APPENDIX A

DESIGN AND CONSTRUCTION OF THE TEST BUILDING

Within this appendix further details are provided for the design and construction of the two-storey test building, which was introduced in Chapter 4. Design lateral forces for a (full-scale) prototype building are determined. These lateral forces are scaled according to similitude criteria for the two-third scale test building. The section sizes and reinforcement requirements for test building are determined using the scaled lateral forces. Subsequently, the actual strength, rather than the design strength, of the test building is predicted with and without additional reinforcement. The construction process for the test building is documented and structural drawings plus specifications for the test building are provided. Details of the materials used in the test building are provided.

A.1. LATERAL FORCE DESIGN OF PROTOTYPE

The prototype building is designed using a displacement-based design (DBD) procedure (Priestley et al. 2007) with design parameters that are defined in Chapter 4. Firstly, the frame system is idealized as an equivalent single-degree-of-freedom (SDOF):

![SDOF representation of prototype building](image)
Step 1: Determine the design displacement $\Delta_d$, the effective mass $m_e$ and effective height $H_e$

The peak design displacement for the SDOF representation:

$$\Delta_d = \frac{\sum_{i=1}^{n} (m_i \Delta_i^2)}{\sum_{i=1}^{n} (m_i \Delta_i)}$$

Where:

$$\Delta_i = \delta_i \frac{\Delta_i}{\delta_i}$$

And:

$$\frac{\delta_i}{\delta_1} = \frac{H_i}{H_n} = \text{the mode shape (linear displacement profile)}$$

So:

$$\Delta_i = \frac{H_i \Delta_i}{H_n}$$

And:

$$\Delta_i = H_i \cdot \theta_d = 3.0 \times 0.02 = 0.06m$$

The effective mass:

$$m_e = \frac{\sum_{i=1}^{n} (m_i \Delta_i)}{\Delta_d}$$

The effective height:

$$H_e = \frac{\sum_{i=1}^{n} (m_i \Delta_i H_i)}{\sum_{i=1}^{n} (m_i \Delta_i)}$$

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Height, Hi (m)</th>
<th>Weight, wi (KN)</th>
<th>Mass, mi (tonnes)</th>
<th>$\Delta_i$ (m)</th>
<th>$m_i \Delta_i$</th>
<th>$m_i \Delta_i^2$</th>
<th>$m_i \Delta_i^*H_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>6</td>
<td>823</td>
<td>83.9</td>
<td>0.120</td>
<td>10.1</td>
<td>1.21</td>
<td>60.4</td>
</tr>
<tr>
<td>1</td>
<td>3</td>
<td>823</td>
<td>83.9</td>
<td>0.060</td>
<td>5.0</td>
<td>0.30</td>
<td>15.1</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>15.10</td>
<td>1.51</td>
<td>75.5</td>
</tr>
</tbody>
</table>

Therefore:

$$\Delta_d = \frac{\sum_{i=1}^{n} (m_i \Delta_i^2)}{\sum_{i=1}^{n} (m_i \Delta_i)} = \frac{1.51}{15.1} = 0.1m$$

$$m_e = \frac{\sum_{i=1}^{n} (m_i \Delta_i)}{\Delta_d} = \frac{15.1}{0.1} = 151\text{tonne} \ (90\% \text{of \ the \ total \ mass)}$$
\[ H_e = \sum_{i=1}^{n} \left( m_i \Delta H_i \right) / \sum_{i=1}^{n} (m_i \Delta_i) = \frac{75.5}{15.1} = 5.0m \ (83\% \text{ of the total height}) \]

**Step 2: Determine the system damping**

An elastic damping, \( \xi_{el} \), of 5% of critical damping is assumed, as considered for equivalent static (force-based) design according to NZS1170.5 (2004). It is assumed that the elastic damping remains constant at all displacements and is not reduced in proportion to the tangent stiffness of the system as proposed by Priestley *et al.* (2007). Because the building is designed with post-tensioned connection, there is effectively no hysteretic damping for the system. This may be conservative if energy-dissipating reinforcement is provided at the base of the columns.

\[ \xi_{eq} = \kappa \xi_{el} + \xi_{hyst} = 5\% + 0 = 5\% \]

**Step 3: Determine the effective period from the design displacement spectrum**

![Design acceleration and displacement spectrum](image)

By entering the displacement spectrum with the design displacement the effective period is obtained: \( T_e = 0.94s \).

**Step 4: Obtain the equivalent lateral stiffness**

\[ K_e = \frac{4\pi^2 m_e}{T_e^2} = 4\pi^2 \left( \frac{151}{0.94^2} \right) = 6750kN/m \]

Note; this is a secant elastic stiffness to the peak displacement response.
Step 5: Determine the base shear

\[ V_b = K_d \Delta_d = 675 \times 0.1 = 675 \text{kN} \]

Step 6: Distribute the base shear up the structure

For a two-storey structure is reasonable to distribute the forces according to first mode response. Hence, additional forces at the roof level are not required to account for higher modes of response:

\[ F_i = \frac{V_b (m_i \Delta_i)}{\sum_{i=1}^{n} (m_i \Delta_i)} \]

A.2. DETERMINATION OF THE FRAME ACTIONS

The lateral loads for the test model are scaled from the prototype design according to similitude requirements (see Chapter 4). The lateral forces are distributed evenly to each frame, ignoring any accidental eccentricity (NZS1170.5 2004).

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Floor Force, F_i (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>100</td>
</tr>
<tr>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>Sum (Vb)</td>
<td>150</td>
</tr>
</tbody>
</table>

An equilibrium-based approach (Priestley et al. 2007), apposed to member stiffness, is used to distribute strength throughout the frame.

Step 1: Determine the total overturning moment (OTM)

\[ OTM = \sum_{i=1}^{n} F_i H_i \]
**Table A.2.2 – Calculation of OTM**

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Hi (m)</th>
<th>Fi (kN)</th>
<th>Fi.Hi (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>4.0</td>
<td>100</td>
<td>400</td>
</tr>
<tr>
<td>1</td>
<td>2.0</td>
<td>50</td>
<td>100</td>
</tr>
<tr>
<td><strong>Sum</strong></td>
<td></td>
<td><strong>Vb=150</strong></td>
<td><strong>OTM=500</strong></td>
</tr>
</tbody>
</table>

**Step 2:** Decide how much of the OTM will be taken by the column bases:

\[ OTM = \sum_{j=1}^{n} M_{c,j} + TL_{base} \]

Where: \( \sum_{j=1}^{n} M_{c,j} \) = the sum all column-base moments

\( T \) = the tension force induced by lateral load in the exterior column

\( L_{base} \) = the total frame length (to column centrelines)

Hence, the sum of the column base moments must be decided. As recommended by Priestley *et al.* (2007), a reasonable approach is to ensure that the point of contra-flexure in the column is at 60% of the interstorey height at the bottom level. Hence:

\[ \sum_{j=1}^{n} M_{c,j} = 0.6V_bH_1 = 0.6 \times 150 \times 2.0 = 180kN.m \]

Therefore, the column-base moments are 36% of the total overturning moment.

**Step 3:** Determine the remaining tension and compression force in the exterior columns:

\[ T = C = \frac{OTM - \sum_{j=1}^{n} M_{c,j}}{L_{base}} = \frac{500 - 180}{8.38} = 38.2kN \]

**Step 4:** Proportion the seismic axial forces to each beam up the height of the building

A rational way to proportion the seismic axial forces into beam shears is to use the total shear force diagram (Priestley *et al.* 2007). This will ensure that an effectively linear displacement profile is maintained.
Hence:

\[ V_{B,i} = T \frac{V_{S,i}}{\sum_{i=1}^{n} V_{S,i}} \]

Where: \( V_{B,i} \) = the beam shear at the \( i^{th} \) floor

### Table A.2.3 – Calculation of beam shears

<table>
<thead>
<tr>
<th>Storey, ( i )</th>
<th>( F_i ) (kN)</th>
<th>( V_{si} ) (kN)</th>
<th>( V_{Bi} ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>100</td>
<td>100</td>
<td>15.3</td>
</tr>
<tr>
<td>1</td>
<td>50</td>
<td>150</td>
<td>22.9</td>
</tr>
<tr>
<td><strong>Sum</strong></td>
<td></td>
<td>( \sum V_{si} = 250 )</td>
<td>( \sum V_{Bi} = 38.2 )</td>
</tr>
</tbody>
</table>

### Step 5: Calculate beam design moments

\[ M_{B,i} = V_{B,i} \frac{L_b}{2} \] (At the column centerlines)

Where \( L_b \) = the length of the bay from column centerline to column centerline (4.19m)

The beam-column connection design moments at the column face are:

\[ M_{cf,i} = M_{B,i} \frac{L_b - h_c}{L_b} \]

Where \( h_c \) = the column width (400mm assumed)

### Table A.2.4 – Calculation of beam moments

<table>
<thead>
<tr>
<th>Storey, ( i )</th>
<th>( V_{Bi} ) (kN)</th>
<th>( MB_i ) (kN.m)</th>
<th>( Mcfi ) (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>15.3</td>
<td>32.1</td>
<td>29.0</td>
</tr>
<tr>
<td>1</td>
<td>22.9</td>
<td>48.0</td>
<td>43.4</td>
</tr>
</tbody>
</table>

### Step 6: Calculate column design moments

The sum of the column moments above and below a given floor must equal the sum of the beam moments.

\[ \sum M_{c,i,above} + \sum M_{c,i,below} = \sum M_{B,i} \]
Appendix A – Design and construction of the test building

It is reasonable to assume that the column moments, immediately above and below the \(i^{th}\) floor, are equal. Making this assumption, the following equations are derived:

For the 1\(^{st}\) storey:

\[
\sum_{j=1}^{n} M_{C,j} = n_b M_{B,i}
\]

For the 2\(^{nd}\) storey (the roof):

\[
\sum_{j=1}^{n} M_{C,j} = 2n_b M_{B,i}
\]

Where \(n_b\) = the number of bays

<table>
<thead>
<tr>
<th>Table A.2.5 – Calculation of total column moments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storey, (i)</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>1</td>
</tr>
</tbody>
</table>

Hence, the worst-case column moment is on the top floor. By equilibrium, the moments induced in the interior and exterior columns can be determined:

<table>
<thead>
<tr>
<th>Table A.2.6 – Calculation of interior and exterior column moments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storey, (i)</td>
</tr>
<tr>
<td>----------------</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>1</td>
</tr>
</tbody>
</table>

A.3. DETAILED DESIGN OF TEST BUILDING

The test building is designed considering purely post-tensioned frame and wall connections. The design takes into strength reduction factors and over-strength design.

A.3.1. Frame design

The frames were designed using existing connection modeling procedures (Newcombe et al. 2008) and improved methodologies, discussed in Chapter 6, to determine the member deformations.
Beam-column connections on Level 2

The actions on an internal beam-column joint are considered to estimate the member deformations.

\[ \theta_D = \theta_b + \theta_c + \theta_f + \theta_{con} \]

The deflection of the frame can be broken into its components:
Appendix A – Design and construction of the test building

Step 1: Calculate the beam and column deformation

If it is assumed that the beam and column sections are solid, that the points of contra-flexure for the beams and columns are at half the bay length and half the interstorey height respectively, the following expressions are appropriate:

\[
\theta_b = \frac{\phi_b}{L_b} \left( \frac{(L_b - h_c)^2}{6} + \frac{E_t h_b^2}{G} \frac{H}{4} \right) \quad \text{And} \quad \theta_c = \frac{\phi_c}{H} \left( \frac{(H - h_c)^2}{6} + \frac{E_t h_c^2}{G} \frac{L}{4} \right)
\]

Where: \( E_t \) and \( G \) are the bending and shear elastic modulus respectively

\( h_b = \) the depth of the beam

\( h_c = \) the depth of the column.

The bending and shear modulus are 10500 MPa and 660 MPa respectively (see section A.8). Furthermore, it is assumed that the beam and column have a width (\( b_c \) and \( b_b \)) of 240mm and depth (\( h_c \) and \( h_b \)) of 400mm. These dimensions will be used at both levels.

The curvature in the beam and column is:

\[
\phi_b = \frac{M_{con}}{E_t I_b} \quad \text{And} \quad \phi_c = \frac{M_{con}}{E_t I_c} \left( \frac{L - h_c}{H} \right)
\]

Where: \( I_b \) and \( I_c \) are the second moment of area of the beam and column respectively.

\[
\phi_b = \frac{43.4 \times 10^6}{10500 \times \frac{240 \times 400^3}{12}} = 3.23 \times 10^{-6} \frac{1}{\text{mm}}
\]

\[
\phi_c = \frac{43.4 \times 10^6}{10500 \times \frac{240 \times 400^3}{12}} \left[ \frac{4191(2000-400)}{2000(4191-400)} \right] = 2.86 \times 10^{-6} \frac{1}{\text{mm}}
\]

Hence:

\[
\theta_b = \frac{3.23 \times 10^{-6}}{4191} \left( \frac{(4191-400)^2}{6} + \frac{10500 \times 400^2}{660 \times 4} \right) = 0.0023
\]

\[
\theta_c = \frac{2.86 \times 10^{-6}}{2000} \left( \frac{(2000-400)^2}{6} + \frac{10500 \times 400^2}{660 \times 4} \right) = 0.0015
\]
**Step 2:** Calculate the joint panel zone deformation

The joint panel deformation can be significant for unreinforced columns. However, on Level 2 extensive steel reinforcement is provided by an internal steel plate arrangement (see Chapter 4). Therefore, it is assumed that joint panel zone deformation can be ignored:

\[ \theta_j = 0 \]

**Step 3:** Determine the allowable connection rotation

To minimize the required post-tensioning reinforcement the maximum allowable connection rotation should be used for design of the post-tensioning.

\[ \theta_{con} = \theta_d - (\theta_b + \theta_c + \theta_j) = 0.020 - (0.0023 + 0.0015 + 0) = 0.020 - 0.0038 = 0.0162 \]

The actual imposed connection rotation at the connection, \( \theta_{imp} \), is slightly higher than the interstorey rotation due to the connection, \( \theta_{con} \):

\[ \theta_{imp} = \frac{\theta_{con}}{1 - \frac{h_c}{L_b}} = \frac{0.0162}{1 - \frac{400}{4191}} = 0.018 \]

**Step 4:** Design beam-column connections

The connection moment capacity is calculated at the allowable imposed connection rotation. Sufficient post-tensioning force must be applied so that the beam-column connections have sufficient moment capacity, within the allowable connection rotation. If the moment capacity of the connection is more than required (due to strength reduction factors and over design) the actual connection rotation, and hence, the total frame deformation, will be less than considered for design. Existing procedures (Newcombe et al. 2008) are used to determine the connection moment capacity.

