APPENDIX ONE:
SEISMIC ASSESSMENT OF TEST UNITS

1. FLEXURAL STRENGTH CALCULATION OF BEAMS AND COLUMNS

Calculation Rules:

- Equilibrium equation using New Zealand Code Approach NZS3101:1995 was
  \[ \alpha_i f_c' a b + A'_s f_c' = A_s f_y + N' \]  
  (1.1)

  where: \( N' \) is the applied axial load, \( N' \) is positive if in compression and negative if in tension, see Fig. A-1; \( a \) is the depth of equivalent rectangular concrete compressive zone, \( a = \beta c \), and \( c \) is the depth of concrete compressive zone in calculating the strains using plane section assumption; \( A'_s \) and \( f_c' \) are the flexural compressive steel area and the compressive steel stress in flexural compression steel respectively; \( A_s \) and \( f_y \) are the flexural tension steel area and the steel tension stress in flexural tension steel; \( b \) is the width of the member.

- \( \beta \) is 0.85 if \( f_c' \leq 30 \) MPa, however, if \( f_c' > 30 \) MPa, \( \beta = 0.85 - 0.008 (f_c' - 30) \)

  but \( \beta \geq 0.65 \) has to be satisfied.

- \( \alpha_1 = 0.85 \) for \( f_c' \leq 55 \) MPa, and \( \alpha_1 = 0.85 - 0.004(f_c' - 55) \) for \( f_c' > 55 \) MPa, but \( \alpha_1 \) must be not less than 0.75.

- Compressive flexural steel strain is found by using plane section theory and assuming the extreme concrete compressive strain is 0.003 as follows:

  \[ \varepsilon_c' = \frac{c - d'}{c} \times 0.003 \]  
  (1.2)

  \[ f_c' = \varepsilon_c' E_c = 600 \times \frac{c - d'}{c} \]  
  (1.3)

- Flexural strength can be found using the following equation:

  \[ M_b = A_s f_y (d - d) + \alpha_1 f_c' a b (d' - a/2) \]  
  (1.4)

  \[ M_c = A_s f_y (d - \frac{h}{2}) + A_s f_c' (d - \frac{h}{2}) + \alpha_1 f_c' a b (\frac{h}{2} - \frac{a}{2}) \]  
  (1.5)

The diagrams for beam and column flexural strength calculation are illustrated in Fig. A-1.

The dimensions and reinforcing amounts used in beam and column strength calculation is listed in Table 1.1(a) and Table 1.1(b) for the interior beam column joint units and the exterior beam-column joint units respectively. It is noted that the two interior beam column joint units had the same dimensions and the same amount of reinforcing bars, and the four
exterior beam-column joint units had the same dimensions and the same amount of reinforcing bars except the beam bar hook details in exterior columns.

Table 1.1(a) Dimensions and Reinforcing Detail Parameters of Units 1 and 2

<table>
<thead>
<tr>
<th></th>
<th>For beam positive bending</th>
<th>For beam negative bending</th>
<th>For column bending</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_s$ (mm$^2$)</td>
<td>905</td>
<td>1809.6</td>
<td>1357</td>
</tr>
<tr>
<td>$A_s'$ (mm$^2$)</td>
<td>1809.6</td>
<td>905</td>
<td>1357</td>
</tr>
<tr>
<td>p (%)</td>
<td>0.656</td>
<td>1.31</td>
<td>1.13</td>
</tr>
<tr>
<td>p' (%)</td>
<td>1.31</td>
<td>0.656</td>
<td>1.13</td>
</tr>
<tr>
<td>d (mm)</td>
<td>460</td>
<td>460</td>
<td>260</td>
</tr>
<tr>
<td>d'(mm)</td>
<td>40</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>b (mm)</td>
<td>300</td>
<td>300</td>
<td>460</td>
</tr>
<tr>
<td>$E_s$ (MPa)</td>
<td>$2 \times 10^5$ MPa</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 1.1(b) Dimensions and Reinforcing Detail Parameters of Units EJ1 through EJ4

<table>
<thead>
<tr>
<th></th>
<th>For beam positive bending</th>
<th>For beam negative bending</th>
<th>For column bending</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_s$ (mm$^2$)</td>
<td>905</td>
<td>1357</td>
<td>905</td>
</tr>
<tr>
<td>$A_s'$ (mm$^2$)</td>
<td>1357</td>
<td>905</td>
<td>905</td>
</tr>
<tr>
<td>p (%)</td>
<td>0.656</td>
<td>0.983</td>
<td>0.468</td>
</tr>
<tr>
<td>p' (%)</td>
<td>0.983</td>
<td>0.656</td>
<td>0.468</td>
</tr>
<tr>
<td>d (mm)</td>
<td>460</td>
<td>460</td>
<td>420</td>
</tr>
<tr>
<td>d'(mm)</td>
<td>40</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>b (mm)</td>
<td>300</td>
<td>300</td>
<td>460</td>
</tr>
<tr>
<td>$E_s$ (MPa)</td>
<td>$2 \times 10^5$ MPa</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 1.2 Parameters $f_e^c$, $f_y$, $f', N^*$, $\alpha_1$, and $\beta$ for all units

<table>
<thead>
<tr>
<th>Unit</th>
<th>$f_e^c$ (MPa)</th>
<th>$f_y$ (MPa)</th>
<th>$f'$ (MPa)</th>
<th>$N^*$ (N)</th>
<th>$\alpha_1$</th>
<th>$\beta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit 1</td>
<td>43.8</td>
<td>321</td>
<td>318</td>
<td>0.0</td>
<td>0.85</td>
<td>0.74</td>
</tr>
<tr>
<td>Unit 2</td>
<td>48.9</td>
<td>321</td>
<td>318</td>
<td>800,000</td>
<td>0.85</td>
<td>0.70</td>
</tr>
<tr>
<td>Unit EJ1</td>
<td>34.0</td>
<td>321</td>
<td>318</td>
<td>0.0</td>
<td>0.85</td>
<td>0.82</td>
</tr>
<tr>
<td>Unit EJ2</td>
<td>29.2</td>
<td>321</td>
<td>318</td>
<td>0.0</td>
<td>0.85</td>
<td>0.85</td>
</tr>
<tr>
<td>Unit EJ3</td>
<td>34.0</td>
<td>321</td>
<td>318</td>
<td>1,800,000</td>
<td>0.85</td>
<td>0.82</td>
</tr>
<tr>
<td>Unit EJ4</td>
<td>36.5</td>
<td>321</td>
<td>318</td>
<td>1,800,000</td>
<td>0.85</td>
<td>0.80</td>
</tr>
</tbody>
</table>
To simplify the calculation of the member flexural strength, the parameters $f'_s$, $f_r$, $N^*$, $\alpha_1$, and $\beta$ are summarised in Table 1.2 for all units, and the calculated member flexural strengths is summarised in Table 1.3 for all the units.

