)

$$\frac{e_{c}}{e_{0}} = 1 + \frac{0.81}{C} \left[ 1 - 5 \left( \frac{e_{c}}{B} \right) ^{2} \left( \frac{e_{c}}{B} f_{c} \right) \frac{e_{c}}{B} \right]$$

(15)
\[ \begin{align*}
\varepsilon_\text{CS} &= 0.225 \rho_0 \sqrt{\frac{P}{f_c}} \\
Z &= 1.5 \rho_0 \sqrt{\frac{P}{f_c}} \\
\rho_0 &= 0.1 \sqrt{\frac{P}{f_c}} \\
P_{\text{occ}} &= k_p f_c' \left( A_{\text{co}} - A_h \right) 
\end{align*} \]

where

- \( \varepsilon_{\text{CS}} \) = strain at 85% of \( f_c' \) for the descending branch,
- \( \varepsilon_o \) = strain at peak stress of unconfined concrete,
- \( \varepsilon_{s1} \) = strain at the beginning of plateau in the concrete stress-strain model,
- \( \varepsilon_{s2} \) = strain at the end of plateau in the concrete stress-strain model,
- \( \rho_0 \) = volume ratio of the steel to the core concrete,
- \( A_{\text{co}} \) = area of core measured from center to center of the perimeter of the specimen,
- \( A_h \) = area of longitudinal steel,
- \( B \) = core width measured from center to center of the perimeter of the specimen,
- \( C \) = distance between laterally support longitudinal bars or AB/4
- \( f_c' \) = compressive strength of control concrete cylinders,
- \( f_p' \) = stress in the lateral steel,
- \( K_p \) = ratio of unconfined concrete stress in compression to \( f_c' \),
- \( K_s \) = strength enhancement factor for confined concrete,
- \( n \) = number of arches containing concrete that is not effectively confined, also equal to the number of laterally supported longitudinal bars,
- \( P_{\text{occ}} \) = axial load capacity of unconfined concrete,
- \( Z \) = tie spacing,
- \( \rho_0 \) = core width measured from center to center of the perimeter of the specimen,
- \( C \) = distance between laterally support longitudinal bars or AB/4

They took account of the fact that the behavior of concrete under concentric compression was different from that of the concrete under eccentric compression. The factors that influence the behavior of concrete under eccentric compression are strain gradient in the section and level of axial load. If these factors are taken into account, the previous equations are modified as follows.

To include the effect of strain gradient, Eq. 15 is modified to be:

\[ \varepsilon_{\text{CS}} = \frac{0.81}{c} \left[ 1 - 5 \left( \frac{T}{B} \right)^2 \right] + 0.25 \sqrt{\frac{P}{f_c}} \]

where \( c \) is the neutral axis depth. Eq. 19 shifts the line \( BD \) to the line \( BD' \) as shown in Fig. 4.

To take the effect of axial load into account in the estimation of \( f_c' \), Eq. 10 is modified as:

\[ f_c' = f_{cp} - f_{\sigma} \]

For the application of this method, the following equation is proposed to estimate the maximum usable strain at the extreme fiber of the compressive concrete, \( \varepsilon_{\text{cm}} \) [Sathyarto (1993)]:

\[ \varepsilon_{\text{cm}} = 0.00165 \varepsilon_{\sigma} + (0.001 + \varepsilon_{s2}) \]

Region 1, for \( \varepsilon_{\text{cm}} \leq \varepsilon_{s1} \):

\[ \beta = \frac{4 - \Omega}{2 (1 - \Omega)} \]

Region 2, for \( \varepsilon_{s1} < \varepsilon_{\text{cm}} \leq \varepsilon_{s2} \):

\[ \alpha = \frac{2 (\Omega - \Omega')^2}{3 (4 - \Omega)} \]

Region 3, for \( \varepsilon_{s2} < \varepsilon_{\text{cm}} \leq \varepsilon_{s3} \) where \( \varepsilon_{s3} \) is the strain at \( f_c' = 0.3 f_c' \):

\[ \beta = \frac{6 \Omega^2 - 4 \Omega + 1}{2 (\Omega (\Omega - 1))} \]