**Impose connection rotation**

The allowable connection rotation \( \theta_{imp} \) is applied to the connection. A preliminary estimate of the post-tensioning is made: 4-0.5 inch (12.7mm) post-
tensioning tendons symmetrically spaced at 60mm centre-to-centre. The tendons are stressed to 60% of their yield stress.

Figure A.3.3 – Rocking post-tensioned connection

Estimate the neutral axis depth, c

Say:

\[ c = 140 \text{mm} \]

Apply member compatibility:

The elongation of the tendon:

\[
\Delta_{pt} = \theta_{imp} (y_i - c) = 0.018 \times \begin{bmatrix}
290 - 140 \\ 230 - 140 \\ 170 - 140 \\ 110 - 140
\end{bmatrix} = \begin{bmatrix}
2.70 \\ 1.62 \\ 0.54 \\ -0.54
\end{bmatrix} \text{mm}
\]

Where: \( y_i \) = the distance to the centroid of the tendon from the base of the beam

According to Newcombe (2008) tendon shortening can be ignored. The strains due to gap openings are:

\[
\Delta \varepsilon_{pt} = \sum \frac{n \Delta_{pt}}{l_{ub}} = \frac{2}{9280} \begin{bmatrix}
2.70 \\ 1.62 \\ 0.54 \\ 0
\end{bmatrix} + \frac{2}{9280} \begin{bmatrix}
0 \\ 0.54 \\ 1.62 \\ 2.70
\end{bmatrix} = \begin{bmatrix}
0.00058 \\ 0.00047 \\ 0.00047 \\ 0.00058
\end{bmatrix}
\]

Where: \( n \) = the number of connections subject to the same rotation
\( l_{ub} \) = the unbonded length of the tendon
The stress in the tendon is checked at the design rotation. The stress in the tendon should be less than 90% of the yield stress:

\[ \varepsilon_{pt} = \varepsilon_{pt,j} + \Delta \varepsilon_{pt} \leq 0.9 \varepsilon_y \]

Where: \( \varepsilon_{pt,j} = \frac{T_{pt}}{A_{pt}E_{pt}} \) and,

\[
T_{pt} = 0.7 f_y A_{pt} \\
A_{pt} = 396 \text{mm}^2 = \text{the area of a tendons} \\
E_{pt} = 190000 \text{ MPa} = \text{the elastic modulus of the tendons} \\
f_y = 1560 \text{ MPa} = \text{the yield stress of the tendons}
\]

Hence:

\[
T_{pt} = 0.6 f_y A_{pt} = 0.6 \times 1560 \times 396 = 371 \text{kN} \\
\varepsilon_{pt,j} = \frac{371 \times 10^3}{396 \times 190000} = 0.00493
\]

For the top and bottom tendons:

\[
\varepsilon_{pt} = 0.00493 + 0.00058 = 0.0055 \leq 0.0074
\]

The monolithic beam analogy (Palermo et al. 2005b) is applied assuming the timber remains elastic, as proposed by Newcombe (2008):

\[
\varepsilon_t = \left( 3 \frac{\theta_{imp}}{L_{cant}} + \phi_{dec} \right) e
\]

Where: \( \phi_{dec} = \text{the decompression curvature} \)

\( L_{cant} = \text{the shear span} = \frac{(4191 - 400)}{2} = 1896 \text{mm} \)

The decompression curvature is:

\[
\phi_{dec} = \frac{2T_{pt}}{E_p b_h h_b^2} = \frac{2 \times 371 \times 10^3}{10500 \times 240 \times 400^2} = 1.84 \times 10^{-6} \frac{1}{\text{mm}}
\]

Hence:

\[
\varepsilon_t = \left( 3 \frac{0.018}{1896} + 1.84 \times 10^{-6} \right) 140 = 0.0042
\]

To determine the stress in the timber Newcombe et al. (2008) proposes an calibrated effective connection modulus is used. For armored connections,
where the perpendicular to grain timber of the column is protected by steel plates:

\[ E_{con} = 0.55E_i = 0.55 \times 10500 = 5775 \text{MPa} \]

The stress in the timber is checked. Note, since there is steel armoring in the connection the parallel-to-grain yield stress can be considered. Hence:

\[ \varepsilon_y = \frac{f_c}{E_{con}} = \frac{45}{5775} = 0.0078 > 0.0042 \Rightarrow O.K! \text{ (Not yielding)} \]

**Calculate the forces in the connection**

Since the timber is elastic we can assume a linear distribution of the stress within the compression region of the timber, hence:

\[ C_i = 0.5\varepsilon_y E_{con} c_i b = 0.5 \times 0.0042 \times 5775 \times 140 \times 240 / 10^3 = 407 \text{kN} \]

For the post-tensioning:

\[ T_{pt} = T_{pt,i} + \Delta T_{pt} = 0.7 f_y A_{pt} + \sum \varepsilon_{pti} E_{pt} A_{pti} \]
\[ = 371 + (0.00058 + 0.00047) \times 2 \times 99 \times 100000 / 10^3 = 411 \text{kN} \]

**Check force equilibrium**

By equilibrium:

\[ C_i = T_{pt} \]
\[ .407 \approx 411 \Rightarrow OK! \text{ (Within 5%)} \]

**Evaluate the moment capacity**

For the design of the test building a strength reduction factor of 0.9 was considered. This aims to take into account uncertainties in the prediction of the moment capacity of the connection. The moment capacity is:

\[ \phi M_n = \phi T_{pt} \left( \frac{h_n}{2} - \frac{c}{3} \right) = 0.9 \times 411 \left( \frac{400}{2} - \frac{140}{3} \right) = 57 \text{kN.m} \]

\[ M^* \leq \phi M_n \]
\[ 43 \leq 57 \Rightarrow O.K! \]
Therefore, the preliminary section sizes and specified post-tensioning are sufficient. Note, some conservatism is built into the design to allow for losses in tendon force during testing and inaccuracies in the calculation of the section properties (i.e. the beams are box sections rather than solid).

**Beam-column connections on Level 3**

Again, the actions on the internal beam-column joint are considered to estimate the member deformations.

\[ \theta_D = \theta_b + \theta_c + \theta_j + \theta_{con} \]

**Step 1: Calculate the beam and column deformation**

Because is the Level 3 top of the frame, to column deformation can be reduced. It is still reasonable to assume that the point of contra-flexure in the columns is at approximately half the interstorey height.

\[
\theta_b = \phi_b \left( \frac{(L_b - h) \cdot E_i \cdot h^2}{6 \cdot G} \right) \quad \text{and} \quad \theta_c = \phi_c \left( \frac{(H - h) \cdot E_i \cdot h^2}{6 \cdot G} \right)
\]

\[
\phi_b = \frac{29 \times 10^6}{10500 \times \frac{240 \times 400}{12} m} = 2.16 \times 10^{-6} \frac{1}{mm}
\]

\[
\phi_c \approx \frac{2 \times 29 \times 10^6}{10500 \times \frac{240 \times 400}{12} \cdot \frac{4191(2000 - 400)}{2000(4191 - 400)}} = 3.82 \times 10^{-6} \frac{1}{mm}
\]

Hence:

\[
\theta_b = \frac{2.16 \times 10^{-6}}{4191} \left( \frac{(4191 - 400)^2}{6} + \frac{10500 \cdot 400^2}{660} \right) = 0.0016
\]

\[
\theta_c = \frac{3.82 \times 10^{-6}}{2 \times 2000} \left( \frac{(2000 - 400)^2}{6} + \frac{10500 \cdot 400^2}{660} \right) = 0.0010
\]

**Step 2: Calculate the joint panel zone deformation**

On Level 3 light reinforcement is provided by large SPAX wood screws (see Chapter 4). These were provided to reinforcement the column surrounding steel pins, which
anchor additional external reinforcement (see Chapter 6). However, it was not expected that the screws would provide a significant increase to the stiffness of the joint panel region. Hence, they are ignored in the design.

A simplified approach, proposed in Chapter 6, is used for determining this joint panel deformation:

\[
\theta_j = \gamma \left(1 - \frac{h_v}{L_h} - \frac{2h_h}{H}\right)
\]

Where: \(\gamma = \gamma_h + \gamma_v\), \(\gamma_h\) and \(\gamma_v\) are the horizontal and vertical joint distortion respectively.

![Figure A.3.4 – Horizontal and vertical joint distortion](image)

The vertical joint distortion can be ignored. For a rectangular section, the horizontal joint distortion can be approximated as:

\[
\gamma_h = \frac{V_{jh}}{GA_{sh}}
\]

Where: \(V_{jh}\) = the average horizontal shear force within the joint panel region. \(A_{sh} = b_c h_c\) = the horizontal shear area within the joint panel.

And:

\[
V_{jh} = \frac{2M_{con}}{h_b} - V_{col} \quad \text{(See Chapter 6)}
\]

Where \(V_{col} \approx \frac{2M_{c,int}}{H} = \frac{2 \times 64}{2.0} = 64 kN\)
Hence:
\[ \bar{V}_{jh} = \frac{2 \times 29}{0.40} - 64 = 81\text{kN} \]

Therefore:
\[ \gamma = \gamma_h = \frac{81 \times 10^3}{660 \times 400 \times 240} = 0.0013 \]

And:
\[ \theta_j = 0.0013 \left( 1 - \frac{400}{4191} - \frac{400}{2 \times 2000} \right) = 0.0010 \]

**Step 3: Determine the allowable connection rotation**

To minimize the required post-tensioning reinforcement the maximum allowable connection rotation should be used for design of the post-tensioning.

\[ \theta_{\text{con}} = \theta_d - (\theta_b + \theta_c + \theta_j) = 0.020 - (0.0016 + 0.0010 + 0.0010) = 0.020 - (0.0036) = 0.0164 \]

And:
\[ \theta_{\text{imp}} = \frac{\theta_{\text{con}}}{1 - \frac{h_s}{L_h}} = \frac{0.0164}{1 - \frac{400}{4191}} = 0.018 \]

**Step 4: Design beam-column connections**

Existing procedures (Newcombe et al. 2008) are used to determine the connection moment capacity.

**Impose connection rotation**

The allowable connection rotation \( \theta_{\text{imp}} \) is applied to the connection. A preliminary estimate of the post-tensioning is made: 4-0.5 inch (12.7mm) post-tensioning tendons symmetrically spaced at 60mm centre-to-centre, identical to Level 2. The tendons are stressed to 60% of their yield stress.

**Estimate the neutral axis depth, c**

Say:
\[ c = 270\text{mm} \]
Apply member compatibility:
The elongation of the tendons, ignoring shortening (Newcombe 2008):

\[
\Delta_{pl} = \theta_{imp}(y_t - c) = 0.018 \times \begin{bmatrix}
290 - 270 \\
230 - 270 \\
170 - 270 \\
110 - 270
\end{bmatrix}
\begin{bmatrix}
0.36 \\
0 \\
0 \\
0
\end{bmatrix}
\begin{bmatrix}
mm
\end{bmatrix}
\]

\[
\Delta \varepsilon_{pt} = \sum \frac{n \Delta_{pl}}{l_{ub}} = \frac{2}{9280} \begin{bmatrix}
0.36 \\
0 \\
0 \\
0.36
\end{bmatrix}
\begin{bmatrix}
0.00008 \\
0 \\
0 \\
0.00008
\end{bmatrix}
\]

And:
\[
\varepsilon_{pt} = \varepsilon_{pt,i} + \Delta \varepsilon_{pt} \leq 0.9 \varepsilon_y
\]

Where: \( \varepsilon_{pt,i} = \frac{T_{pt}}{A_{pt} E_{pt}} \) and,
\[
T_{pt} = 371 kN \quad \text{(same as Level 2)}
\]

Hence:
\[
\varepsilon_{pt,i} = \frac{371 \times 10^3}{396 \times 190000} = 0.00493
\]

And:
\[
\varepsilon_{pt} = 0.00493 + 0.00008 = 0.0050 \leq 0.0074
\]

The monolithic beam analogy (Palermo et al. 2005b) is applied assuming the timber remains elastic:

\[
\varepsilon_t = \left(3 \frac{\theta_{imp}}{L_{cant}} + \phi_{dec}\right) c
\]

Hence:
\[
\varepsilon_t = \left(3 \frac{0.018}{1896} + 1.84 \times 10^{-6}\right) 270 = 0.0082
\]

Again, to determine the stress in the timber Newcombe et al. (2008) proposes an calibrated effective connection modulus. This aims to account for the interaction of parallel and perpendicular to grain timber of each side of the connection. In Newcombe et al., (2008), the
connection modulus was proposed as a ratio of the parallel to grain elastic modulus. However, there is little correlation between the parallel and perpendicular to grain modulus in timber. Hence, the calibrated numerical value from Newcombe (2008) is used here instead:

\[ E_{\text{con}} = 1400\text{MPa} \]

The stress in the timber is checked. Note, since there is no steel armoring in the connection the perpendicular to grain yield stress should be considered. Hence:

\[ \varepsilon_y = \frac{f_y}{E_{\text{con}}} = \frac{12}{1400} = 0.0086 > 0.0082 \Rightarrow O.K! \text{ (Not yielding)} \]

**Calculate the forces in the connection**

Since the timber is elastic we can assume a linear distribution of the stress within the compression region of the timber, hence:

\[ C_t = 0.5\varepsilon_y E_{\text{con}} c_1 b = 0.5 \times 0.0082 \times 1400 \times 270 \times 240 / 10^3 = 372\text{kN} \]

For the post-tensioning:

\[ T_{pt} = T_{pt,1} + \Delta T_{pt} = 0.7 f_y A_{pt} + \sum\varepsilon_{pt} E_{pt} A_{pt,1} \]

\[ = 371 + 3 = 374\text{kN} \]

**Check force equilibrium**

By equilibrium:

\[ C_t = T_{pt} \]

\[ \therefore 372 \approx 374 \Rightarrow O.K! \text{ (Within 5%)} \]

**Evaluate the moment capacity**

\[ \phi M_n = \phi T_{pt} \left( \frac{h_c}{2} - \frac{c}{3} \right) = 0.9 \times 374 \times \left( \frac{400}{2} - \frac{270}{3} \right) = 37\text{kN.m} \]

\[ M^* \leq \phi M_n \]

\[ 29 \leq 37 \Rightarrow O.K! \]

Therefore, the preliminary section sizes and specified post-tensioning are sufficient.
Column-base connections

The column-base connections are designed in a similar manner as the beam-column connections. Axial loads are induced by gravity loading, and seismic tension/compression on the exterior columns. The sum of moment capacity of the columns must be at least than 0.6$V_b H_1$, according to the assumptions for the frame analysis (section A.2). However, due to laboratory constraints the interstorey height of the first floor is restricted. Hence, the column connections are effectively moved up the height of the column by 300mm (to allow for the steel foundations). Therefore, the actual connection design moment is reduced to:

$$\sum M_{col, actual} = \frac{0.6 \times 2000 - 300}{0.6 \times 2000} \times 0.6V_b H_1 = 0.75(0.6V_b H_1) = 0.75 \times 180 = 135kN\cdot m$$

It is assumed the reduction and increase in moment capacity for the columns subjected to seismic tension and compression forces respectively is similar. Hence, the interior column base connections were designed to have a moment capacity of at least 0.2$V_b H_1$ (reduced by 25%): 45kN.m. For the interior connection the gravity induced axial load is approximately 37kN.