**Table 1.3 Member Flexural Strength of Test Units**

<table>
<thead>
<tr>
<th>Unit</th>
<th>Beam Negative Bending</th>
<th>Beam Positive Bending</th>
<th>Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit 1</td>
<td>c = 53.6 mm</td>
<td>c = 39 mm</td>
<td>c = 38 mm</td>
</tr>
<tr>
<td></td>
<td>a = 39 mm</td>
<td>a = 29 mm</td>
<td>a = 28 mm</td>
</tr>
<tr>
<td></td>
<td>$M_b^- = 250$ kN-m</td>
<td>$M_b^+ = 129$ kN-m</td>
<td>$M_c = 108$ kN-m</td>
</tr>
<tr>
<td>Unit 2</td>
<td>c = 51.8 mm</td>
<td>c = 38 mm</td>
<td>c = 67 mm</td>
</tr>
<tr>
<td></td>
<td>a = 36 mm</td>
<td>a = 27 mm</td>
<td>a = 47 mm</td>
</tr>
<tr>
<td></td>
<td>$M_b^- = 251$ kN-m</td>
<td>$M_b^+ = 129$ kN-m</td>
<td>$M_c = 198$ kN-m</td>
</tr>
<tr>
<td>Unit EJ1</td>
<td>c = 48.2 mm</td>
<td>c = 40.2 mm</td>
<td>c = 34.5 mm</td>
</tr>
<tr>
<td></td>
<td>a = 40 mm</td>
<td>a = 33 mm</td>
<td>a = 28.3 mm</td>
</tr>
<tr>
<td></td>
<td>$M_b^- = 190$ kN-m</td>
<td>$M_b^+ = 129$ kN-m</td>
<td>$M_c = 120$ kN-m</td>
</tr>
<tr>
<td>Unit EJ2</td>
<td>c = 56.7 mm</td>
<td>c = 41.4 mm</td>
<td>c = 36.1 mm</td>
</tr>
<tr>
<td></td>
<td>a = 43 mm</td>
<td>a = 35.2 mm</td>
<td>a = 30.6 mm</td>
</tr>
<tr>
<td></td>
<td>$M_b^- = 189$ kN-m</td>
<td>$M_b^+ = 128$ kN-m</td>
<td>$M_c = 119$ kN-m</td>
</tr>
<tr>
<td>Unit EJ3</td>
<td>c = 48.2 mm</td>
<td>c = 40.2 mm</td>
<td>c = 154.8 mm</td>
</tr>
<tr>
<td></td>
<td>a = 40 mm</td>
<td>a = 33 mm</td>
<td>a = 127 mm</td>
</tr>
<tr>
<td></td>
<td>$f'_s = 600 \frac{c - 40}{c} &gt; 321$</td>
<td>$f'_s = 321$MPa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$M_b^- = 190$ kN-m</td>
<td>$M_b^+ = 129$ kN-m</td>
<td>$M_c = 392$ kN-m</td>
</tr>
<tr>
<td>Unit EJ4</td>
<td>c = 47.3 mm</td>
<td>c = 39.8 mm</td>
<td>c = 148.3 mm</td>
</tr>
<tr>
<td></td>
<td>a = 37.8 mm</td>
<td>a = 31.8 mm</td>
<td>a = 118.7 mm</td>
</tr>
<tr>
<td></td>
<td>$f'_s = 600 \frac{c - 40}{c} &gt; 321$</td>
<td>$f'_s = 321$MPa</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$M_b^- = 190$ kN-m</td>
<td>$M_b^+ = 129$ kN-m</td>
<td>$M_c = 400.0$ kN-m</td>
</tr>
</tbody>
</table>

The ratio of the sum of column moment capacity to the sum of beam moment capacity, calculated at the centre-line of the joint core, is listed in Table 1.4 for all test units.

**Table 1.4 Ratio of Column Flexural Strength to Beam Flexural Strength at a Joint**

<table>
<thead>
<tr>
<th>Units</th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit EJ1</th>
<th>Unit EJ2</th>
<th>Unit EJ3</th>
<th>Unit EJ4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sum M_c / \sum M_b$</td>
<td>0.63</td>
<td>1.16</td>
<td>2.07 (+)</td>
<td>2.07 (+)</td>
<td>6.76 (+)</td>
<td>6.90 (+)</td>
</tr>
</tbody>
</table>

(+) means that the value is associated with the positive beam bending direction and (-) means that the value is associated with the beam negative bending direction.
From Table 1.4, it is clear that Unit 1 would develop plastic hinges in columns, but Unit 2 would develop plastic hinges in beams. For all the four exterior beam-column joint units EJI through EJ4, the beam would develop plastic hinge.

2. **CALCULATION OF MEMBER YIELD CURVATURES**

1. **Depth of Concrete Compressive Zone at First Yield:**

   - **For Beams and Columns without Axial Load**

   Members should be still in the elastic range at first yield stage. In this case, the depth of the concrete compressive zone can be found by assuming a triangular distribution of concrete compressive stress. Under this assumption, k can be found as follows for the member with zero axial load [P1]:

   \[ k = \left[ (\rho + \rho')^2 n^2 + 2(\rho + \rho')^2 \frac{d'}{d} n \right]^{\frac{1}{2}} - (\rho + \rho') n \]  \hspace{1cm} (1.6)

   where \( k \) is the coefficient of the concrete compressive zone, \( kd \) is the depth of the concrete compressive zone, and \( n \) is the ratio of steel elastic modulus to concrete elastic modulus.

   \( E_s = 2 \times 10^5 \text{ MPa} \) for all units, and \( E_c = 3320 \sqrt{f_c'} + 6900 (\text{MPa}) = 28872, 30116, 26259, 24840, 26259 \) and \( 20958 \text{ MPa} \), according to NZS3101:1995, for Units 1, 2, EJ1, EJ2, EJ3 and EJ4 respectively. When ACI equation \( E_c = 4730 \sqrt{f_c'} (\text{MPa}) \) is used, \( E_c = 31303, 33076, 27580, 25560, 27580 \) and \( 28576 \text{ MPa} \), for Units 1, 2, EJ1, EJ2, EJ3 and EJ4 respectively. Using the second set of concrete elastic modulus, the ratio of steel elastic modulus to concrete elastic modulus is calculated and listed in Table 1.5.

   **Table 1.5 Ratio of Steel Elastic Modulus to Concrete Elastic Modulus**

<table>
<thead>
<tr>
<th>Unit</th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit EJ1</th>
<th>Unit EJ2</th>
<th>Unit EJ3</th>
<th>Unit EJ4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( n = E_s / E_c )</td>
<td>6.4</td>
<td>6.0</td>
<td>7.3</td>
<td>7.8</td>
<td>7.3</td>
<td>7.0</td>
</tr>
</tbody>
</table>

   - **For Columns with Compressive Axial Load**

   Say the compressive axial load is \( N^* \), assuming that concrete compressive stress be in elastic range, so use the following equation to find \( k \) at the first yield stage (see Fig.A-2),

   \[ A_s f_c + N^* = A_s \varepsilon_c E_s + \frac{1}{2} kd \varepsilon_c E_b b \] \hspace{1cm} (1.7)

   \[ \varepsilon_c' = \frac{(kd - d')}{(d - kd)} \varepsilon_c \] \hspace{1cm} (1.8)

   \[ \varepsilon_c = kd / (d - kd) \times \varepsilon_c \] \hspace{1cm} (1.9)
(2). Member Curvature and Moment Capacity at First Yield

\[ \Phi_y = \frac{f_y}{E_y d(1-k)} \]  

(1.10)

\[ M_y = A_y f_y \left( d - \frac{1}{3} kd \right) + A' y f'_y \left( \frac{1}{3} kd - d' \right) \quad \text{if} \quad N^* = 0.0 \]  

(1.11)

\[ M_y = A_y f_y \left( d - \frac{1}{2} h \right) + A' y f'_y \left( d - \frac{1}{2} h \right) + \frac{1}{2} kd f_c b \left( \frac{1}{2} h - \frac{1}{3} kd \right) \quad \text{if} \quad N^* \neq 0.0 \]  

(1.11)'

Calculated member curvatures at first yield using the method described above are summarised in Table 1.6 for all the units.

Calculation of yield curvatures of columns with axial load, such as, for Unit 2, Units EJ3 and EJ4, are described in detail below, because of its complexity.