The value of \( \Omega, D \) and \( G \) are:

\[ \Omega = \frac{\varepsilon_{s2}}{\varepsilon_{s1}} \]

\[ D = \varepsilon_{s2} \]

\[ G = \varepsilon_{s1} \]

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EXPERIMENT

Commonly, a reinforced concrete column has only uniform normal strength steel bars for the longitudinal reinforcement. However, in this research ultra high and normal strength steel bars, with specified yield strength of 430 MPa an 1050 MPa, respectively, were used together in a column section. To study the behavior of such column under simulated severe seismic loading, three column units with dimension, arrangement of steel bars and loading set-up as shown in Fig. 6, were tested.

EXPERIMENTAL RESULTS AND DISCUSSION

In this paper, only the flexural strength of the columns is discussed. The cyclic behavior of such column will be presented in the next edition or can be read else where (Satyam (1993)). The comparison between flexural strengths of the columns units predicted

Fig. 6. Configuration of steel bars in column units and loading-set-up
### Table 1. Comparisons between calculated and experimental results of flexural strength.

<table>
<thead>
<tr>
<th>Column Units</th>
<th>Calculated flexural strength (Mks)</th>
<th>Experimental strength (Mks)</th>
<th>Calculated/Experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Method A</td>
<td>Method B</td>
<td>Method A</td>
</tr>
<tr>
<td>1</td>
<td>507</td>
<td>711</td>
<td>711</td>
</tr>
<tr>
<td>2</td>
<td>502</td>
<td>704</td>
<td>705</td>
</tr>
<tr>
<td>3</td>
<td>502</td>
<td>704</td>
<td>555</td>
</tr>
</tbody>
</table>

Note: The reduction factor is to be 1.0 in the code calculation.

From the experimental results shown in Table 1, it can be seen that:

1. The current code method cannot recognize the present of ultra high strength steel in the columns with mixed steel bars for their longitudinal reinforcement.
2. The code calculation results tend to underestimate the flexural strength of columns with mixed steel bars for their longitudinal reinforcement, especially for the ones with more ultra high strength steel content.
3. Method A and Method B give a better prediction of the columns flexural strength, but Method A gives a better results.
4. The more the content of ultra high strength steel, the higher the increase of flexural strength as follows.

### Table 2. The increase of flexural strength for the increase of ultra high strength steel content.

<table>
<thead>
<tr>
<th>Column Units</th>
<th>Content of ultra high strength steel (%)</th>
<th>Increase of flexural strength to Unit 3 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>55.2</td>
<td>36.0</td>
</tr>
<tr>
<td>2</td>
<td>29.1</td>
<td>12.17</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>-</td>
</tr>
</tbody>
</table>

5. It is proposed that the content of the ultra high strength steel in a column section is calculated using the following equation [Satyarno (1993)]:

$$\alpha_{ul} = \left[ \frac{0.25}{k - 1} \right] \left[ \frac{A_{/g}}{f_{y} \cdot (1 + \frac{A_{/g}}{A_{us}})} \right] \left( \frac{A_{/g}}{A_{us}} \right)$$

where

- $A_{g}$ = gross section area of concrete column,
- $A_{u}$ = area of normal strength steel,
- $A_{ul}$ = area of ultra high strength steel,
- $f_{y}$ = yield strength of normal strength steel,
- $f_{ul}$ = yield strength of ultra high strength steel.

### CONCLUSIONS

1. The mixed ultra high and normal steel strength bars for longitudinal reinforcement can give columns a higher protection against premature yielding under severe earthquake as the flexural strength is improved.
2. The code method to calculate the flexural strength of columns with mixed ultra high and normal strength steel bars for longitudinal reinforcement cannot be adequately applied. Instead, the proposed Method A and Method B can be satisfactorily used, in which Method A gives a better results.
3. Although it is not reported in this paper, see Satyarno (1993), columns with mixed ultra high and normal strength steel bars for longitudinal reinforcement also show a good ductility.

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### REFERENCES

ACI Committee 318, 1989, "Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary (ACI 318R-89)", American Concrete Institute, Detroit, MI.


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