Figure A.3.5 – Column-base external reinforcement
Impose connection rotation

The allowable connection rotation $\theta_{imp}$ is applied to the connection. Here it is assumed that the elastic deformation of the column is negligible. Hence, the full design rotation of 2.0% is applied to the connection. A preliminary estimate of the external reinforcement is made: 2-16mm diameter fused mild steel bars per side, spaced at 220mm (see Figure A.3.5).

Estimate the neutral axis depth, $c$

Say:

$c = 70\text{mm}$

Apply member compatibility:

The elongation of the bars:

$$\Delta_s = \theta_{imp} (y_i - c) = 0.02 \times \left(\frac{310 - 70}{90 - 70}\right) = \left(\frac{4.8}{0.4}\right) \text{mm}$$

Where: $y_i$ = the distance to the centroid of the bars from the edge of the column.

The strains due to gap openings are (Newcombe et al. 2008):

$$\varepsilon_s = \frac{\Delta_s - 2\Delta_{sp}}{l'_{ub}}$$

Where: $\Delta_{sp}$ = the strain penetration or slippage of the anchorages

$l'_{ub} = 165\text{mm}$ = the unbonded (fuse) length of the bars.

It is assumed that the slippage of the anchorage pin and the bolted connections is 0.5mm. Because the deformation of the bars closest to the neutral axis is less than the slippage deformation their contribution can be ignored. Hence:

$$\varepsilon_s = \left(\frac{(4.8 - 0.5)}{165}\right) = \left(\frac{0.0261}{0}\right)$$

The maximum strain of the bar (at the design displacement of the structure) should not exceed 5% strain for mild steel (Marriott 2009).

$\varepsilon_s = 2.5\% \leq 5\%$
The monolithic beam analogy (Palermo et al. 2005b) is applied assuming the timber remains elastic, as proposed by Newcombe (2008):

$$\varepsilon_i = \left(3 \frac{\theta_{imp}}{L_{cant}} + \phi_{dec}\right)$$

Where: $\phi_{dec}$ = the decompression curvature

$L_{cant}$ = the shear span = $0.6H - 300 = 900$mm

The decompression curvature negligible due to low axial forces: $\phi_{dec} \approx 0$

Hence:

$$\varepsilon_i = \left(3 \frac{0.02}{900} + 0\right)70 = 0.0047$$

To determine the stress in the timber Newcombe et al. (2008) proposes an calibrated effective connection modulus is used for column-base connections:

$$E_{con} = 0.55E_i = 0.55 \times 10500 = 5775MPa$$

The stress in the timber is checked. Note, the parallel-to-grain yield stress should be considered. Hence:

$$\varepsilon_y = \frac{f_y}{E_{con}} = \frac{45}{5775} = 0.0078 > 0.0047 \Rightarrow O.K! \text{ (Not yielding)}$$

**Calculate the forces in the connection**

Since the timber is elastic we can assume a linear distribution of the stress within the compression region of the timber, hence:

$$C_i = 0.5 \varepsilon_i E_{con} c_d b = 0.5 \times 0.0047 \times 5775 \times 70 \times 240 / 10^3 = 228kN$$

For the steel bars, an elasto-perfectly-plastic steel relationship is assumed, with a yield stress of 430MPa.

$$T_s = A_s f_y = 2 \times \frac{\pi}{4} 16^2 \times 430 / 10^3 = 173kN$$
Check force equilibrium
By equilibrium:
\[ C_i = T_s + N^* \]
\[ \therefore 228 \approx 173 + 37 = 210 \Rightarrow OK! \text{ (Within 10\%)} \]

Evaluate the moment capacity
\[
\phi M_n = \phi \left[ T_i \left( y_i - \frac{c}{3} \right) + N^* \left( \frac{h}{2} - \frac{c}{3} \right) \right] = 0.9 \left[ 173 \times \left( 310 - \frac{70}{3} \right) + 37 \times \left( 200 - \frac{70}{3} \right) \right]
\]
\[ = 50kN.m \]
\[ M^* \leq \phi M_n \]
\[ 45 \leq 50 \Rightarrow O.K! \]
Therefore, the preliminary section sizes and specified reinforcement are sufficient.

Design pin-anchorage
The pins used to anchor the external reinforcement to the column were designed using Eurocode 5 (EC5 1994) equations for a pure embedment failure. The axial capacity of a pin according Eurocode 5 is:
\[ \phi V_n = 0.9 \times 703 = 632kN \]
The peak demand is derived from the two rods yielding in tension (ignoring overstrength):
\[ V^* \text{pin} = 346kN \]
\[ V^* \leq \phi V_n \]
\[ 346 \leq 632 \Rightarrow O.K! \]

Beam and column section capacity
The critical section capacity check is for the interior column on Level 3, with a moment demand of 64kN.m.

Flexure:
For column design, it has been proposed (Newcombe 2008) that a dynamic amplification factor, \( \omega \), of 1.6 should be considered. In addition, an overstrength factor, \( \phi^0 \), of 1.25 is assumed to take into
account inaccuracies in for the prediction of post-tensioned connection moment capacity and over-design.

The nominal moment capacity is:

\[ \phi M_n = \phi k Z f_b \]

The bending strength, \( f_b \), is 45MPa. For large timber members at size factor, \( k_24 \), should be considered. From NZS 3604, for a 400mm deep section; \( k_24 = 0.95 \). Since the beam is subjected to axial load, the buckling factor, \( k_8 \), is computed; \( k_8 = 1.0 \). No other \( k \)-factors are required for short term earthquake loading. The lowest section modulus, \( Z \), is within the joint panel region, between beams. Here, the section width is reduced to allow the post-tensioning to pass through the column. According to New Zealand Standards, a strength reduction factor need not be considered for overstrength design.

\[ \therefore \phi M_n = 1.0 \times 0.95 \times 45 \times \frac{180 \times 400^2}{6} = 205kN.m \]

\[ \omega \phi^0 M_{ci} \leq \phi M_n \]

\[ 1.6 \times 1.25 \times 64 \leq 205 \]

\[ 128 \leq 205 \Rightarrow O.K! \]

**Shear:**

The maximum shear applied to the column will be within the joint panel region at Level 2. However, this is heavily reinforced with internal steel plates (see Chapter 4). Therefore, the critical shear demand will occur within the joint panel region on Level 3. The peak shear is determined using the previous section analysis:

\[ V^* = C_t - V_{col} = 228 - \frac{64}{1.0} = 164kN \]

It is tentatively recommended that the shear force dynamic amplification factor, \( \omega_v \), for the design of reinforced concrete frames (NZS3101 2006) of 1.3 can be used.
The shear capacity is:

\[ \phi V_n = \phi k f_s A_s \]

Where: \( f_s = 5.3 \text{MPa} \) = the shear strength of the timber

Within the joint panel region, the internal stresses are predominately due to shear; hence, the shear area, \( A_s \), is the full section area:

\[ \therefore \phi V_n = 1.0 \times 1.0 \times 5.3 \times 180 \times 400 / 10^3 = 382kN \]

\[ \omega \phi V^* \leq \phi V_n \]

\[ 1.3 \times 1.25 \times 164 \leq 382 \]

\[ 267 \leq 382 \Rightarrow O.K! \]

Therefore, the section capacity of the beams and columns is sufficient. With the addition of external mild steel reinforcement (see Chapter 4), the moment demand on the sections may increase by a factor of approximately 1.5. Even if this occurs there will be sufficient section capacity.

**A.3.2. Wall design**

The elastic member deformations for timber walls can also be significant (depending on geometry). Again, improved methodologies, discussed in Chapter 7, were used to determine the wall deformation for the test building and existing procedures were used to describe the connection response (Newcombe et al. 2008). It was assumed in the design that the concrete slab or edge beams provide no coupling action to the walls.

**Determination of the wall actions**

The actions (forces, shears and moments) on each wall are assumed to be simply the building actions from the prototype divided by eight and reduced according to similitude. Hence, no eccentricities are considered for proportioning load to each wall.
### Table A.3.1 – Calculation of wall actions

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Fi (kN)</th>
<th>Vsi (kN)</th>
<th>Msi (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>25</td>
<td>25</td>
<td>50</td>
</tr>
<tr>
<td>1</td>
<td>13</td>
<td>38</td>
<td>114</td>
</tr>
<tr>
<td><strong>Sum</strong></td>
<td></td>
<td><strong>V_b=38</strong></td>
<td></td>
</tr>
</tbody>
</table>

**Base connections**

The actions on a wall are considered to estimate the member deformations and determine allowable connection rotation. It is assumed that the wall deformation profile is effectively linear. Hence, the rotation to the top of the wall is essentially the same as to the effective height.

![Figure A.3.6 – Wall actions](image)

The deflection of the wall can be broken into its significant components:

\[
\theta_D = \theta_f + \theta_s + \theta_{con}
\]

![Figure A.3.7 – Deflection components of timber frame](image)
**Step 1:** Calculate flexural deformation

It is assumed that the bending moment diagram is effectively linear.

\[
\Delta_f = \frac{M_y H_{total}^2}{3E_i I_{w}}
\]

Where: \( H_{total} = 3.7\text{m} \) = the height from the wall base to the top floor.

The walls sections are 800×144mm and have a void of 400×50mm. Therefore:

\[
I_{w} = \frac{1}{12}(144 \times 800^3 - 50 \times 400^3) = 5.88 \times 10^9 \text{mm}^4
\]

Hence:

\[
\Delta_f = \frac{114 \times 10^6 \times 3700^2}{3 \times 10500 \times 5.88 \times 10^9} = 8.4\text{mm}
\]

Therefore, the drift due to flexure is approximately:

\[
\theta_f = \frac{\Delta_f}{H_{total}} = \frac{8.4}{3700} = 0.0023
\]

**Step 2:** Calculate shear deformation

Again, it is assumed that shear deformation is approximately linear with height. The shear deformation is the sum of the shear deformation at each floor, \( \Delta_{si} \). Hence:

\[
\Delta_s = \sum \Delta_{si} = \sum \frac{V_{si} H_i}{G A_s}
\]

Where: \( H_i = 1.7\text{m} \) and 2.0m for Level 1 and Level 2 respectively

The shear area is approximately:

\[
A_s = \frac{2}{3}(144 \times 800 - 50 \times 400) = 63.5 \times 10^3 \text{mm}^2
\]

Hence:

\[
\Delta_s = \frac{1}{660 \times 63.5 \times 10^3}(38 \times 10^3 \times 1700 + 25 \times 10^3 \times 2000) = 2.7\text{mm}
\]
Therefore, the drift due to shear is approximately:
\[ \theta_s = \frac{\Delta_s}{H_{total}} = \frac{2.7}{3700} = 0.0007 \]

**Step 3:** Determine the allowable connection rotation

To minimize the required post-tensioning reinforcement the maximum allowable connection rotation should be used for design of the post-tensioning.
\[ \theta_{con} = \theta_f - (\theta_f + \theta_s) = 0.020 - (0.0023 + 0.0007) = 0.020 - 0.0030 = 0.017 \]

The actual imposed connection rotation at the connection, \( \theta_{imp} \), is equal to the interstorey rotation due to the connection, \( \theta_{con} \):
\[ \theta_{imp} = \theta_{con} = 0.017 \]

**Step 4:** Design wall base connections

Existing procedures (Newcombe et al. 2008) are used to determine the connection moment capacity. The gravity load on each wall will be approximately 9kN.

**Impose connection rotation**

The allowable connection rotation \( \theta_{imp} \) is applied to the connection. A preliminary estimate of the post-tensioning is made: 5-0.5 inch (12.7mm) post-tensioning tendons symmetrically spaced at 60mm centre-to-centre. The tendons are stressed to 60% of their yield stress.