**Table 1.6 Member Curvatures at First Yield**

<table>
<thead>
<tr>
<th>Unit</th>
<th>Beam Negative Bending</th>
<th>Beam Positive Bending</th>
<th>Column</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>k</td>
<td>( \Phi_y \left( \times 10^{-6} \right) )</td>
<td>k</td>
</tr>
<tr>
<td>Unit 1</td>
<td>0.311</td>
<td>5.0</td>
<td>0.213</td>
</tr>
<tr>
<td>Unit 2</td>
<td>0.304</td>
<td>5.0</td>
<td>0.209</td>
</tr>
<tr>
<td>Unit EJ1</td>
<td>0.288</td>
<td>4.9</td>
<td>0.23</td>
</tr>
<tr>
<td>Unit EJ2</td>
<td>0.30</td>
<td>5.0</td>
<td>0.24</td>
</tr>
<tr>
<td>Unit EJ3</td>
<td>0.29</td>
<td>4.9</td>
<td>0.23</td>
</tr>
<tr>
<td>Unit EJ4</td>
<td>0.28</td>
<td>4.9</td>
<td>0.23</td>
</tr>
</tbody>
</table>

(3). Detailed Calculation of Yield Curvature of Columns with Axial Load

The calculation of yield curvature of columns with axial load, such as the columns for Unit 2, Units EJ3 and EJ4 is described in detail as follows due to the complexity caused by the presence of column axial load.

* Column Yield Curvature of Unit 2

\[ f_y = 321 \text{MPa}, \quad A_y = A'_y = 1357 \text{ mm}^2, \quad f'_c = 48.9 \text{MPa}, \quad b = 460 \text{mm}, \quad d = 300-40 = 260 \text{ (mm)} \]

\[ \rho = \rho' = 0.0113, \quad E_c = 4730 \sqrt{f'_c} \text{ MPa} = 33076 \text{ MPa}, \quad E_s = 200000 \text{ MPa}, \quad n = 6 \]

Substituting the parameters above into Eqs.(1.7), (1.8) and (1.9) leads to
1357 × 321 + 800000 = 1357 \times \frac{260k - 40}{260(1 - k)} \times \varepsilon_y E_e + 0.5k \times \frac{260k}{1 - k} \varepsilon_y E_e \times 460

This gives \( k = 0.43 \)

Check, \( f_c = \frac{k}{1 - k} f_y / h = 40.35 \text{ MPa} < f_c' = 43.8 \text{ MPa} \), Approximately “ok”

\( \Phi_y = 10.7 \times 10^{-6} \)

Column Yield Curvature of Unit EJ3

Substituting the relevant parameters of Unit EJ3 in Table 1.1(b), Table 1.2 and Table 1.5 into equations 1.7, 1.8 and 1.9 leads to

\[
905 \times 321 + 1800000 = 905 \times \frac{42k - 4}{42(1 - k)} \times \varepsilon_y E_e + 0.5k \times \frac{420k}{1 - k} \varepsilon_y E_e \times 460
\]

This gives \( k = 0.48, \quad f_c = \frac{k}{1 - k} f_y / h = 40.87 \text{ MPa} > f_c' = 34 \text{ MPa}, \) so concrete is in non-linear state and linear concrete stress distribution is obviously not true. Hence using the following equation [C1]:

\[
A_s f_c' + \alpha_1 f_c c b = A_s f_y + N'
\]  

(1.12)

where: \( \varepsilon_c' = (c - d')/(d - c) \varepsilon_y \)

\[
f_c' = \frac{c - 40}{420 - c} f_y, \quad \alpha_1 \quad \text{and} \quad \beta \quad \text{are listed for different concrete stress states in Reference C1,}
\]

using trial method to find the “c”.

For concrete of \( f_c' \), \( \varepsilon_c = 0.00215 \) (Concrete Peak Strain)

(1). Try \( \varepsilon_c / \varepsilon_c' = 0.75, \quad \alpha_1 = 0.762, \quad \beta = 0.691 \)

so: \( A_s f_c' + \alpha_1 f_c c b = 905 \frac{c - 40}{420 - c} \times 321 + 0.691 \times 0.762 \times 34 \times c \times 460 = 2090505 \)

\[
905(c - 40) + 321 + 8235.112 c(420 - c) = 2090505(c - 40)
\]

\[
c^2 - 709.3c + 108061 = 0, \quad c = 221.6 \text{ mm}
\]

Check: \( \varepsilon_c = \frac{d - c}{c} \varepsilon_t = 1.44368 \times 10^{-3} \), so below yield, try again

(2) Try \( \varepsilon_c / \varepsilon_c' = 1, \quad \alpha_1 = 0.884, \quad \beta = 0.728 \)

so: \( A_s f_c' + \alpha_1 f_c c b = 905 \frac{c - 40}{420 - c} \times 321 + 0.884 \times 0.728 \times 34 \times c \times 460 = 2090505 \)

\[
c = 656.7c + 88414 = 0, \quad \text{so} \ c = 189 \text{ mm} \)
Check: \( \varepsilon_s = \frac{d-c}{c} \varepsilon_s = 2.6266 \times 10^{-3} \), so much bigger than first yield strain. Use interpolation method to find a good c,

\[
c = 221.6 + \frac{189 - 221.6}{2.6266 \times 10^{-3} - 1.44368 \times 10^{-3}} (1.605-1.44368) \times 10^{-3} = 217 \text{ mm}
\]

In that case, \( f_s' = 280 \text{ MPa}, \alpha_i = 0.762 + \frac{0.884 - 0.762}{189 - 221.6} (217 - 221.6) = 0.779 \)

\[
\beta = 0.691 + \frac{0.728 - 0.691}{189 - 221.6} (217 - 221.6) = 0.696
\]

\[
\Phi_{yc}^+ = \Phi_{yc}^- = \varepsilon_s / (d-c) = \frac{1.605 \times 10^{-3}}{420 - 217} = 7.91 \times 10^{-6}
\]

\[
M_{yc}^+ = M_{yc}^- = 905 \times 321 \times (230-40) + A_i f_s' (230-40) + \alpha_i f_c' \beta c b (230 - 0.5 \beta c) = 387609107 \text{ N-mm} = 388 \text{ kN-m}
\]

* Column Yield Curvature of Unit EJ4

\( f_s' = 321 \text{ MPa}, A_i = A_s = 905 \text{ mm}^2, f_c' = 36.5 \text{ MPa}, b = 460 \text{ mm}, d = 460 - 40 = 420 \text{ (mm)} \)

\[
\rho = \rho' = 0.468\%, E_c = 4730 \sqrt{f_c'} \text{ MPa} = 28576 \text{ MPa}, E_s = 200000 \text{ MPa}, n = 7
\]

Substituting the parameters above into Eqs.(1.7), (1.8) and (1.9) leads to:

\[
905 \times 321 + 1800000 = 905 \times \frac{42k - 4}{42(1-k)} \times \varepsilon_s E_s + 0.5 k \times \frac{420k}{1-k} \varepsilon_s E_s \times 460
\]

This gives \( k = 0.473 \)

Check: \( f_c' - \frac{k}{1-k} f_s' / n - 41 \text{ MPa} > f_c' = 34 \text{ MPa} \), so concrete is in non-linear state and Equation (1.12) should be used, similar to that for Unit EJ3.

Using trial method to find the "c".

For concrete of \( f_c' = 36.5 \text{ MPa}, \varepsilon_c' = 0.00215 \) (Concrete Peak Strain), use the values associated with \( f_c' = 34 \text{ MPa} \) due to unavailable data for \( f_c' = 36.5 \text{ MPa} \) in Reference C1.