**Estimate the neutral axis depth, c**

Say:
\[ c = 270mm \]

**Apply member compatibility**

The elongation of the tendons, ignoring shortening (Newcombe 2008) is:
\[
\Delta \varepsilon_{pt} = \sum \frac{n \Delta \varepsilon_{pt}}{l_{ub}} = \frac{1}{4250} \begin{bmatrix}
4.25 \\
3.23 \\
2.21 \\
1.19 \\
0.17 \\
\end{bmatrix} = \begin{bmatrix}
0.00100 \\
0.00076 \\
0.00052 \\
0.00028 \\
0.00004 \\
\end{bmatrix}
\]

And:
\[
\varepsilon_{pt} = \varepsilon_{pt, j} + \Delta \varepsilon_{pt} \leq 0.9 \varepsilon_y
\]

Where: \[
\varepsilon_{pt, j} = \frac{T_{pti}}{A_{pt} E_{pt}} \quad \text{and,}
\]
\[
T_{pti} = 464 \text{kN}
\]

Hence:
\[
\varepsilon_{pt, j} = \frac{464 \times 10^3}{495 \times 190000} = 0.00493
\]

And, for the worst-case tendon:
\[
\varepsilon_{pt} = 0.00493 + 0.00100 = 0.0059 \leq 0.0074
\]

The monolithic beam analogy (Palermo et al. 2005b) is applied assuming the timber remains elastic, as proposed by Newcombe (2008):

\[
\varepsilon_i = \left(3 \frac{\theta_{\text{imp}}}{L_{\text{cant}}} + \phi_{\text{dec}}\right) \varepsilon
\]

Where: \(L_{\text{cant}}\) = the shear span = \(H_e - 300\text{mm} = 3333-300 = 3033\text{mm}\)

The decompression curvature is:
\[
\phi_{\text{dec}} = \frac{2T_{pti}}{E_i b_n l_w^2} = \frac{2 \times 464 \times 10^3}{10500 \times 144 \times 800^2} = 0.96 \times 10^{-6} \frac{1}{\text{mm}}
\]

Hence:
\[
\varepsilon_i = \left(3 \frac{0.017}{3033} + 0.96 \times 10^{-6} \right) 270 = 0.0048
\]
Appendix A – Design and construction of the test building

To determine the stress in the timber Newcombe et al. (2008) proposes an calibrated effective connection modulus, similar to column-base connections, is used.

\[ E_{\text{con}} = 5775 \text{MPa} \]

The yield stress of the timber:

\[ \varepsilon_y = \frac{f_c}{E_{\text{con}}} = \frac{45}{5775} = 0.0078 > 0.0048 \Rightarrow O.K! \text{ (Not yielding)} \]

**Calculate the forces in the connection**

Since the timber is elastic we can assume a linear distribution of the stress within the compression region of the timber. Because the neutral axis passes through the void in wall, the total wall width can not be considered for calculating the force applied by the timber. Instead the following expression applies:

\[ C_t = 0.5\varepsilon_y E_{\text{con}} \left( c, b - \frac{(c-x)^2}{c} - b_v \right) \]

Where: \( b_v = 50 \text{mm} \) = the width of the void  
\( x = 200 \text{mm} \) = the length of the solid part of the wall

\[ C_t = 0.5 \times 0.0048 \times 5775 \left( 270 \times 144 - \frac{(270 - 200)^2}{270} 50 \right) / 10^3 = 526 kN \]

For the post-tensioning:

\[ T_{pt} = T_{pt,j} + \Delta T_{pt} = 0.7 f_y A_{pt} + \sum \varepsilon_{pt} E_{pt} A_{pt} \]

\[ = 464 + (0.00100 + 0.00076 + 0.00052 + 0.00028 + 0.00004) \times 190000 \times 99 / 10^3 \]

\[ = 464 + 19 + 14 + 10 + 5 + 1 = 513 N \]

**Check force equilibrium**

By equilibrium:

\[ C_t = T_{pt} + N^* : \]

\[ \therefore 526 \approx 522 \Rightarrow OK! \text{ (Within 5%)} \]
Evaluate the moment capacity

\[
\phi M_n = \phi \left[ \sum T_{po} \left( v_i - \frac{c}{3} \right) + N^* \left( \frac{l_w}{2} - \frac{c}{3} \right) \right]
\]

\[
= 0.9 \left[ 112 \times \left( 520 - \frac{270}{3} \right) + 107 \times \left( 460 - \frac{270}{3} \right) + 103 \left( 400 - \frac{270}{3} \right) + 98 \left( 340 - \frac{270}{3} \right) \right] \times 10^3
\]

\[
= 148 kN.m
\]

\[M^* \leq \phi M_n\]

\[114 \leq 148 \Rightarrow O.K!\]

Therefore, the preliminary section sizes and specified post-tensioning are sufficient for the connection response.

**Wall section capacity**

The critical section capacity is at the base of wall, with a moment demand of 114kN.m.

**Flexure:**

The section sizes were defined in the preliminary design to minimize deflection, now the section strength must be checked. The wall section will be subjected higher modes of amplification of moments and shears. Since there is no tension shift mechanism for timber walls, it is assumed that it is appropriate to use the reinforced concrete column amplification factors (\(\omega = 1.0\) at the ground floor). In addition, an overstrength factor, \(\phi^0\), of 1.25 is assumed to take into account inaccuracies in for the prediction of post-tensioned connection moment capacity and over-design.

\[
\phi M_n = \phi f_s Z
\]

Where: \(k = k_1 k_8 k_{23}\)

\(k_1 = 1.0\) for short term loads

\(k_8 = 1.0\) (no buckling)

\(k_{23} = 0.85\) for size effect

\[
Z = \frac{2 I_w}{l_w} = \frac{2 \times 5.88 \times 10^9}{800} = 14.7 \times 10^6 \text{mm}^3
\]
\[ \therefore \phi M_n = 1.0 \times 0.85 \times 45 \times 14.7 = 562 kN.m \]

\[ \omega \phi^3 M^* \leq \phi M_n \]
\[ 1.0 \times 1.25 \times 114 \leq 562 \Rightarrow O.K! \]

**Shear:**

Again the wall must be designed for increased forces due to higher modes. It is tentatively recommended that the dynamic amplification factor for the design of reinforced concrete walls is used herein:

\[ \omega = 0.9 + \frac{n_t}{10} = 0.9 + \frac{2}{10} = 1.1 \]

Where: \( n_t \) = number of floors

\[ V^* = 38kN \]
\[ \phi V_n = \phi f_s \cdot A_s \]

Where: \( k = 1.0 \) for short term loads
\( f_s = 5.3\text{MPa} \)

\[ \therefore \phi V_n = 1.0 \times 1.0 \times 5.3 \times 63.5 = 337kN \]

\[ \omega, \phi^3 V^* \leq \phi V_n \]
\[ 1.1 \times 1.25 \times 38 \leq 337 \]

Therefore the section size is sufficient. With the addition of external mild steel reinforcement (see Chapter 4), the moment demand on the walls may increase by a factor of approximately two. Even if this occurs there will be sufficient section capacity.

**A.3.3. Design of the floor diaphragm**

The floor diaphragm is designed to remain essentially elastic for all tests performed on the test building. There are several methodologies for determination of the design forces for the diaphragm (see Chapter 3 and 8). The upper limit for diaphragm design forces according to the UBC (UBC 1997) and IBC (IBC 2003) codes are one times the peak ground acceleration (PGA). However, research has shown this to be non-conservative (see Chapter 3). For flexible diaphragms, it is proposed that each floor is design for
three times the design peak ground acceleration. This corresponds with the parts and portions section in NZS1170.5. Hence, for the prototype structure:

\[ F_{dia} = 3 \cdot pga \cdot m_i = 3 \times 0.532 \times 9.81 \times 84 = 1315kN \]

Scaling to the test building:

\[ F_{dia} = 585kN \]

While this may be appropriate for dynamic earthquake loading, under quasi-static earthquake loading the floor forces not related to the design PGA but the achievable strength of the lateral load resisting systems. Hence, to ensure the floor diaphragm and diaphragm connections remain elastic the base shear from Stage 1 experimental testing are used to design the diaphragm.

\[ V_b = 666kN \text{ (in the EW direction)} \]

This force is increased by 15% to account for the increases in frame strength due to interaction of the floor slab:

\[ V_b = 766kN \text{ (in the EW direction)} \]

This force must be split to level 3 and 2. Hence, the peak diaphragm force is on the Level 3 with:

\[ F_{dia,L3} = 511kN \text{ (in the EW direction)} \]

\[ F_{dia,L2} = 256kN \]

It is conservatively assumed that this diaphragm force acts in both the NS and EW directions. A depiction of the floor diaphragm connections are given below. Further detail is given in the structural drawings within this Appendix.
Slab reinforcement

The slab reinforcement was designed using strut and tie analysis, considering clauses in NZS3101 (2006) for maximum and minimum reinforcement. The mesh was designed to transfer ram induced forces throughout the floor. The worst-case demand on the slab reinforcement was due to loading in the EW direction, illustrated in Figure A.3.9 below.

430 MDT – 200 ductile mesh is specified, which has a bar diameter of 8mm at 250mm centers with a minimum tensile stress, $f_u$, of 430MPa. Ignoring the tensile strength of the concrete, the mesh must provide:

$$F^* = 255kN$$

Hence:

$$\phi F = \phi_u f_u A x b_{slab}$$
Where: $A_s = 201\text{mm}^2 / m = \text{area per meter width}$

$b_{slab} = 4.35\text{m} = \text{width of slab}$

Therefore:

$$\therefore \phi F = 0.9 \times 430 \times 201 \times 4.35 = 343kN \geq 255kN$$

Bent ‘U-bars’ were designed to ensure adequate transfer of slab forces, to notched shear key connections in edge joists (see the structural drawings below). These U-bars provide a shear transfer mechanism, even if the concrete slab is cracked due to out-of-plane loading. Simple shear capacity and development length formula from NZS3101 (2006) are used to design the U-bars.

Additional reinforcing (drag) bars are provided along the edge joists, which distributes the slab forces evenly to the notched connections (see the structural drawings below).

**Notched joist connections**

The notched connections in edge joists (and edge beams) are used to transfer diaphragm forces. These notched connections are identical to those used to provide composite action for gravity loading. The shear capacity of the notch connection was determined considering Yeoh (2010).

**Edge joist-to-frame connections**

The notched edge joists are screwed to the beams (within the seismic frames). The number of screws was determined using equations presented in Chapter 3 (Bejtka and Blass 2002; EC5 1994). On Level 3, screws inclined at 45 degrees were used. On Level 2, screws were placed perpendicular to the beams.

**Edge beam-to-wall connections**

The edge beams were screwed to the walls using orthogonal wood screws (see Chapter 4). The wood screws were positioned into a square pattern, to allow rotation of the wall relative to the edge beam while avoiding screw fracture due to excessive displacement. Standard Eurocode 5 (1994) equations were used to determine the number of screws required.
A.3.4. Other design

For brevity, details of the design of several other components are not presented in this Appendix. Some further information is provided below:

- The timber-concrete composite floor system for the prototype structure was designed according to Yeoh (2010) and TDG (2008). The prototype design was scaled (by 2/3) for the test building.

- Gravity corbels and top-hung joist hangers were designed using bearing formula and shear/axial pull-out wood screw capacities from NZS3603 (1999). These joist hanger connection and corbels were tested by Carradine et al, (2010; 2009).

- The anchorage of the external reinforcement for the frames and walls was designed according to Eurocode 5 (EC5 1994). The frame anchorage pins were designed remain elastic, with timber bearing as the critical mechanism. The screws used to anchor the UFPs to the walls were designed for axial pullout and shear. Pull-out forces were induced because the UFPs were anchored on only one side of the wall (for ease of construction).

- Steel shear keys were used at the column and wall bases, which were designed according to NZS 3404 (1997).

- SPAX screws were used to reinforce the timber surrounding steel anchorage pins in the columns on Level 3. Sufficient screws were provided to prevent timber fracture due to limited edge distances, according to NZS 3603 (1999). Stut and tie analysis was employed to evaluate the force demands on the screws.

- An internal steel plate arrangement was detailed for the columns at Level 2. This was designed to distribute compressive bearing stresses, applied by the rocking beams, and to avoid loading of the column perpendicular to grain. The steel components were designed according to NZS 3404 (1997) and the bearing capacities of the timber were determined using TDG (2008).
A.4. CAPACITY PREDICTION

Using a similar procedure to that shown for design, the capacity of the frame and wall system at 2% drift can be predicted, with and without additional external steel reinforcement. Due to the presence of additional reinforcement, over-design and strength reduction factors, the actual frame and wall capacity will be greater than required by the design. These predictions are compared with experimental results later in this appendix and in Chapter 5.

To make accurate predictions of the frame and wall capacity, iteration of the modeling procedure is required. This is because elastic deformation of the members depends on the moment capacity of the connections. To perform this task the modeling procedure was implemented into a numerical analysis computer package (MATLAB) which could predict the response of the frame and walls at any interstorey drift. Multiple design drift limit states and more accurate calculation of the section properties were used in the MATLAB analyses.

From the predicted response, the hysteretic area-based damping of the system can be evaluated (Jacobsen 1960; Priestley et al. 2007). The area-based equivalent viscous damping, $\xi_{hyst,ab}$, is described in the figure below.

![Figure A.4.1 – Area-based equivalent viscous damping (Priestley et al. 2007)](image)

According to Priestley et al (2007), the total system damping can be determined by taking a weighted average of the damping provided by individual components, where the weighting is defined by the overturning moment (OTM) contribution. For the frame system, the column-base connections and the additional external reinforcement will

\[ \xi_{hyst,ab} = \frac{A_h}{2\pi F_{am} \Delta_m} \]
provide some hysteretic damping, which can be weighted to approximate the total system damping of the frame.

Assuming that the hysteretic response of the column-base connections and additional reinforcement is essentially elasto-plastic, the following formula can be used to determine the component area-based damping (Blandon and Priestley 2005):

$$\xi_{hyst,comp} = \frac{2(\mu - 1)}{\mu \pi (1 + r(\mu - 1))}$$

Where:  
\(\mu\) = the ductility achieved by the component  
\(r\) = the bi-linear factor.

The overall system damping can be determined by taking the weighted average of \(n\) components using the following relation:

$$\xi_{hyst,sys} = \frac{\sum_{i=1}^{n} \xi_{hyst,comp,i} M_{b,comp,i}}{OTM}$$

Where:  
\(\xi_{hyst,comp,i}\) = the hysteretic damping of the \(i^{th}\) component  
\(M_{b,comp,i}\) = the base moment contribution from the component.

**A.4.1. Post-tensioned frame**

The resultant connection capacities are illustrated in Figure A.4.2. Some of the MATLAB output is shown in Figure A.4.3, Figure A.4.4 and Figure A.4.5.

![Figure A.4.2 – Post-tensioned frame bending moment diagram at 2% drift](image-url)
The total overturning moment (OTM) for each frame is 619kN.m. This equates to a total base shear (or ram force) of 188kN and 199kN for Stage 1 and 2 respectively.

The base connections are the only components which have hysteretic energy dissipation potential for the post-tensioned frame (according to this simplified analysis). The connections achieve a ductility of approximately 6, with a bi-linear factor of 0.03, giving an area-based damping of approximately 46% (assuming an elasto-plastic hysteresis). Hence, the system area-based damping is approximately:

$$\xi_{hyst,ab} = \frac{46 \times 3 \times 55}{620} = 12\%$$

The re-centering ratio, $\lambda$, (as defined in Chapter 2) for post-tensioned only frame at 2% drift is:

$$\lambda = \frac{M_{pt} + M_N}{M_s} \geq \alpha_0$$

Where:

$$M_{pt} = TL_{base} = \left(\frac{40 + 63}{4.19/2}\right) \times 2 = 412kNm$$

$$M_s = \sum M_{col} = 55 \times 3 = 165kNm$$

$$M_N = 0$$

Hence:

$$\therefore \lambda = \frac{412}{165} = 2.5$$
Appendix A – Design and construction of the test building

Figure A.4.3 – Analysis output for Level 2 post-tensioned connections at 3 design points
a) Connection moment b) Neutral axis depth c) Tendon forces d) Timber strain e) Deformation components at 3 design points
Figure A.4.4 – Analysis output for Level 3 post-tensioned connections a) Connection moment b) Neutral axis depth c) Tendon forces d) Timber strain e) Deformation components at 3 design points
Appendix A – Design and construction of the test building

Figure A.4.5 – Analysis output for column-base connections
a) Connection moment
b) Neutral axis depth
c) Strain in steel bars
A.4.2. Hybrid post-tensioned frame

With the addition on the mild steel reinforcement between the beam-column connections (creating a hybrid system), the capacity of the frame system will increase. This can be predicted, using a similar design procedure to that shown above, with additional details from Newcombe et al. (2008).

It is assumed that the external reinforcement has a yield stress of 430MPa, a bi-linear stress-strain relationship and anchorage slippage of 0.5mm. The response of the column base connections is unchanged from the post-tensioned frame.

The resultant connection capacities are illustrated in Figure A.4.6. Some of the MATLAB output is shown in Figure A.4.7 and Figure A.4.8.

![Figure A.4.6 – Hybrid post-tensioned frame bending moment diagram at 2% drift](image)

The total overturning moment (OTM) for each frame is 720kN.m. This equates to a total base shear (or ram force) of 218kN and 232kN for Stage 1 and 2 respectively.

The base connections and additional beam-column reinforcement provide hysteretic energy dissipation to the system. Once activated, the beam-column reinforcement achieves a ductility of approximately 2.2 and 1 on Level 2 and 3 respectively. Hence, the steel does not yield on Level 3 according to the analytical predictions. The bi-linear factor is approximately 0.01, giving an area-based damping of approximately 34% for the Level 2 connections. The contribution of mild steel from the Level 1 connections to the OTM is 83kN.m (12%). Hence, adding the dissipative contributions of the base-
connection and mild steel reinforcement from the beam-column connections, the system area-based damping is approximately:

\[ \tilde{\xi}_{hyst,ab} = \frac{46 \times 165 + 83 \times 34}{720} = 14\% \]

The re-centering ratio, \( \lambda \), (as defined in Chapter 2) for hybrid frame at 2% drift is:

\[ \lambda = \frac{M_{pt} + M_N}{M_s} \geq \alpha_0 \]

Where:

\[ M_{pt} = \left( \frac{41 + 61}{4.19/2} \right) 4.19 \times 2 = 408kNm ; \]
\[ M_s = 55 \times 3 + \left( \frac{19 + 5}{4.19/2} \right) 4.19 \times 2 = 261kNm ; \]
\[ M_N = 0 . \]

Hence:

\[ \therefore \lambda = \frac{408}{261} = 1.6 \]

The re-centering ratio is greater than minimum value, \( \alpha_0 \), suggested by current precast concrete design (NZCS 2010) of 1.5.
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe

Figure A.4.7 – Analysis output for Level 2 hybrid connections at 3 design points a) Connection moment capacity b) Neutral axis depth c) Tendon forces d) Forces in external reinforcement e) Deformation components at 3 design points
Figure A.4.8 – Analysis output for Level 3 hybrid connections a) Connection moment capacity b) Neutral axis depth c) Tendon forces d) Forces in external reinforcement e) Deformation components at 3 design points
A.4.3. Post-tensioned wall

The wall capacity is predicted using MATLAB, following a similar approach to that of the frame design (see the previous section). The resultant connection capacity is illustrated in Figure A.4.9. Some of the MATLAB output is shown in Figure A.4.10.

![Post-tensioned wall bending moment diagram at 2% drift](image)

Figure A.4.9 – Post-tensioned wall bending moment diagram at 2% drift

Both walls provide an overturning moment (OTM) of 328kN.m. This equates to a total base shear (or ram force) of 105kN (53kN per wall).

The hysteretic area-based damping for this system is zero, according to this analysis.
Appendix A – Design and construction of the test building

Figure A.4.10 – Analysis output for post-tensioned wall connection a) Connection moment capacity b) Neutral axis depth c) Tendon forces d) Stress in timber e) Deformation components at 3 design points

Design Level: 1=SLS, 2=ULS and 3=MCE
A.4.4. Hybrid post-tensioned wall

For the hybrid wall, UFP coupling devices positioned between the walls generate additional axial tension and compression forces in the respective walls (Kelly et al. 1972). This increases the capacity of the wall system and results in additional shear forces in the walls and increased elastic deformation. Design and modeling procedures for the coupled wall systems are provided in Chapter 7. The resultant connection capacity and coupling forces are illustrated in Figure A.4.11. Some of the MATLAB output is shown in Figure A.4.12.

![Figure A.4.11 – Hybrid post-tensioned wall bending moment diagram and UFP shears at 2% drift](image)

Both walls provide an overturning moment (OTM) of 503kN.m (177kN.m from Wall 1, 166kN.m from Wall 2 and 160kN.m from the coupling action provided by the UFPs). This equates to a total base shear (or ram force) of 161kN (53kN per wall).

The UFP provide hysteretic energy dissipation potential to the wall system. The connections achieve a ductility of approximately 8, with a bi-linear factor of 0.01, giving an area-based damping of approximately 52%. Hence, the system area-based damping is approximately:

$$\xi_{hyst,ab} = \frac{52 \times 160}{503} = 17\%$$
The re-centering ratio, $\lambda$, (as defined in Chapter 2) for coupled walls at 2% drift is:

$$\lambda = \frac{M_{\mu} + M_N}{M_s} \geq \alpha_0$$

Where: $M_{\mu} + M_N = 177 + 166 = 343\, kNm$;

$M_s = 160\, kNm$.

Hence:

$$\therefore \lambda = \frac{343}{160} = 2.1$$

The re-centering ratio is greater than minimum value, $\alpha_0$, suggested by current precast concrete design (NZCS 2010) of 1.5.
Figure A.4.12 – Analysis output for hybrid post-tensioned wall connection
a) Connection moment capacity
b) Moment demand in each wall
c) Neutral axis depth
d) Tendon forces
e) Stress in timber
e) Deformation components at 3 design points
A.5. EXTERNAL REINFORCEMENT TESTS

A.5.1. Introduction

Material tests were performed on the column-base external reinforcement (energy dissipaters) depicted in Figure A.5.1. Similar steel and detailing was used for the beam-column external reinforcement; therefore, material tests were only performed on the column-base reinforcement. Twenty seven devices were fabricated; twenty-four were used for the building and three were tested under cyclic loading to ensure they perform satisfactorily.

Each mild steel energy dissipater had a 24mm thread and was machined down to 16mm over a central fuse length. The dissipater was incased by a 32mm diameter pipe and filled with epoxy resin to prevent buckling in compression.

![Figure A.5.1 – Column-base external reinforcement](image)

A.5.2. Test setup

The test setup was designed to simulate the actual building anchorage details as close as possible (see Figure A.5.1). Two pots were attached to fixing nuts at the top and bottom
of the dissipaters. Two strain gauges were attached at the center of the fuse length on opposite sides. One strain gauge was placed at the base of dissipater, in the section of rod that was 24mm in diameter (without any thread).

Figure A.5.2 – Column-base external reinforcement test setup

A.5.3. Loading protocol

The loading protocol was based on ACI T1.1 (2001). Three cycles were performed at different displacement amplitudes. The displacement amplitude was increased by 1.5 times the previous amplitude (see Figure A.5.3).

Only elongations (or positive displacements) were considered. From section analysis of the column-base connection (Newcombe et al. 2008), as shown in the previous section, it was determined the dissipaters will not be subject to significant negative or compressive displacements. At small displacements, the neutral axis is larger than the depth of the dissipation devices but the negative displacement demand will be negligible.
A.5.4. Measured properties

The diameter of the fuse length was measured in three locations to gain an accurate estimate of the stress in the steel. The length between fixing nuts on the top and bottom of the dissipater is also recorded.

![Figure A.5.3 – Loading protocol a) Large cycles b) Small cycles](image)

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<td>15.96</td>
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<td>16.03</td>
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<table>
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<th>Test #</th>
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<td>T2</td>
<td>361</td>
</tr>
<tr>
<td>T3</td>
<td>362</td>
</tr>
</tbody>
</table>
A.5.5. Summary of test results

For all tests, the stable hysteretic loops were achieved until above 5% strain, no compression buckling occurred and yielding was localized to the fuse length. The yield strength of the steel was approximately 420MPa on average. Some specific conclusions are given below with reference to Figure A.5.4.

For Test 1:

- The strain gauges stopped recording at a maximum displacement of approximately 10mm.
- A 14mm stable displacement was achieved.
- Therefore, stable loops were achieved at approximately 5% strain. No buckling of the dissipaters occurred.

For Test 2:

- The strain gauges stopped recording at a maximum displacement of approximately 4.3mm.
- A 14mm stable displacement was achieved.
- Maximum stable strain is approximately 6%. No buckling of the dissipaters occurred.
- There were slightly lower stresses in compression. This indicates that there is some load sharing from the tube. This could be via friction between the tube, epoxy and bar.

For Test 3:

- The testing methodology was altered slightly. The potentiometer (which was attached to the nuts) and not the loading bridge followed the displacement protocol. In the previous tests, there was a small amount of elastic deformation in the test apparatus, which prevented the energy dissipaters from achieving
zero displacement after the first cycle. This may be slightly non-conservative as higher compressive strains may result in a tendency for the devices to buckle.

- The strain gauges stopped recording at a maximum displacement of approximately 7mm.
- A 14mm stable displacement was achieved.
- Hence, the maximum stable strain is approximately 7% with no buckling of the dissipaters.
- As in test 2, there were slightly lower stresses in compression
Figure A.5.4 – Steel material test results

a) Force-displacement response
b) Stress-strain response
c), d) & e) Failure mode of Test 1, 2 & 3 respectively
# Experimental Post Tensioned Timber Building

**Drawing List**

<table>
<thead>
<tr>
<th>Assembly</th>
<th>Components</th>
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</thead>
<tbody>
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<td>0-1</td>
<td>Material View</td>
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<tr>
<td>1-1</td>
<td>Plan level 1</td>
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<td>1-2</td>
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<tr>
<td>1-3</td>
<td>Plan level 3</td>
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<td>2-1</td>
<td>Detail B.1</td>
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<tr>
<td>3-1</td>
<td>Detail B.2</td>
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<tr>
<td>3-2</td>
<td>Assembly Details</td>
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**Components**

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<td>C1, C2, C3, C4</td>
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<tr>
<td>Composite Beam (Outer)</td>
<td>C1, C2, C3, C4</td>
</tr>
<tr>
<td>Floor Unit</td>
<td>F1, F2, F3, F4, F5, F6</td>
</tr>
</tbody>
</table>
Appendix A – Design and construction of the test building

PLAN LEVEL 1
(Strong floor)
Scale 1:50

Fabricated steel column base
and column keeper plates
supplied and installed by
Engineer
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe

---

**FLOOR NOTES**

All prefabricated floor units to have 15mm ply and 200x45 LVL. Floor joists (4 per unit) **not** cut back to allow site access to energy dispenser. CONTRACTOR to provide temporary support to alignment beams to ENGINEER approval. (low timber blocks, well notched at each location) ***Floor joists at edges of prefabricated floor units connected on site by CONTRACTOR

---

**PLAN LEVEL 2**

Scale 1:50

| Frame columns | 300x244 | LVL |
| Frame beams | 200x200 | LVL |
| LGI beams | 63x301 | LVL x3 |
| PB1 beams | 150x300 | LVL |
| and walls | 400x144 | LVL |

Fabricated steel energy dissipators supplied and installed byContractor
Appendix A – Design and construction of the test building

FLOOR NOTES

All preabricated floor units to have 15mm ply and 200x45 LVL floor joists (4 per unit)

** Floor joist cut back to allow site access to energy depressor. CONTRACTOR to provide temporary support to adjacent beams to ENGINEER approval. (Allow timber blocks, wall nailed at each location)

*** Floor joists at edges of preabricated floor units connected on site by CONTRACTOR

PLAN LEVEL 3

Scale 1:50

frame columns 235x44 LVL

floor beams 224x40 LVL, LVL beams 63x309 LVL x2

Fabricated steel energy dissipators supplied and installed by ENGINEER
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe

Frame columns: 300x544 LVL
Frame beams: 400x544 LVL
Primary beam and edge beams omitted for clarity
Frame units omitted for clarity

4-12.7mm dia. 7-wire post-tensioning strand supplied and installed by Sub Contractor

Energy dissipators supplied and installed by Sub Contractor (omitted for clarity)

4-12.7mm dia. 7-wire post-tensioning strand supplied and installed by Sub Contractor

Fabricated steel column base and column/beam plate supplied and installed by Sub Contractor
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe
Appendix A – Design and construction of the test building

Level 1 column

Level 2 column

Typical column

Level 2 corbel CB1, column C2, C5
Level 2 corbel CB2, column C1, C3, C4, C6
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe

Beam 86, 87, (as shown)
Beam 86, 85, (opposite None)
Area section

Beam 86, 85, (as shown)
Beam 86, 87, (opposite None)
Plan

2-50mm dia fabricated steel pipe supplied by MANUFACTURER and installed by CONTRACTOR to be screwed from below.

300x45 LVL side carnel with flat keys and W150 Connection on top surface

SPHC screws shown for outer cladding only, inner cladding to be screwed from below.

Beam details refer dwg 4-3
Appendix A – Design and construction of the test building

Typical beam B5, B6, B7, B8

Side corbel B5, B6, B7, B8

Side corbel B5, B6, B7, B8
Appendix A – Design and construction of the test building

end fixing                              typical beam                              typical slab key

beam spacers
inner edge beam 63x300 LVL
outer edge beam 63x300 LVL to be fixed
on site, refer deg 2-3

edge beam spacer

14g-10x150mm long R617x2
167 wood screw Class 3
8 screws per 500mm as shown

50x35x34g steel flat
supplied by UNICORN and
installed by MANUFACTURER.

M12 x 130 long countersunk to
project 45mm above LVL beam
LVL beam to have max 17mm
depth recess >100mm long

150x50 x10mm ply side
plate, drilled
refer plan for exact keys
with to side plates
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe

Floor unit F1

Plan

Floor unit F1
dimension

Floor unit details refer diag B-6

TENDER

Department of Civil Engineering

EXPERIMENTAL
POST TENSIONED
TIMBER BUILDING TESTING

FLOOR UNIT F1

MAX 120
MIN 50

MAX 4.1
MIN 7
Appendix A – Design and construction of the test building

Floor unit F2 (as shown)
Floor unit F3 (assemble base)

Floor unit details refer page 8-9
Appendix A – Design and construction of the test building

Floor unit F5

Plan

Elevation

Floor unit details refer dwg 8-8
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe
Appendix A – Design and construction of the test building

floor joist hangers

floor joist fixing

TENDER
A.6.2. Steel components

NOTE: 1. The partial bevel weld must not protrude from the depth of the bevel.
2. All plates are mild steel.
3. All bolts are grade B8.
Note: four units were used to couple the walls (eight in total).
Appendix A – Design and construction of the test building

Section A-A1: Diagram showing dimensions and connections.