(1). Try \( \varepsilon_i / \varepsilon_c' = 0.75, \alpha_i = 0.762, \beta = 0.691 \)

so: \( A_i f_s' + \alpha_i f_c' \beta c b = 905 \frac{c - 40}{420 - c} \times 321 + 0.691 \times 0.762 \times 36.5 \times c \times 460 = 2090505 \)

\( c^2 - 689.5c + 100658.6 = 0 \) so \( c = 210 \text{ mm} \)

Check: \( \varepsilon_s = \frac{d-c}{c} \varepsilon_s = 1.6125 \times 10^{-3} \), and it is close to steel yield strain of \( \varepsilon_s = 1.605 \times 10^{-3} \).
\[ f_s' = \frac{c - 40}{420 - c} f_y = 260 \text{ MPa} \]

\[ \Phi_{y'} = \Phi_y = \frac{\varepsilon_y}{(d - c)} = \frac{1.605 \times 10^{-3}}{420 - 210} = 7.66 \times 10^{-6} \]

\[ M_{y'} = M_y = 905 \times 321 \times (230 - 40) + A_s f_s' (230 - 40) + \alpha_1 f' c b (230 - 0.5 \beta c) \]

\[ = 392205015 \text{ N-mm} = 392 \text{ kN-m} \]

3. **Calculation of Member Ultimate Curvature of Test Units**

Member ultimate curvatures are calculated using the measured material strengths and assuming that the ultimate concrete compressive strain is 0.004. Similar to the flexural strength calculation, find the distance form the extreme compression fibre to the neutral axis, c, which satisfies the equilibrium equation (1.1).

\[ \alpha_1 f_s' a b + A_s f_s' = A_s f_s + N' \quad (1.1) \]

The previous equations 1.2 and 1.3, in the case of using the ultimate concrete compressive strain of 0.004, become:

\[ \varepsilon_s' = \frac{c - d'}{c} 0.004 \quad (1.2)' \]

\[ f_s' = 800 \frac{c - 40}{c} \quad (1.3)' \]

\[ \Phi_u = \frac{0.004}{c} \quad (1.13) \]

The calculated member ultimate curvature is listed in Tables 1.7 for tests on Units 1, 2, EJ1 through EJ4.
Table 1.7 Calculated Member Ultimate Curvature

<table>
<thead>
<tr>
<th>Unit</th>
<th>Beam Negative Bending</th>
<th>Beam Positive Bending</th>
<th>Column</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>c (mm)</td>
<td>$\Phi_x \times 10^{-5}$</td>
<td>c (mm)</td>
</tr>
<tr>
<td>Unit 1</td>
<td>50</td>
<td>8.0</td>
<td>39</td>
</tr>
<tr>
<td>Unit 2</td>
<td>50</td>
<td>8.0</td>
<td>37</td>
</tr>
<tr>
<td>Unit EJ1</td>
<td>46.7</td>
<td>8.6</td>
<td>40.2</td>
</tr>
<tr>
<td>Unit EJ2</td>
<td>48.6</td>
<td>8.2</td>
<td>41</td>
</tr>
<tr>
<td>Unit EJ3</td>
<td>46.2</td>
<td>8.7</td>
<td>40</td>
</tr>
<tr>
<td>Unit EJ4</td>
<td>46</td>
<td>8.7</td>
<td>40</td>
</tr>
</tbody>
</table>

Table 1.8 Calculated Member Curvature Ductility Factor $\mu_\phi$

<table>
<thead>
<tr>
<th>Unit</th>
<th>Part of the unit</th>
<th>Bending direction</th>
<th>$\Phi_x \times 10^{-6}$</th>
<th>$\Phi_x \times 10^{-6}$</th>
<th>$\mu_\phi$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit 1</td>
<td>Beams</td>
<td>Negative bending</td>
<td>5.0</td>
<td>80</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Positive bending</td>
<td>4.4</td>
<td>103</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td>Columns</td>
<td></td>
<td>8.6</td>
<td>106</td>
<td>12</td>
</tr>
<tr>
<td>Unit 2</td>
<td>Beams</td>
<td>Negative bending</td>
<td>5.0</td>
<td>80</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Positive bending</td>
<td>4.4</td>
<td>108</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>Columns</td>
<td></td>
<td>10.7</td>
<td>64</td>
<td>6</td>
</tr>
<tr>
<td>Unit EJ1</td>
<td>Beam</td>
<td>Negative bending</td>
<td>4.9</td>
<td>86</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Positive bending</td>
<td>4.5</td>
<td>99</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>Columns</td>
<td></td>
<td>4.8</td>
<td>113</td>
<td>24</td>
</tr>
<tr>
<td>Unit EJ2</td>
<td>Beam</td>
<td>Negative Bending</td>
<td>5.0</td>
<td>82</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Positive Bending</td>
<td>4.6</td>
<td>98</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>Columns</td>
<td></td>
<td>4.9</td>
<td>109</td>
<td>22</td>
</tr>
<tr>
<td>Unit EJ3</td>
<td>Beam</td>
<td>Negative Bending</td>
<td>4.9</td>
<td>87</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Positive Bending</td>
<td>4.5</td>
<td>100</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>Columns</td>
<td></td>
<td>7.9</td>
<td>24</td>
<td>3</td>
</tr>
<tr>
<td>Unit EJ4</td>
<td>Beam</td>
<td>Negative Bending</td>
<td>4.9</td>
<td>87</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Positive Bending</td>
<td>4.5</td>
<td>100</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>Columns</td>
<td></td>
<td>7.7</td>
<td>25</td>
<td>3</td>
</tr>
</tbody>
</table>
4. **Member Curvature Ductility Factor**

Based on the calculated member curvature at first yield (Table 1.6) and at ultimate stage (Table 1.7), the curvature ductility factors of members are computed and listed in Table 1.8 for all the test units.

5. **THE IMPOSED SHEAR FORCES ON THE MEMBERS:**

5.1 **Storey Shear Strength and Imposed Column Shear Forces**

The storey shear strength, $V_c$, of each unit is developed at the attainment of the theoretical flexural strengths of the critical members. For all the six tests, except the test on Unit 1, the theoretical storey shear force strength of the unit is dominated by the flexural strengths of the beams (beam).

$V_c$ is calculated as follows:

For interior beam-column joint units,

$$V_c = \frac{M_b^+ + M_b^-}{(1905 - 150)} \times \frac{1905}{3200} \text{(N)}$$

for weak beam-strong column systems, such as, Unit 2

$$V_c = \frac{M_b}{1600 - 250} \text{(N)}$$

for weak column-strong beam systems, such as, Unit 1

For exterior beam-column joint units, the storey shear force strength of the unit is dominated by the beam flexural strength,

$$V_c^+ = \frac{M_b^+}{1905 - 230} \times 1905/3200 \text{(N)}$$

for positive beam bending

$$V_c^- = \frac{M_b^-}{1905 - 230} \times 1905/3200 \text{(N)}$$

for negative beam bending

Note that the flexural moment capacity has a unit N-mm in above equations.

The theoretical storey shear force strength is summarised in Table 1.9 for all the units.

**Table 1.9** Storey Shear Force Strength of Test Units

<table>
<thead>
<tr>
<th>Unit</th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit EJ1</th>
<th>Unit EJ2</th>
<th>Unit EJ3</th>
<th>Unit EJ4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>80</td>
<td>128</td>
<td>46</td>
<td>67</td>
<td>46</td>
<td>67</td>
</tr>
<tr>
<td>$V_c$ (kN)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\nu_{ke}$ (MPa)</td>
<td>0.10 $\sqrt{f'_c}$</td>
<td>0.15 $\sqrt{f'_c}$</td>
<td>0.06 $\sqrt{f'_c}$</td>
<td>0.06 $\sqrt{f'_c}$</td>
<td>0.06 $\sqrt{f'_c}$</td>
<td>0.06 $\sqrt{f'_c}$</td>
</tr>
</tbody>
</table>
The imposed shear forces on members should be calculated at the development of the theoretical storey shear strength of the unit. Therefore, the imposed column shear force is actually the storey shear strengths of the unit, \( V_c \).