NOTES:
1. All dimensions are in millimeters.
2. All materials are specified.

200x25mm FLAT
(Grade 300)

2-80x10mm FLAT
Web Plates
(Grade 300)

NOTE: Tack welds are 1.5mm diameter and 25mm long.

× 4 TYPE 1 Anchorage (L3)
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe

100x25mm FLAT (Grade 300) × 4 TYPE 2 Anchorage (L2)

200x25mm FLAT (Grade 300) 2-80x10mm FLAT Web Plates (Grade 300) × 4 TYPE 3 Anchorage (Walls)

NOTE: 1. Use fillet welds around the perimeter of the web plates.
**EXPERIMENTAL POST TENSIONED TIMBER BUILDING**

**STAGE TWO 20.08.09**

<table>
<thead>
<tr>
<th>ASSEMBLY</th>
<th>COMPONENTS</th>
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<td>2-1 Assembly Details</td>
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<td>1-2 Plan</td>
<td>3-1 Steel Materials List</td>
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</tbody>
</table>

**Appendix A - Design and construction of the test building**

engineer supervising draughtsman
Michael Newcombe
Prof. Andrew Buchanan
Michael Newcombe

Original c/o Sid Kennedy
FLOOR SLAB REINFORCING NOTES
The slab will have a minimum of 50mm topping. Refer to specification for concrete properties.
** Tailing apparatus installed by ENGINEER after concrete pour
*** Edge joints are facedown to become by ENGINEER

PLAN LEVEL 2
Scale 1:50
M1 Mesh: 451x2400 NMTA30-200

STAGE 2
Department of Civil Engineering
EXPERIMENTAL POST TENSIONED TIMBER BUILDING (STK)
PLAN LEVEL 2
Basic Reinforcement

<p>| | | |</p>
<table>
<thead>
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<tr>
<td>Wedj</td>
<td>150</td>
<td>Wedj</td>
</tr>
</tbody>
</table>
Appendix A – Design and construction of the test building

FLOOR SLAB REINFORCING NOTES
The slabs will have a minimum of 30mm topping. Refer to specification for concrete properties.
** Testing expenses included by ENGINEER after concrete pour
*** Edge joints are finalized to be done by ENGINEER
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe
Reinforcing Steel and Mesh:

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<thead>
<tr>
<th>ITEM</th>
<th>GRADE (MPa)</th>
<th>DIAMETER (mm)</th>
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<td>8 @ 250mm (201mm²)</td>
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<td><img src="./image5.png" alt="Diagram" /></td>
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A.7. DESIGN SPECIFICATIONS OF TEST BUILDING

STRUCTURAL TIMBER FABRICATION

TIM.4 PRELIMINARY
Refer to the Preliminary and General Clauses of this Specification and to the General Conditions of Contract, which are equally binding on all trades. This section of the Specification shall be read in conjunction with all other sections.

In this specification the Principal is the University of Canterbury (UC), the Engineer is the design engineer nominated by the University of Canterbury. The Project Manager is from Manoel and is nominated by the University of Canterbury.

TIM.2 SCOPE
This section of the contract includes the following:

(a) Fabrication, supply and delivery of all glulam timber members, including beams, columns and structural walls, all manufactured from LVL components with steel brackets and fasteners as detailed.

(b) Fabrication, supply and delivery of all flooring units, manufactured from LVL joists and plywood panels with coach screws and fasteners as detailed.

TIM.3 MATERIALS AND WORKMANSHIP
Wood and wood based products, and construction and workmanship, shall be in accordance with the relevant New Zealand Standards, including the following, as appropriate:

- AS/NZS 4367: Structural laminated veneer lumber
- AS/NZS 2269:2004: Structural Plywood

TIM.4 TIMBER QUALITY
This clause applies to all Laminated Veneer Lumber (LVL), plywood, and manufactured timber components.

TIM.4.1 Species
All timber shall be Radiata Pine.

TIM.4.2 LVL and Plywood
All LVL shall be manufactured in a factory with a current licence from the Engineered Wood Products Association of Australia (EWPA), manufactured in accordance with AS/NZS 4357 Structural laminated veneer lumber.

The Contractor shall comply with the requirements of NZS 4357 and AS/NZS 1004.4 for LVL manufacture except where modified by this specification, and use adequate plant and equipment controlled by qualified personnel. Only fabricators certified by the Plywood Association of Australia, which itself is audited by JAS-ANZ (Joint Accreditation System – Australia and New Zealand) will be considered suitably qualified.
Each piece of LVL shall be branded at least once with the product brand name, the date and time of manufacture, the manufacturer’s mill number, and the PAA and JAS-ANZ logos.

All plywood shall be manufactured in a factory with a current licence from the Engineered Wood Products Association of Australasia (EWPA), manufactured in accordance with AS/NZS 2268:2004 Structural Plywood.

TIM.4.3 Treatment
No timber treatment is required.

TIM.4.4 Moisture Content
The moisture content of all timber components shall not exceed 15%. Moisture content variation between adjacent glued laminates must not exceed 2%.

All LVL members shall be provided to the manufacturer true and straight with minimal cupping.

The fabricator shall measure the moisture content before gluing, and inform the Project Manager and Engineer if not within acceptable limits.

TIM.4.5 Grade of LVL
The characteristic modulus of elasticity of LVL must be between 10,000 to 11,000 MPa.

TIM.5 FABRICATED TIMBER

TIM.5.1 Manufacture
All fabricated timber beams, columns and walls shall be fabricated as shown on the drawings and from LVL in accordance with AS/NZS 1558.1666, in a plant holding a current SANZ licence.

Flooring units shall be fabricated as shown on the drawings with plywood nailed to LVL joints.

TIM.5.2 Nominated Manufacturers
Three nominated manufacturers shall be used for the fabrication of the columns, beams, walls and floors: MacIntosh Timber Lamintes, Timber Bond and Hunters. Table 1 is a possible list of which structural elements each nominated manufacturer may construct. The fabricator may make changes to the list by mutual agreement. Any changes shall be advised to the Engineer.

The position of the structural elements is given in Figure 1.

<table>
<thead>
<tr>
<th>Structural element</th>
<th>Number of elements</th>
<th>MacIntosh</th>
<th>Timber Bond</th>
<th>Hunters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame beams</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>Frame columns</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Walls</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Primary beam</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Edge beams</td>
<td>1</td>
<td>2</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Floor units</td>
<td>4</td>
<td>3</td>
<td>4</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Suggested structural elements to be constructed by nominated manufacturers.
TIM 5.3 Adhesive
The adhesive for all fabricated timber shall be Resorenoi adhesive, as specified in NZS 3608 for Service Class 1. All adhesives shall be mixed, applied and cured strictly in accordance with the manufacturer’s recommendations.

TIM 5.4 End Joints
All LVL components shall be provided full length. No end joints are permitted.

TIM 5.5 Testing
Shear blocks from beams, columns and walls shall be tested as specified in AS/NZS 1320.

TIM 5.6 LVL Member Thickness
All LVL members shall be machined on those surfaces to be glued. The finished thickness shall be as necessary to produce the overall beam, column and wall dimensions shown on the drawings.

Extra local machining will be required to fit steel components if the final thickness of the central column laminate is less than 65.5mm.

TIM 5.7 Dimensions and tolerances
The dimensions of the finished timber members shall be as shown on the drawings. All finished timber members shall be within +/- 3mm of the dimensions shown on the drawings except for the following special tolerances:

Special tolerances are as follows:
- Length of beams: +/- 2mm
- Thickness of columns: +/- 2mm
- Squareness of columns at beam-column joints: +/- 1mm
- Squareness of beam end (both ends): +/- 1mm
- Squareness of column and wall end (bottom end): +/- 1mm
- Diameter of drilled holes in beams and in columns at level 3: < +/- 1.0mm
- Squareness of drilled holes in beams and columns: +/- 2 degree
- Squareness of pins within the columns and beams: +/- 1 degree
Appendix A – Design and construction of the test building

The gap between the end grain of the 64mm central laminate and the steel plate insert shall be no more than 1mm.

All dimensions shall be checked before cutting members to final length. Any members not meeting the specified tolerances shall be liable for rejection.

For the holes in the columns drilled adjacent to internal steel components, the hole diameter shall be within 4 mm of the diameter of the pins. If the hole diameter 1mm larger than the diameter of the pins then gaps shall be filled with epoxy according to TIM 5.10.

TIM 5.8 Slots and Holes
All drilling of holes for dowels, and other fixing of bars or brackets, shall be done in the factory.

Flooring units shall have notches in the LVL joists, coach screws, and holes in the plywood sheathing as shown on the drawings.

Where prefabricated steel components are to be inserted into columns, the central column LVL member laminates must be cut to accommodate the steel component, within the tolerances specified in TIM 5.7. Column laminates adjacent to the central laminations may require local machining, according to TIM 5.6, to fit the steel components.

The holes drilled into the columns adjacent to internal steel components may be oversized but shall be filled with epoxy if outside of the tolerances specified in TIM 5.7.

TIM 5.9 Structural steel work items
Take delivery of prefabricated steel plates and pins from the Project Manager or Engineer and build in as detailed in the drawings.

For the pins, holes shall be the same diameter as the steel pins as shown in the drawings.

The pins shall be positioned, as shown in the drawings, so that the U-shaped slots are perpendicular to the column face.

TIM 5.10 Epoxy for gap filling
Epoxy shall be used to keep some pins and column steel components in place, as detailed in the drawings.

Epoxy shall be either of the following listed below unless the Engineer gives written approval for an alternative:

- Araldite 2205 (Nuplex Building Products)
- Araldite K-80 (Nuplex Building Products)
- West System ADR010 / ADH06 (Adhesive Technologies Ltd, Auckland)
- West System Z105 / Z205 or Z105 / Z205 (Adhesive Technologies Ltd, Auckland)
- East 221 epoxy (Polymer Developments, Auckland)

Mix and apply the epoxy in strict accordance with the manufacturer’s instructions.

The method of epoxying shall be such that all spaces around the end plate of the steel component and the column face, and the spaces between the pin and the column timber, are thoroughly and completely filled with epoxy. The steel pins within the column adjacent to the internal steel components shall not be bonded to the epoxy. A lubricant shall be used to prevent bonding between the steel pin and epoxy.

Drill injection holes and air holes as required.
TIM 6.11 Camber
No camber is required. All members shall be supplied straight, to the tolerances specified below.

TIM 6.12 Finish
Exposed surfaces shall be finished with a sanded "Standard" finish in accordance with AS/NZS 1328:1999.

Apply one coat of an approved water-resistant sealer to all surfaces before delivery. No other painted surface finishes are required.

Touch up all damaged surfaces and cut ends before delivery. Application of finishes shall be strictly in accordance with the manufacturer’s recommendations.

TIM 6.13 Wood fasteners
Wood fasteners shall be used as shown in the drawings.

Nails. Type 17 wood screws and coach screws shall be supplied by the manufacturer.

All Type 17 wood screws that are 160mm and 200mm long shall have at least 70mm and 100mm of thread respectively.

TIM 6.14 Timber corbels
Timber corbels shall be attached to the beams and columns as shown in the drawings.

In addition to the wood fasteners attaching the corbels, corbels shall be glued to the beams and columns.

TIM 6.15 Cupped member
Members which have cupped shall be straightened to the best of the manufacturer's ability according to the supplier's recommendations.

TIM 6 FLOORING AND GRAVITY BEAMS

TIM 6.1 Joists
Joists shall be constructed including notches and coach screws as shown in the drawings.

TIM 6.2 Plywood
Plywood shall be 15mm untreated 5 ply, CD grade pinus radiata in accordance with AS/NZS 2289

Plywood shall be nailed with 3.15mm x 60mm galvanised flat head product nails. Plywood is nailed down directly onto LVL joists at no greater than 300mm centres between each nail. Nails in plywood shall be hammered flush with the ply surface.

The top surface of the plywood and all notches in the LVL joists shall be painted with two coats of approved exterior paint.

TIM 6.3 Joint, Primary Beam and Edge Beam hangers
Take delivery of steel joint and gravity beam hangers from the Project Manager or Engineer.

Attach hangers as shown in the drawings.
If any splitting of the LVL occurs all subsequent holes shall be predrilled.

**TIM 7 MARKING**
In addition to the requirements of AS/NZS 1025 the members shall be marked with a suitable label to indicate their position in the assembled structure.

**TIM 8 PROTECTION**
Protect all structural timber members and steel components from the weather and from damage of any kind during transit, storage on site and erection.

Wrap or otherwise protect members from exposure to rain or other water. Store well clear of the ground. Discolouration due to differential exposure to sunlight shall be avoided.

Store and handle so that no damage occurs. Any damaged members will be liable for rejection or repair at no cost to the Principal.

**TIM 9 DELIVERY TIME**
All nominate manufacturers shall deliver all structural elements on-site by the date specified by the Project Manager.

If a nominated manufacturer fails to deliver the structural elements by the date specified by the Project Manager the Principle reserves the right to reject the structural elements and nominate another manufacturer at no cost to the Principle.

**TIM 10 QUALITY ASSURANCE DOCUMENTATION AND RANDOM TESTING**
All nominated manufacturers shall complete quality assurance documentation provided by the Project Manager.

The Project Manager and Engineer shall perform random testing at the nominated manufacturer’s facility on a date specified by the project manager.
STRUCTURAL TIMBER ERECTION

ERE 1 PRELIMINARY
Refer to the Preliminary and General Clauses of this Specification and to the General Conditions of Contract, which are equally binding on all trades. This section of the Specification shall be read in conjunction with all other sections.

ERE 2 SCOPE
Stage 1 of the contract includes the erection of the structural timber columns, beams, walls and flooring units and initial post-tensioning.

Stage 2 of the contract includes additional timber fastening, supply and placement of reinforcing mesh and casting of reinforced concrete slabs.

Stage 3 of the contract includes dismantling of the building and re-erection at a new site.

ERE 3 STAGE 1 - ERECTION

ERE 3.1 Members
Take delivery of all structural timber members and erect to accurate line and level as detailed. The members shall be erected as shown on the drawings, in accordance with all dimensions and notes. Where fixings are not shown, consult the Engineer.

All members shall be erected true and plumb and temporary erection bracing shall be introduced wherever necessary to take care of all vertical and lateral loads to which the structure may be subjected. Such bracing shall be left in place until the building is stable. Once all members are in place post-tensioning shall be performed.

Temporary fasteners shall be used to hold the beams in position. These fasteners shall be removed after stressing.

ERE 3.2 Post-tensioning
All initial post-tensioning shall be performed by a nominated subcontractor, UBB Contech Ltd.

All post-tensioning shall be 12.7mm diameter, 7-wire strands. The number of strands, position of strands in the walls and frames and force in the tendons shall be as shown in the drawings.

The force in each strand after stressing shall be +/-10% of the design force indicated on the drawings. The force in the tendons at any time shall not exceed 140 kN. Each tendon shall be stressed twice beyond the design force before the final stressing.

Use specialized tendon anchorages and shims from the lengthwise as shown on the drawings.

ERE 3.3 Floor units
Erect floor units in place as shown on the drawings.

If any changes are required to be made to the flooring units dimensions, these shall be made by the contractor at no cost to the Principal.
Once floor units are in place they shall be fastened to the frame and edge beams as shown in the drawings.

Adjacent floor units shall be fastened together using standard removable 75mm wood screws at 500mm centres.

ERE.3.4 Pin connections
Take delivery of pins from Engineer and connect walls as detailed on the drawings.

ERE.3.5 Damage
Any damage on erected units is to be repaired and surface marks removed to the satisfaction of the Engineer.
Any damage to the surrounding site or equipment belonging to the Principal shall be repaired at no cost to the Principal.

ERE.3.6 Lifting equipment
Lifting equipment shall be of adequate capacity to safely lift and maintain work in a stable condition until it is securely braced. The contractor shall check construction loads imparted to the structure during erection. Any damage caused shall be repaired at no cost to the Principal.

ERE.3.7 Erection tolerances
The acceptable tolerances in the completed erected structure are as follows:
- Column deviation from vertical line: ± 5 mm
- All other dimensions: ± 5 mm

ERE.3.8 Temporary members
Temporary members shall be supplied were required to support the corner of floor units as shown in the drawings.
Temporary members shall be fastened using standard removable wood fasteners so that the floor units are safe and stable.
STAGE 2 – CONCRETE SLAB

CON 1 PRELIMINARY
Refer to the Preliminary and General Clauses of this Specification and to the General Conditions of Contract, which are equally binding on all trades. This section of the Specification shall be read in conjunction with all other sections and the structural drawings for stage 2 construction.

In this specification the Principal is the University of Canterbury (UC), the Engineer is the design engineer nominated by the University of Canterbury. The Project Manager and Contractor are from Mainzeal Ltd and are nominated by the University of Canterbury.

CON 2 SCOPE
This section of the Contract includes the following:

(a) Preparation of the concrete slab formworks and propping before the concrete pours.
(b) Supply, casting and curing of all in situ concrete on prefabricated plywood panels erected in Stage 1.
(c) Fabrication, supply and delivery of slab reinforcing bar and mesh.
(d) Fixing of anchor bars and starter bars into concrete.
(e) Requirements for concrete properties for the slab.

CON 2 MATERIALS AND WORKMANSHIP
The Contractor shall adhere to all requirements of NZS 3106:1997 (including Amendments 1 and 2), except where specified otherwise herein or instructed otherwise by the Engineer. A copy of this standard shall be kept on the site and relevant parts read with the following clauses of this Specification. Concrete production shall be in accordance with NZS 3104:2003. Cement used in concrete shall be of New Zealand manufacture, complying with all requirements of NZS 3122:1995.

CON 4 INSPECTION
The Engineer may inspect construction in accordance with NZS 3109:1987, Clause 1.3.

CON 5 CONCRETE
Two batches concrete shall be provided, one for each floor of the building. The concrete shall be a special purpose concrete SL as defined in NZS3122.

The concrete shall achieve a slump of between 120mm and 150mm tested over a representative sample in accordance with NZS 3112.1. The Contractor will perform the slump tests. If the concrete does not meet the slump requirements the batch will be rejected.

The 28 day compressive strength of the concrete shall be greater than 25MPa, averaged over 3 samples tested in accordance with NZS 3112.2. The Engineer shall test the samples. Nine test cylinders shall be taken by the Engineer.

The average 58 day drying shrinkage of the concrete shall be no larger than 500 micro strain when tested in accordance with AS 1012.13. The Engineer shall test the samples.

Aggregates shall satisfy the requirements of Clause 8.4 of NZS 3106, and shall be 13 mm maximum size.

All concrete shall be supplied by a ready mixed concrete plant which has a current certificate of audit to demonstrate compliance with NZS 3104:2003. Chloride based accelerators shall not be used in any concrete.
Appendix A – Design and construction of the test building

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CON.6 REINFORCEMENT
Reinforcement, including all necessary distance pieces required to maintain cover, shall be supplied and fixed by Contractor. The slab reinforcing shall be positioned and bent as per the drawings.

The Contractor, or a competent workman, is to be responsible for checking that reinforcement is not displaced during concreting. Any reinforcement so displaced is to be corrected by the Contractor.

CON.7 VIBRATORS
Vibrators shall be used for the placing of all concrete. Vibrators and their use shall comply with the requirements of NZS 3100, Section 7.6.

Vibrators shall be moved to new positions as frequently as necessary to ensure uniform vibration of the whole mass and fully compacted concrete. On no account shall vibrators come within 12 mm of the face of the formwork. Vibrators shall not be used to transfer concrete from one position to another.

CON.8 FORMWORK AND CONCRETING
Formwork shall be constructed by the Engineer.

Construction joints are not required.

The formwork shall be sealed at joints and edges to avoid any concrete seepage using a silicon gel or equivalent.

Stripping times shall not be less than those in NZS 3100, Clause 6.4 unless otherwise agreed by the Engineer.

CON.9 PROPPING
Propping shall be provided by the Engineer.

Each floor shall be propped before the concrete is poured and shall remain propped for at least 3 days.

CON.10 CONCRETE FINISHES

CON.10.1 Slab Finishes
Finish slabs with a steel trowelled finish as specified in NZS 3114:1997 Grade U3. Floating and subsequent finishing shall be at the correct times and intervals to obtain the quality of compaction and finish required. No trowelling in of fines or dry cement will be permitted.

The Contractor shall arrange the pouring of concrete to allow adequate time for floating and finishing.

All trowelling ridges shall be removed while green or by subsequent light grinding. Slabs not fulfilling the standard of finish required shall be ground smooth or otherwise treated to the satisfaction of the Engineer.

CON.10.2 Curing
All concrete shall be cured as defined in NZS 3100, Clause 7.6.

An acrylic resin curing membrane, Sika Antisol A or equivalent shall be applied over the entire slab.
CON 11 PROTECTION, REPAIRS AND CLEANING

Surface concrete and slab surfaces shall be protected from damage at all times. Concrete surfaces shall be repaired or remedied to the satisfaction of the Engineer at no extra cost.

All concrete work in the building, both in situ and precast, shall be closely inspected for faults in surface finish, damage to corners and edges, dirty marks, splashes or dribbles, and visible imperfections of any kind. Inspections are to be carried out within 14 days of placing the concrete. Any remedial work shall be carried out within a further 14 days.

Significant defects in concrete finish shall be referred to the Engineer for specific instructions on repair work. Patching or filling of concrete and making good broken edges and corners shall be done with coloured sands and cement where necessary, to match precisely the colour of the surrounding concrete when dry. Epoxy or similar adhesives shall be used when required.

The removal of surface markings shall be most carefully done by appropriate methods such as wire brushing, pneumatic abrading, carbournium stone, or washing and scrubbing, such as will remove the marks without scratching, discolouring or otherwise affecting the surrounding or underlying concrete.

CON 12 CHANES, PENETRATIONS AND UPSTANDS

No concrete shall be cut or hacked unless specific approval is obtained from the Engineer. This includes existing concrete.
A.8. MATERIAL INFORMATION

Material tests were performed on the concrete which was used for the floor slab. This was required to ensure that the concrete satisfied the design specification (see above) in terms of strength and shrinkage. Material tests were not performed on the Laminated Veneer Lumber (LVL) timber in the test building. Because LVL is anisotropic, the material tests required to accurately define the material properties quickly become cumbersome. Furthermore, complicated end bearing effects may significantly alter the apparent strength of the timber. Instead a combination of existing material test data (Newcombe 2008) and manufacturer information were used to approximate the properties of the material.

A.8.1. Laminated Veneer Lumber Timber

Some basic material properties provided by the LVL producers (Futurebuild 2010; NelsonPine 2010) are given in Table A.3. The average modulus of elasticity was determined by considering an factory test data (Banks 2010), rather than lower-bound design values.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Units</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity</td>
<td>E</td>
<td>MPa</td>
<td>11000</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>G</td>
<td>MPa</td>
<td>660</td>
</tr>
<tr>
<td>Bending strength</td>
<td>fb</td>
<td>MPa</td>
<td>48</td>
</tr>
<tr>
<td>Tension parallel to grain</td>
<td>f_t</td>
<td>MPa</td>
<td>30</td>
</tr>
<tr>
<td>Compression Parallel to grain</td>
<td>fc</td>
<td>MPa</td>
<td>45</td>
</tr>
<tr>
<td>Shear in beams</td>
<td>f_s</td>
<td>MPa</td>
<td>6.0</td>
</tr>
<tr>
<td>Compression perpendicular-to-grain</td>
<td>f_p</td>
<td>MPa</td>
<td>12</td>
</tr>
</tbody>
</table>

The stiffness of the LVL perpendicular-to-grain is not specified by the LVL manufacturers. However, material tests performed by Davies and Fragiacomo (2008), which was documented in Newcombe (2008), suggest a perpendicular-to-grain elastic
modulus of approximately 300MPa. This value is not expected to vary for different grades of LVL, according to the manufacturers (Banks 2010). However, these tests did not consider edge bearing or stress diffusion that would occur in a continuous length of timber (such as a column) subjected to localized compression. Newcombe (2008), showed that edge bearing and stress diffusion effectively increased both the stiffness and strength of the timber element aligned perpendicular-to-grain. Furthermore, LVL material tests presented in Newcombe (2008), indicate the manufacture specified characteristic compressive stresses for LVL are accurate. In Appendix B, experimental data for the test building is analyzed to verify the accuracy of LVL material properties.

A.8.2. Concrete cylinder tests

Ten concrete cylinders were taken halfway through the pour of the floor slabs for the test building. Four cylinders were placed in the fog room and tested at 28days (C1 to C4). Three cylinders were left in ambient air on-site and tested at 28days (C5 to C7). The three remaining cylinders were left in the fog room and tested the same day as seismic testing on the test building began (C8 to C10).

<table>
<thead>
<tr>
<th>Cylinder</th>
<th>Compressive strength (kN)</th>
<th>Compressive strength (MPa)</th>
<th>Average (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>18.6</td>
<td>23.7</td>
<td></td>
</tr>
<tr>
<td>C2</td>
<td>22.8</td>
<td>29.1</td>
<td></td>
</tr>
<tr>
<td>C3</td>
<td>17.8</td>
<td>22.7</td>
<td></td>
</tr>
<tr>
<td>C4</td>
<td>20.8</td>
<td>26.5</td>
<td>25.5</td>
</tr>
<tr>
<td>C5</td>
<td>19.9</td>
<td>25.4</td>
<td></td>
</tr>
<tr>
<td>C6</td>
<td>18.8</td>
<td>24.0</td>
<td></td>
</tr>
<tr>
<td>C7</td>
<td>19.4</td>
<td>24.7</td>
<td>24.7</td>
</tr>
<tr>
<td>C8</td>
<td>17</td>
<td>21.7</td>
<td></td>
</tr>
<tr>
<td>C9</td>
<td>25.3</td>
<td>32.3</td>
<td></td>
</tr>
<tr>
<td>C10</td>
<td>20.6</td>
<td>26.3</td>
<td>26.7</td>
</tr>
</tbody>
</table>
The overall average 28 day compressive strength was 25.2MPa, marginally within the specification.

A.8.3. Concrete shrinkage tests

Determining the shrinkage of the concrete slab was crucial to the experimental test setup. If the slab shrinkage was significant the floor units would sag. Significant sag would have made it difficult to connect the in-plane loading apparatus and could have caused the floor to buckle.

Standardized shrinkage tests according to AS1012.13 (1992) were performed on the floor slab concrete. As per the standard 3 prism samples (S1, S2 and S3) were taken and the shrinkage was calculated up to 56 days after the concrete pour.

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Strain (microstrain)</th>
<th>Average (microstrain)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>S1</td>
<td>S2</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>21</td>
<td>412</td>
<td>444</td>
</tr>
<tr>
<td>35</td>
<td>540</td>
<td>552</td>
</tr>
<tr>
<td>56</td>
<td>636</td>
<td>672</td>
</tr>
</tbody>
</table>

The specified shrinkage limitation for the concrete was 600 microstrain at 56 days (see the specification). Hence, the concrete was not within the specification by 45 microstrain.

A sample of was taken to more accurately determine the actual shrinkage deformation of the timber-concrete composite (TCC) floor. The sample is 500×500×50mm and had similar boundary conditions to the actual concrete slab (see Figure A.8.1) such as a similar surface area to volume ratio (10%) and plywood formwork on the base. No steel reinforcement was included in the sample. All measurements were taken at approximately midday.
The deformation was recorded for 3 gauge lengths over a period of 150 days. The recorded deformation is plotted below.

The following conclusions are drawn:

- The shrinkage strain plateaus at approximately 820 microstrain.
- This is significantly larger than the shrinkage of the AS1013.12 samples.
- Using an effective modulus approach (Fenwick and Mackechnie 2009), assuming fully composite action, a 10mm floor deflection is predicted due to shrinkage.
A.9. CONSTRUCTION OF THE TEST BUILDING

All prefabricated timber components were delivered and stacked on-site (see Figure A.9.1a). The timber components were assembled by professional contractors (from Mainzeal) whom have experienced in the construction of concrete and steel buildings. The post-tensioning was applied to the frames and walls by specialist contractors (BBR Contech Ltd). Subsequently, a subcontractor (Allied Concrete Ltd) cast the concrete slab in-situ.

The assembly of the timber components (see Figure A.9.1) took 15 hours (2 working days) using 4 construction workers (less than half of the construction time predicted by Mainzeal). Additional works involved post-tensioning and casting the floor slabs. The post-tensioning (see Figure A.9.1c) was completed in 2 hours (approximately 15% of the assembly time). The concrete pouring, leveling and floating took 1 day (Figure A.9.1d). A pre-camber was not built into the floor system; hence propping was required for 48 hours.

Half of the prefabricated timber components were delivered to site one day late. Notably, this had little impact on the construction time. The modularity and simplicity of the structural system allowed many alternative construction sequences.