The maximum nominal column shear stress at the theoretical flexural strength of the unit is given by

\[
\nu_{n,c} = \frac{V_c}{b_c d_c}
\]  

(1.14)

Hence the nominal column shear stress at the theoretical flexural strength of the unit is 0.10\( \sqrt{f'_c} \), 0.15\( \sqrt{f'_c} \), 0.06\( \sqrt{f'_c} \), 0.064\( \sqrt{f'_c} \), 0.06\( \sqrt{f'_c} \) and 0.06\( \sqrt{f'_c} \) MPa for Units 1, 2, EJ1 to EJ4 respectively.

### 5.2 Imposed Beam Shear Force

The imposed beam shear force should be calculated according to the storey shear force strength of the unit.

For a weak beam-strong column system, the maximum imposed beam shear forces are usually obtained at the development of beam negative flexural strength because beam negative flexural strengths are larger than beam positive flexural strengths. For a weak column-strong beam system, the imposed beam shear forces are obtained at the development of the system’s storey shear force strength using force equilibrium condition.

\[
V_b = V_c \times \frac{3200}{3810} = 67 \text{ kN}
\]  

for Unit 1

\[
V_b = M_b (\text{kN-mm}) \left(1905 - \frac{1}{2}h_b\right)(\text{mm}) = 143 \text{ kN}
\]  

for Unit 2

\[
V_b = M_b (\text{kN-mm}) \left(1905 - \frac{1}{2}h_b\right)(\text{mm}) = 113 \text{ kN}
\]  

for Unit EJ1 through EJ4

The imposed beam shear forces for all test units are listed in Table 1.10.

<table>
<thead>
<tr>
<th></th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit EJ1</th>
<th>Unit EJ2</th>
<th>Unit EJ3</th>
<th>Unit EJ4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_b ) (kN)</td>
<td>67</td>
<td>143</td>
<td>113</td>
<td>113</td>
<td>113</td>
<td>113</td>
</tr>
<tr>
<td>( \nu_{n,b} ) (MPa)</td>
<td>0.073( \sqrt{f'_c} )</td>
<td>0.15( \sqrt{f'_c} )</td>
<td>0.14( \sqrt{f'_c} )</td>
<td>0.15( \sqrt{f'_c} )</td>
<td>0.14( \sqrt{f'_c} )</td>
<td>0.14( \sqrt{f'_c} )</td>
</tr>
</tbody>
</table>

The maximum nominal beam shear stress at the theoretical flexural strength of the units is given by
\[ \nu_{x,b} = \frac{V_s}{b_w d} \] (1.15)

Hence the nominal beam shear stress at the theoretical flexural strength of the units is 0.073 \( \sqrt{f_c} \), 0.15 \( \sqrt{f_c} \), 0.14 \( \sqrt{f_c} \), 0.15 \( \sqrt{f_c} \), 0.14 \( \sqrt{f_c} \) and 0.14 \( \sqrt{f_c} \) MPa for Units 1, 2, EJ1 to EJ4 respectively.

### 5.3 Imposed Maximum Horizontal Joint Shear Force

The imposed horizontal joint shear force is

\[ V_{jh} = T_1 + T_2 - V_c \quad \text{for interior beam-column joints} \] (1.16)

\[ V_{jh} = T_b - V_c \quad \text{for exterior beam-column joint} \] (1.17)

where: \( T_1 \) and \( T_2 \) are the tensile forces in tension reinforcement of the left and right beams respectively for interior beam-column joints, when the storey shear strength is developed; \( T_b \) is the tensile forces in beam tension reinforcement for exterior beam-column joints, when the storey shear strength is developed.

For Unit 1, which was a weak column-strong beam system, the imposed horizontal joint shear force is estimated by assuming that the two beams share equally the imposed bending moment because the beams still in the elastic range. In elastic range, the beam steel tension stress, \( f' \), can be found by getting \( k \) using equation 1.6. With the known \( k \) and the known external bending moment, using equation 1.11 can give the correspondent beam steel tension stress. Beam steel tension forces, \( T_1 \) and \( T_2 \) then can be calculated based on beam steel tension stress.

Typically, external bending moment is 118 kN-m for the beams of Unit 1, the \( k \) is found to be \( k = 0.311 \) for beam negative bending of Unit 1 and \( k = 0.213 \) for beam positive bending of Unit 1. As a result, \( \varepsilon_s = 7.88 \times 10^{-4} \) and \( \varepsilon_p = 1.52E-03 \) for beam negative bending and positive bending respectively.

Therefore, \( V_{jh} = T_1 + T_2 - V_c = 483 \) kN for Unit 1

For the rest five tests, including tests on Unit 2, Unit EJ1 through Unit EJ4, the storey shear force strength of the unit is governed by the beam flexural strength, so the beam steel tension forces are the steel forces at yield level.

\[ V_{jh} = (A_s + A_y)f_y - V_c = 6 \times 452 \times 321 - 128000 \text{ (N) } = 744kN \quad \text{for Unit 2} \]

\[ V_{jh} = A_s f_y - V_c = 1357 \times 321 - 67500 \text{ (N) } = 368 \text{ kN} \quad \text{for Units EJ1 through EJ4} \]
Table 1.11  Imposed horizontal joint shear force (kN)

<table>
<thead>
<tr>
<th>Unit</th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit EJ1</th>
<th>Unit EJ2</th>
<th>Unit EJ3</th>
<th>Unit EJ4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_{jh}$ (kN)</td>
<td>483</td>
<td>744</td>
<td>368</td>
<td>368</td>
<td>368</td>
<td>368</td>
</tr>
<tr>
<td>$\nu_{jh}$ (MPa)</td>
<td>$0.5 \sqrt{f'_c}$</td>
<td>$0.8 \sqrt{f'_c}$</td>
<td>$0.3 \sqrt{f'_c}$</td>
<td>$0.3 \sqrt{f'_c}$</td>
<td>$0.3 \sqrt{f'_c}$</td>
<td>$0.3 \sqrt{f'_c}$</td>
</tr>
</tbody>
</table>

Similarly, the nominal horizontal joint shear stress at the development of the flexural strengths of the test units can be calculated using

$$\nu_{jh} = \frac{V_{jh}}{b_j h_c}$$  \hspace{1cm} (1.18)

It gives $0.5 \sqrt{f'_c}$, $0.8 \sqrt{f'_c}$, $0.3 \sqrt{f'_c}$, $0.3 \sqrt{f'_c}$, $0.3 \sqrt{f'_c}$ and $0.3 \sqrt{f'_c}$ MPa for Units 1, 2 EJ1 to EJ4 respectively. Alternatively, the nominal horizontal joint shear stress at the development of the flexural strengths of the test units is $0.08 f'_c$, $0.11 f'_c$, $0.05 f'_c$, $0.06 f'_c$, $0.05 f'_c$, $0.05 f'_c$ MPa for Unit 1, Unit 2, Unit EJ1, Unit EJ2, Unit EJ3 and Unit EJ4 respectively.

The imposed joint horizontal shear forces and the nominal horizontal joint shear stress are summarised in Table 1.11 for all test units.

6.  ESTIMATION OF SHEAR CAPACITY OF MEMBERS AND BEAM-COLUMN JOINTS

Estimation of the shear capacity was carried out using both the NZS3101 Method and the current seismic assessment procedures recommended by Park. Measured material strengths and a strength reduction factor of unity are used here.

6.1  NZS3101: 1995 Method

The probable shear force strengths of the plastic hinge regions are calculated using NZS3101: 1995 design provisions for structures designed for ductility. The shear strengths of other regions are calculated using the non-seismic design provisions of NZS3101: 1995. It is noted that NZS3101 does not have a method for calculating the shear strength of existing beam-column joints.

(1).  Beam Shear Force Capacity

According to NZS3101:1995, the beam shear force capacity is calculated as follows:

$$V_{pb} = \nu_c b_w d + \frac{A_f f_w d}{s} = k \sqrt{f'_c} b_w d + \frac{A_f f_w d}{s}$$  \hspace{1cm} (1.19)
where: \( \tau_c \) = nominal shear stress carried by the concrete mechanism, \( f_{c'} \) = probable concrete compressive strength, \( b_w \) = width of beam web, \( d \) = effective depth of beam, \( A_v \) = area of transverse shear reinforcement, \( \rho_w = A_t / b_w d \) and \( A_t \) is area of tension reinforcement.

In the non-seismic provisions of NZS3101: 1995,

\[
\tau_c = (0.07 + 10 \rho_w) \sqrt{f_{c'}} \quad (f_{c'} \text{ is in unit of MPa}) \tag{1.20}
\]

In plastic hinge regions,

\[
\tau_c = 0.0 \tag{1.21}
\]

For the beams of Unit 1, non-seismic provision is applied because the beams were not expected to form plastic hinges. For other test units, including Unit 2 and Units EJ1 through EJ4, \( \tau_c \) is taken as zero in calculating the beam shear force capacity. The calculated beam shear force capacity for Unit 1, Unit 2, Unit EJ1 through EJ4 using the method of NZS3101: 1995 are respectively 146 kN, 22 kN, 22 kN, 22 kN, 22 kN and 22 kN.

(2). Column Shear Force Capacity

According to NZS3101:1995, the column shear capacities are calculated as follows:

\[
V_{pc} = \tau_c b_w d + \frac{A_v f_{p}^d}{s} = V_{pc,c} + V_{pc,t} \tag{1.22}
\]

\[
\tau_c = \left(1 + \frac{3N^*}{A_g f_c} \right) \nu_b \tag{1.23}
\]

where: \( A_g \) = column gross sectional area, \( \rho_w \) = column tensile reinforcement ratio, \( b_w \) = column width, \( d \) = effective depth of column section, \( A_c \) = area of transverse reinforcement, \( f_{p}^d \) = yield strength of transverse reinforcement.

In non-seismic provisions of NZS3101:1995,

\[
\nu_b = \frac{5}{6} \sqrt{f_{c'}} = (0.07 + 10 \rho_w) \sqrt{f_{c'}} \tag{1.24}
\]

In plastic hinge regions where the axial load is less than 0.1 \( f_{c'} \),

\[
\tau_c = 0.0 \tag{1.25}
\]

Hence,

\[
V_{pc,c} = 0.0 \quad \text{for Unit 1}
\]

\[
= \left(1 + \frac{3N^*}{A_g f_c} \right) (0.07 + 10 \rho_w) \sqrt{f_{c'}} b_w d = 209 \text{ kN} \quad \text{for Unit 2}
\]
\[ V_{pc,c} = \left(1 + \frac{3N^*}{A_e f_c} \right) (0.07 + 10 \rho_w) \sqrt{f_c' \ b_w \ d} \]

\[
\begin{align*}
= 132 \text{ kN} & \quad \text{for Unit EJ1} \\
= 122 \text{ kN} & \quad \text{for Unit EJ2} \\
= 230 \text{ kN} & \quad \text{for Unit EJ3} \\
= 232 \text{ kN} & \quad \text{for Unit EJ4}
\end{align*}
\]

The contribution of column transverse reinforcement to the total column shear force capacities is:

\[ V_{pc,c} = \frac{A_v f_{y} d}{s} = 56.6 \times 2 \times 318 \times 260/230 = 41 \text{ kN} \quad \text{for Units 1 and 2} \]

\[ V_{pc,c} = \frac{A_v f_{y} d}{s} = 56.6 \times 318 \times 420/305 = 25 \text{ kN} \quad \text{for Units EJ1 to EJ4} \]

Column shear force strength is the sum of the contribution of concrete to the shear strength and the contribution of column transverse reinforcement to the shear strength.

6.2 Method Proposed by Park in Reference P6

Detailed description of the method proposed by Park for estimating the shear force strength of members and beam-column joints can be seen in Chapter 2. For the members where plastic hinges were expected, the member shear strength will be estimated by taking into account of the imposed member curvature ductility. Generally, the method proposed by Park in Reference P6 will give less conservative estimations of the shear force capacity of the members and beam-column joints in existing reinforced concrete structures.

(1). Beam Shear Force Capacity

The beam shear capacities are estimated as follows, according to the method proposed by Park [P6]:

\[ V_{pb} = V_{b,c} + V_{b,s} \]

(1.26)

in which: \( V_{b,c} = k \sqrt{f_c' \ b_w \ d} \), \( V_{b,s} = A_v f_{y} d / s \)

For the beams of Unit 1 where plastic hinges were not expected, \( k \) is taken as 0.2 (see Chapter 2). For the beams of Unit 2 and Units EJ1 through EJ4 where plastic hinges were expected, \( k \) is found according to the imposed ductility factor of the members (see Table 1.12 for all units).
Using the values of parameters summarised in Table 1.1(a) and Table 1.1(b) and the coefficient \( k \) in Table 1.12, the beam shear force capacity is estimated for all the units (see Table 1.12).

**Table 1.12** Coefficient \( k \) for Estimating Beam Shear Force Capacity

<table>
<thead>
<tr>
<th>Units</th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit EJ1</th>
<th>Unit EJ2</th>
<th>Unit EJ3</th>
<th>Unit EJ4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f'_e ) (MPa)</td>
<td>43.8</td>
<td>48.9</td>
<td>34</td>
<td>29.2</td>
<td>34</td>
<td>36.5</td>
</tr>
<tr>
<td>( k )</td>
<td>0.2</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
<td>0.05</td>
</tr>
<tr>
<td>( V_{pb} ) (kN)</td>
<td>204</td>
<td>70</td>
<td>62</td>
<td>59</td>
<td>62</td>
<td>63</td>
</tr>
</tbody>
</table>

(2). **Column Shear Force Capacity**

Using the proposed method for estimating the column shear force capacities by Priestley, rather than the method by Park, the column shear force capacities are estimated as follows (see Chapter 2):

**Table 1.13** Estimation of Column Shear Force Capacity Using Method in P6

<table>
<thead>
<tr>
<th>Units</th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit EJ1</th>
<th>Unit EJ2</th>
<th>Unit EJ3</th>
<th>Unit EJ4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f'_e ) (MPa)</td>
<td>43.8</td>
<td>48.9</td>
<td>34</td>
<td>29.2</td>
<td>34</td>
<td>36.5</td>
</tr>
<tr>
<td>( N^* ) (kN)</td>
<td>0.0</td>
<td>800</td>
<td>0.0</td>
<td>0.0</td>
<td>1800</td>
<td>1800</td>
</tr>
<tr>
<td>a (mm)</td>
<td>28</td>
<td>47</td>
<td>28</td>
<td>31</td>
<td>127</td>
<td>119</td>
</tr>
<tr>
<td>tan( \alpha )</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>0.104</td>
<td>0.107</td>
</tr>
<tr>
<td>( k )</td>
<td>0.1</td>
<td>0.29</td>
<td>0.29</td>
<td>0.29</td>
<td>0.29</td>
<td>0.29</td>
</tr>
<tr>
<td>( V_{c,e} ) (kN)</td>
<td>73</td>
<td>224</td>
<td>286</td>
<td>265</td>
<td>286.2</td>
<td>296.9</td>
</tr>
<tr>
<td>( V_n ) (kN)</td>
<td>0</td>
<td>72</td>
<td>0</td>
<td>0</td>
<td>187.2</td>
<td>192.6</td>
</tr>
<tr>
<td>( V_{c,a} ) (kN)</td>
<td>61</td>
<td>61</td>
<td>38.8</td>
<td>38.8</td>
<td>38.8</td>
<td>38.8</td>
</tr>
<tr>
<td>( V_{pe} ) (kN)</td>
<td>34</td>
<td>358</td>
<td>325</td>
<td>304</td>
<td>512</td>
<td>528</td>
</tr>
</tbody>
</table>

\[
V_{pe} = V_{c,e} + V_{c,a} + V_n
\]  
\( (1.27) \)

in which,  
\[
V_{c,e} = v_e \cdot A_g = k \sqrt{f'_e} \cdot 0.8 \cdot A_g
\]  
\( (1.28) \)
\[ V_{c,s} = \frac{A_s f_y d'}{s} \left( \cot 30^\circ \right) \] \hfill (1.29)

\[ V_n = N^* \tan \alpha \] \hfill (1.30)

\[ \tan \alpha = \frac{(h_c - a)}{l_c} \]

where: \( a \) is the equivalent depth of the rectangular compressive concrete block at ultimate state.

Detailed calculation of column shear force capacity is listed in Table 1.13 for all the units, based on the expected curvature ductility imposed as listed in Table 1.8.

(3). **Horizontal Shear Force Capacity of Beam-Column Joints**

The maximum horizontal shear capacity of beam-column joints is calculated using only the current seismic assessment procedures proposed by Park. NZS3101: 1995 gives no indication for estimating the shear force capacities of existing beam-column joint cores.

For both interior and exterior beam-column joints, the probable horizontal shear force capacity is obtained by the following equation [P6].

\[ V_{ph} = V_c b_j h_j + V_{p,h,s} \] \hfill (1.31)

where: \( V_c = \frac{k}{f'c} \sqrt{\frac{1 + \frac{N^*}{A_s f'_c}}{A_s f'_c}} \) and \( k \) is the coefficient associated with the imposed ductility factor, \( b_j \) and \( h_j \) are the effective joint width and depth respectively, and they are determined based on NZS3101: 1995.

According to NZS3101:1995, \( h_j \) is taken as \( h_c \), which is the overall depth of column in the direction of the horizontal joint shear to be considered, \( b_j \) is taken as:

I. where \( b_c > b_w \): either \( b_j = b_w \) or \( b_j = b_c + 0.5 h_c \), whichever is the smaller;

II. where \( b_c < b_w \): either \( b_j = b_w \) or \( b_j = b_c + 0.5 h_c \), whichever is the smaller.

As a result, \( b_j = 450 \) mm and \( h_j = 300 \) mm for two interior beam-column joints, but \( b_j = 460 \) mm and \( h_j = 460 \) mm for the four exterior beam-column joint units.

\[ V_{p,h,s} = \text{contribution of horizontal joint shear reinforcement, and it is zero for two interior beam-column joints and } V_{p,h,s} = 56.6 \times 318 = 18 \text{ kN for the four exterior beam-column joint units. Detailed calculation is seen in Table 1.14.} \]
Table 1.14. Estimated Horizontal Joint Shear Capacity

<table>
<thead>
<tr>
<th>Unit</th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit EJ1</th>
<th>Unit EJ2</th>
<th>Unit EJ3</th>
<th>Unit EJ4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N^*$ (kN)</td>
<td>0.0</td>
<td>800.0</td>
<td>0.0</td>
<td>0.0</td>
<td>1800.0</td>
<td>1800.0</td>
</tr>
<tr>
<td>$f'_c$ (MPa)</td>
<td>43.8</td>
<td>48.9</td>
<td>34</td>
<td>29.2</td>
<td>34</td>
<td>36.5</td>
</tr>
<tr>
<td>$k$</td>
<td>0.3</td>
<td>0.3</td>
<td>0.1</td>
<td>0.3</td>
<td>0.1</td>
<td>0.3</td>
</tr>
<tr>
<td>$\nu_c$</td>
<td>1.99</td>
<td>4.07</td>
<td>0.583</td>
<td>1.621</td>
<td>2.302</td>
<td>4.325</td>
</tr>
<tr>
<td>$V_{pb}$ (kN)</td>
<td>268</td>
<td>550</td>
<td>141</td>
<td>361</td>
<td>505</td>
<td>933</td>
</tr>
</tbody>
</table>

The estimated shear force capacity of beams, columns and beam-column joints, for all the units, is listed in Table 1.15. The investigation of the amount of transverse reinforcement for resisting the shear force is seen in Chapter 4.

Table 1.15 Shear Force Capacity of Beams, Columns and Beam-Column Joints (kN)

<table>
<thead>
<tr>
<th>Part of Units</th>
<th>Unit 1</th>
<th>Unit 2</th>
<th>Unit EJ1</th>
<th>Unit EJ2</th>
<th>Unit EJ3</th>
<th>Unit EJ4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beams, $V_{pb}$ (kN)</td>
<td>146</td>
<td>22</td>
<td>22</td>
<td>22</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td></td>
<td>(204)</td>
<td>(70)</td>
<td>(62)</td>
<td>(59)</td>
<td>(62)</td>
<td>(63)</td>
</tr>
<tr>
<td>Columns, $V_{pc}$ (kN)</td>
<td>41</td>
<td>250</td>
<td>157</td>
<td>147</td>
<td>255</td>
<td>257</td>
</tr>
<tr>
<td></td>
<td>(134)</td>
<td>(358)</td>
<td>(325)</td>
<td>(304)</td>
<td>(512)</td>
<td>(528)</td>
</tr>
<tr>
<td>Beam-Column Joints, $V_{pb}$ (kN)</td>
<td>(268)</td>
<td>(550)</td>
<td>(141)</td>
<td>(361)</td>
<td>(505)</td>
<td>(933)</td>
</tr>
</tbody>
</table>

Note: Values without bracket are given by NZS3101: 1995, and the values with brackets are given by the method proposed by Park [P6].

7. Requirement of Transverse Reinforcement Quantities for Anti-buckling

For all the tests, the axial load ratios on the columns are low. In this case ($N^* < 0.3 \, A_g \, f'_c$), the transverse reinforcement is more required for preventing buckling of longitudinal bars than that for confining the compressed concrete. Hence, apart from the investigation of the amount of transverse reinforcement according to the shear requirement as conducted before, the amount of transverse reinforcement is also investigated according to the requirement for preventing bar buckling using NZS3101: 1995. The procedure proposed in Reference P6 does not have a method in this regard.

7.1 Beams

- Code Specification on Spacing Limit of Beam Transverse Reinforcement
According to NZS3101: 1995, centre-to-centre spacing of stirrups or ties along the beam members shall not exceed the smaller of the least lateral dimension of the cross section or 16 times longitudinal bar diameter; centre-to-centre spacing of stirrups or ties in potential plastic hinge regions shall not exceed either d/4 or 6 times the diameter of any longitudinal compression bar to be restrained in the outer layers.

For Unit 1 where beams were not expected to form plastic hinges, \( b_w = 300 \text{ mm} < 16d_b = 384 \text{ mm} \). Hence, \( s = 300 \text{ mm governs} \).

For Unit 2 and Unit EJ1 through EJ4, beams were expected to develop plastic hinges, \( d/4 = 460/4 = 115 \text{ mm} < 6d_b = 144 \text{ mm} \). Hence, \( s = 115 \text{ mm governs} \).

- **Code Specification on Size Limit of Beam Transverse Reinforcement**

According to NZS3101: 1995, the diameter of the stirrup-ties in beams shall not be less than 5 mm. In addition, the area of one leg of a stirrup-tie placed in potential plastic hinge regions in the direction of potential buckling of the longitudinal bar shall not be less than:

\[
A_{se} = \frac{\sum A_b f_y}{96 f_{yr}} \frac{s}{d_b} 
\]

(1.32)

where \( \sum A_b \) is the sum of the area of the longitudinal bars reliant on the tie.

For Unit 1 where beams were not expected to form plastic hinges, \( A_{se} = \text{Area of D5} = 19.6 (\text{mm}^2) \). **Area of per set shall not be less than 40 mm}^2 \text{ for Unit 1}**.

For Unit 2 and Units EJ1 through EJ4, beams were expected to develop plastic hinges, the limit on area of one leg of a stirrup-tie shall be calculated by Equation 1.32.

\[
A_{se} = \frac{\sum A_b f_y}{96 f_{yr}} \frac{s}{d_b} \\
= \frac{2D24 \times 321}{96 \times 318} \times \frac{115}{24} = 45.5 (\text{mm}^2) 
\]

for Unit 2

\[
= \frac{1.5D24 \times 321}{96 \times 318} \times \frac{115}{24} = 34.2 (\text{mm}^2) 
\]

for Units EJ1 to EJ4

**Area of per set shall not be less than 91 mm}^2 \text{ for Unit 2 and 68 mm}^2 \text{ for Units EJ1 to EJ4}**.

7.2 **Columns**

- **Code Specification on Spacing Limit of Column Transverse Reinforcement**

According to NZS3101: 1995, centre-to-centre spacing of stirrups or ties along the column members shall not exceed the smaller of 1/3 of the least lateral dimension of the cross section or 10 times longitudinal bar diameter; centre-to-centre spacing of stirrups or ties in potential plastic hinge regions shall not exceed either 1/4 of the least lateral dimension of the cross section or 6 times the diameter of any longitudinal compression bar to be restrained.
For Unit 1 where columns were expected to develop plastic hinges, \( \frac{b_c}{4} = 75 \text{ mm} < 6d_b = 144 \text{ mm} \). Hence, \( s = 75 \text{ mm} \) governs.

For Unit 2 and Unit EJ1 through EJ4, columns were not expected to develop plastic hinges, \( \frac{b_c}{3} = 100 \text{ mm} < 10d_b = 240 \text{ mm}, \text{ so } s = 100 \text{ mm} \) governs \( \text{ for Unit 2} \)

\( \frac{b_c}{3} = 153 \text{ mm} < 10d_b = 240 \text{ mm}, \text{ so } s = 153 \text{ mm} \) governs \( \text{ for Unit EJ1 to EJ4} \)

- **Code Specification on Size Limit of Column Transverse Reinforcement**

According to NZS3101: 1995, the diameter of the stirrup-ties in columns shall not be less than 10 mm for the column longitudinal bars with diameter 20 mm to 32 mm. The area of one leg of a stirrup-tie, when governed by the requirement for anti-buckling, shall not be less than:

\[
A_{tu} = \sum A_b f_y \frac{s}{135 f_{yr}} \frac{s}{d_b} \tag{1.33}
\]

In potential plastic hinge regions of columns, the area of one leg of a stirrup-tie, when governed by the requirement for anti-buckling, shall not be less than:

\[
A_{ts} = \sum A_b f_y \frac{s}{96 f_{yr}} \frac{s}{d_b} \tag{1.32}
\]

where \( \sum A_b \) is the sum of the area of the longitudinal bars reliant on the tie.

For Unit 1 where columns were expected to develop plastic hinges, the limit on area of one leg of a stirrup-tie is calculated using Equation 1.32.

\[
A_{te} = \sum A_b f_y \frac{s}{96 f_{yr}} \frac{s}{d_b} = \frac{D24 \times 321}{96 \times 318} \times \frac{75}{24} = 14.9 (\text{mm}^2) \text{ for Unit 1}
\]

For Unit 2 and Unit EJ1 through EJ4, columns were not expected to develop plastic hinges, the limit on area of one leg of a stirrup-tie shall be calculated by Equation 1.33.

\[
A_{tu} = \sum A_b f_y \frac{s}{135 f_{yr}} \frac{s}{d_b} = \frac{D24 \times 321}{135 \times 318} \times \frac{100}{24} = 14.1 (\text{mm}^2) \text{ for Unit 2}
\]

\[
= \frac{D24 \times 321}{135 \times 318} \times \frac{153}{24} = 21.5 (\text{mm}^2) \text{ for Units EJ1 to EJ4}
\]

In this case, area of per set shall not be less than 60 mm\(^2\) for Unit 1, and shall not be less than 57 mm\(^2\) for Unit 2, and shall not be less than 43 mm\(^2\) for Units EJ1 to EJ4.

**7.3 Beam-Column Joints**

NZS3101: 1995 also has specification to limit the spacing and size of column transverse reinforcement within beam-column joints.
According to Clause 11.4.4.5 of NZS3101: 1995, the spacing of sets of column ties or hoops within a joint shall not exceed 10 times the column bar diameter or 200 mm, whichever is less.

\[ 10d_b = 10 \times 24 = 240 \text{ mm} \]

Hence, so \( s = 200 \text{ mm} \) governs.

- Area Limit

According to Clause 11.4.4.5 of NZS3101: 1995, the quantities of horizontal joint reinforcement shall conform to that required by Eq. 1.34.

The area of one leg of horizontal joint reinforcement shall not be less than:

\[ A_{te} = \frac{\sum A_p f_y}{96 f_{yr} d_b} \cdot s = \frac{452 \times 321}{96 \times 318} \cdot \frac{200}{24} = 39.6 \text{ (mm}^2) \]

Hence, area of per set of horizontal transverse reinforcement within the joints shall not be less than 79 mm\(^2\).

Summary of the results obtained from this theoretical consideration is seen in Chapter 4 of the thesis.
8. Development of the Longitudinal Reinforcement within Joints

NZS3101: 1995 has the specifications on the maximum diameter of beam bars passing through the joints. Note that NZS3101: 1995 specifies the use of deformed longitudinal reinforcement.

According to NZS3101: 1995, the maximum diameter of beam bars passing through the interior joints should satisfy the following requirement by equation 1.34:

\[
\frac{d_b}{h_c} \leq 3.3 \alpha_f \frac{\sqrt{f'_c}}{\alpha_o f_y}
\]  

(1.34)

where, \( \alpha_f = 1.0 \) for one-way frames and \( \alpha_o \) is 1.0 when the plastic hinges are not expected in the beams, and \( \alpha_o = 1.25 \) when plastic hinges are expected to develop at column faces.

This gives \( \frac{d_b}{h_c} \leq 14.7 \) for Unit 1 and \( \frac{d_b}{h_c} \leq 17.4 \) for Unit 2.

NZS3101: 1995 also has the specifications on the maximum diameter of deformed column bars passing through the joints. According to NZS3101: 1995, when columns are designed to develop plastic hinges in the end regions, equation 1.34 needs to be satisfied; but when columns are not intended to develop plastic hinges in the end regions, the maximum diameter of column bars may exceed that given by equation 1.34 by 25%.

\[
\frac{d_b}{h_b} \leq 3.2 \frac{\sqrt{f'_c}}{f_y}
\]  

(1.35)

For Unit 1, columns are expected to develop plastic hinges, \( \frac{d_b}{h_b} \leq 15.1 \), and for Unit 2 and Units EJ1 to EJ4, columns are not expected to develop plastic hinges, \( \frac{d_b}{h_b} \leq 11.5, 13.8, 14.9, 13.8, 13.3 \) respectively.