The fabrication tolerances (see the above specification) significantly affected the speed of construction. Equivalent buildings in precast concrete typically require that building tolerances are taken up by grout pads between the beam and column faces. Using grout (or epoxy) pads is labor intensive, requires skilled labor and strict quality assurance onsite. By specifying strict tolerances and performing an accurate foundation setout, these measures can be avoided. In addition, strict tolerances ensure that the frames remain straight and the walls remain plumb during stressing.

Aspects of onsite safety and labor significantly increased the construction speed, when compared to steel and concrete. Handrails could be attached to beams before they were lifted in position. Structural elements were significantly lighter, compared to concrete, reducing the required cranage time and risk of injury when positioning elements. Floor
units created a safe working platform and were fastened to beams from the top, avoiding the need for ladders to fasten the joists. Using prefabricated elements reduced onsite clutter and minimized the number of workers on-site, creating a more efficient work environment.
Figure A.9.1 Construction progress a) Start b) At the end of day 1 c) Floor unit d) At end of day 2 e) Applying post-tensioning d) Concrete pour
APPENDIX B

FURTHER DETAIL ON THE EXPERIMENTAL RESPONSE
OF THE TEST BUILDING

This appendix provides further details for the experimental response of the test building, summarized in Chapter 5. The global hysteretic results for all tests described and illustrated. Subsequently, the connection-response of the wall and frame system is analyzed in an attempt to evaluate the contribution from each structural component to the total strength of the building.

Other parameters are examined to further characterize the experimental response and performance of the building. These parameters include tendons forces, frame elongation, slab elongation, column and wall inclination, timber properties and floor sagging deformation.

B.1. GLOBAL BUILDING RESPONSE

Within this section, further details (in addition to Chapter 5) are provided on the global response of the test building in terms of overturning moment (OTM) versus drift for all of the seismic tests.

B.1.1. Uni-directional response

Figure B.1.1a shows the hysteretic response of the building with post-tensioned (PT only) frames without floor diaphragms (Stage 1, Test 1). The figure demonstrates that there is limited inelastic response with little hysteretic energy dissipation, as shown in Figure B.1.2a, up to the design level of displacement (2% interstorey drift). There is a slight loss of strength and stiffness observed in Figure B.1.1a. This can be partially
attributed to inelastic deformation of the timber in the west beam-column (BC) connections, induced by concentrated loads applied by the loading apparatus. The presence of inelastic (or permanent) deformation is verified by considering the area-based damping in Figure B.1.2a. There is a significant reduction in the energy dissipation of the frames from the first to the second and third cycles, indicating that permanent deformation of the timber has occurred during the first cycle. Furthermore, some of the column-base connection attachments failed during this test (see Appendix C), which equated to approximately 12% of the frame capacity, and contributed to the loss strength and stiffness for the frame system. This may have also affected the energy dissipation potential of the system.

The frame systems moment capacity is higher than predicted at 2% drift by approximately 17%. However, taking into consideration that some of the column-base connections failed (equating to approximately 215kN.m), this percentage is likely to increase to approximately 35% (430kN.m). The hysteretic damping varies between 5% and 3%, which is much lower than the 12% predicted. This indicates the base connections are not providing as much energy dissipation to the system as expected. This could be due to slippage in the base connections and larger than predicted non-dissipative contributions to the OTM. Furthermore, it is likely that there are modelling inaccuracies associated with the damping predictions (see Appendix A).

External (non-stressed) reinforcement was added to the BC connections, creating a Hybrid system (Stage 1, Test 2). Figure B.1.1b shows that there is minimal loss of stiffness and strength. Considering the hysteretic damping, at small drifts there is little difference between the first and subsequent cycles (see Figure B.1.2b). At drifts in excess of 1%, the hysteretic damping reduces after the first cycle (see Figure B.1.2b). This indicates that further inelastic deformation occurred during this test at Drifts above 1%. The moment capacity is higher than predicted by 36% (480kN.m) at 2% drift. Again, the hysteretic damping varies between 3% and 5% and is significantly under the predicted value (14%). Hence, the BC connections are not providing significant hysteretic damping to the system. There was only a slight increase in frame strength at
2% drift due to the presence the external reinforcement. See Chapter 5 for more detail regarding the effects of external reinforcement.

Figure B.1.1c shows the hysteretic response for the building with post-tensioned (PT only) frames and TCC floor diaphragms (Stage 2, Test 1). There is no significant loss in stiffness and strength. The hysteretic damping only reduces slightly from the first cycle to subsequent cycles (see Figure B.1.2c), indicating minor inelastic response in either the timber elements or the floor slab during this test. This inelastic deformation is likely to be derived from minor cracking of the concrete slab and crushing of the timber at higher drifts (in addition to the crushing that occurred in previous tests). The moment capacity is 46% (570kN.m) higher than predicted. With the floor diaphragm, the difference between the experimental and predicted response at 2% drift is exacerbated. The area-based damping has increased to a maximum of approximately 6% but is well under the predicted value (12%). There was a slight increase in the strength of the frame system due to the presence of the TCC floor. Refer to Chapter 5 for more detail on the influence of the floor on the response of the frame.

For the Hybrid frames (Stage 2, Test 2) with floor diaphragms, the hysteretic response for the building was highly stable, as shown in Figure B.1.1d, with no apparent stiffness or strength degradation. The hysteretic damping is effectively equal for each cycle (Figure B.1.2d), indicating that there is no further significant inelastic deformation of the timber or the floor slab (during this test). The moment capacity is 59% (790kN.m) higher than predicted. The area-based damping varies between 3% and 6% and is well under the predicted value (14%). Again, external reinforcement did not provide significant energy dissipation to the system.
Figure B.1.1. Uni-directional hysteretic response of frames: a) PT only (Stage 1, Test 1) b) Hybrid (Stage 1, Test 2) c) PT only with floor (Stage 2, Test 1) d) Hybrid with floor (Stage 2, Test 2)
Figure B.1.2. Uni-directional area-based damping of frames: a) PT only (Stage 1, Test 1) b) Hybrid (Stage 1, Test 2) c) PT only with floor (Stage 2, Test 1) d) Hybrid with floor (Stage 2, Test 2)
B.1.2. Uni-directional wall response

The hysteresis loops for all the uni-directional wall tests (from Stage 2) are described below and compared the capacity predictions. During testing, it was determined that there was significant overturning moment capacity derived from the interaction of the floor system and the wall response. Hence, the walls were tested with and without edge beam support to the columns to gauge the relative contribution of edge beams and the floor diaphragm to the capacity of the system.

Figure B.1.3a shows the hysteretic response of building with post-tensioned (PT only) walls with edge beam support (Stage 2, Test 3). The response is essentially elastic up to 2% drift. The hysteretic damping is relatively low (see Figure B.1.4a), compared to the frames, and reduces slightly for the second and third cycles, indicated minor levels of inelastic deformation in the wall elements or the floor slab. The overturning moment capacity of the system is 102% higher than predicted at 2% drift. This can be attributed to the coupling action provided by the floor system and edge beams and the out-of-plane response of the frames. The frame base connections provide approximately 390kN.m of overturning moment acting out-of-plane. Subtracting this from the peak experimental response gives approximately 940kN.m. With this reduction, the experimental response is still 43% (280kN.m) higher than predicted. Hence, the coupling action provided by the floor system and the edge beams increases the system capacity by approximately 40%, assuming that the prediction methodology is accurate. This coupling effect is likely to be amplified due to the short span between frames, which was required due to experimental testing constraints. See Chapter 5 for more detail in the effect of the floor system on the strength of the wall system.

The hysteretic response of the building with Hybrid walls (Stage 2, Test 4) is shown in Figure B.1.3b. The system response is stable and has no significant reduction in hysteretic damping with subsequent cycles (see Figure B.1.4b), indicating that there is no (further) inelastic deformation in the wall elements or slab. The additional reinforcement (UFP couplers) provided limited energy dissipation to the wall system. The hysteretic damping remained effectively similar for each cycle, indicating no
inelastic deformation of the wall elements. The moment capacity of the wall system is 70% higher than predicted at 2% drift. Subtracting the out-of-plane moment capacity of the frame, the experimentally achieved moment is still 31% (320kN.m) higher than predicted. The hysteretic damping varies between 7% and 3%, which is much less than predicted (17%). Because the non-dissipative components of the wall system are much stronger than predicted, the overall system damping is significantly reduced. Refer to Chapter 5 for a comparison of the wall response with and without UFP couplers.

The hysteretic response of the post-tensioned (PT only) wall system, without edge beam supports is presented in Figure B.1.3c (Stage 2, Test 6). Again, the system is stable. There is little hysteretic damping, which varies from 4% to 2% (see Figure B.1.4c). The moment capacity of the wall system is 87% higher than predicted. Subtracting the out-of-plane moment capacity of the frames, the experimental moment capacity is still 28% (190kN.m) higher than predicted.

Figure B.1.3d gives the hysteretic response of the Hybrid wall system without edge beam supports (Stage 2, Test 5). The system is highly stable, exhibiting no significant loss of stiffness or loss of hysteretic energy dissipation potential (Figure B.1.4d). The moment capacity is 58% higher than predicted. Without the moment contribution for the frame acting out-of-plane, the capacity is still 20% (200kN.m) higher than predicted. The achieved hysteretic damping is similar with or without the edge beam supports (3% to 7%), and is much lower than predicted (17%).
Figure B.1.3. Uni-directional hysteretic response of walls: a) PT only (Stage 2, Test 3) b) Hybrid (Stage 2, Test 4) c) PT only, without edge beam support (Stage 2, Test 6) d) Hybrid, without edge beam support (Stage 2, Test 5)
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Figure B.1.4. Uni-directional area-based damping of walls: a) PT only (Stage 2, Test 3) b) Hybrid (Stage 2, Test 4) c) PT only, without edge beam support (Stage 2, Test 6) d) Hybrid, without edge beam support (Stage 2, Test 5)
B.1.3. Bi-directional Response

Two bi-directional tests were performed. For the first test (Stage 2, Test 7), additional external reinforcement was present on both the frames and walls. Another test was attempted (Stage 2, Test 7), using only the final displacement amplitude of the loading protocol (corresponding to 3% drift). Fracture of some of the column-base reinforcement occurred due to low cycle fatigue (see Appendix C) after multiple tests, which resulted in the termination of the test after two bi-directional clovers were complete. The global bi-directional response is presented by calculating the total overturning moment (OTM) capacity versus the drift applied to the control columns (termed control drift in Chapter 5), in each orthogonal direction. The area-based damping is plotted for the uni-directional cycle (in the NS and EW direction), subsequent to the bi-directional clover.

Figure B.1.5c and d, show the hysteretic response of building, in the EW and NS direction respectively, with Hybrid frames and walls (Stage 2, Test 7). There is a noticeable loss of strength in the EW (frame) direction above 2% drift, due to a column fracture at the top floor (see Appendix C) and further inelastic deformation in the BC connections. In the NS (wall) direction, there is also some loss of strength and stiffness for the 3% drift cycle, indicating inelastic deformation of the timber wall elements. The area-based damping curves show an increase in energy dissipation at 3% drift, which is likely to be due to increased activation of the UFP couplers and, to a limited extent, inelastic timber deformation at the base of the wall elements. In the frame direction, the overturning moment capacity for the system is 48% higher than predicted at 2% drift. In the wall direction, the moment capacity of the system 58% higher than predicted. The area-based damping is still much less than predicted (see Figure B.1.6).

Figure B.1.5a and c, show the achieved hysteretic response of building, in the EW and NS direction respectively, with post-tensioned (PT only) frames and walls (Stage 2, Test 8). There was significant reduction in stiffness and strength after Test 7. The moment capacity is still approximately 6% and 48% higher than predicted for the frame
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and wall directions respectively. The area-based damping values for the one complete cycle are 5% and 7%, still well under the predicted value (12%).

Figure B.1.5. Bi-directional hysteretic response a) EW direction, PT only (Test 8) b) EW direction, Hybrid (Test 7) c) NS direction, PT only (Test 8) d) NS direction, Hybrid (Test 7)
B.2. CONNECTION RESPONSE

The moment in the connections can be estimated using strain gauges and potentiometers that placed throughout the test building. Curvatures, $\phi$, are calculated using the strain gauge or potentiometer data, the second moment of inertia, $I$, and the modulus of elasticity, $E$. Hence:

$$M = \phi EI$$

Manufacturers of LVL provide MOE design values that are an estimated lower bound (Futurebuild 2010; NelsonPine 2010). It is likely that the average MOE of LVL is higher than specified. An MOE for the frames and walls was assumed to be 11GPa. This value is recommended by the LVL producers (Banks 2010) based on factory material tests and is within the range in the specification shown in Appendix A.
B.2.1. Uni-directional frame tests

For the north frame, 90mm strain gauges were attached to some of the beams and columns. These are used to estimate the curvatures, and hence moments in the structural elements. Using linear extrapolation, from the position of the strain gauges, the moment at the beam-column (BC) connections can be evaluated. Potentiometers that straddle each BC connections are used to approximate the connection rotation.

Caution was exercised when interpreting the strain gauge data. If the strain gauges were placed in a region where Bernoulli’s linear strain assumption does not apply, the evaluated connection moment would be inaccurate. Regions of ‘disturbed’ strain (NZS3101 2006) surround the BC connection interface. From analysis of the stress flow around a BC connection (see Chapter 6), it is evident that the strain gauges placed on the columns near the BC connections are likely to be in a region of disturbed strain. For the beams, the strain distribution appears to be effectively linear. Hence, the data from the column strain gauges is not considered, other than around the column-base connections.

To present the data each connection in the north frame is labeled, as shown in Figure B.2.1. For each test, the connection moment is calculated and plotted against the control drift and the connection rotation. All moments are taken as being positive for positive drifts. It is noted that connection potentiometers were not present around the connections L2-3 and L3-3 (see Figure B.2.1).

The contribution of connection rotation to the total interstorey drift is investigated. The ratio of the drift due to connection rotation, $\theta_{\text{con}}$, and the control drift, $\theta_{D}$, is plotted. Furthermore, an attempt is made to evaluate the depth of the compression region (neutral axis depth) within the BC connections. However, due to shear distortion in the column, the neutral axis depth is difficult to define accurately, as illustrated in Figure B.2.2.
Figure B.2.1. Connection labels on north frame

**Stage 1: Test 1 and Test 2**

Figure B.2.4a, c and d show the moment-drift response of the frame connections in Test 1. Area enclosed within the hysteresis for the Level 3 (L3) connections on the west side (L3-1 and L3-2) indicate inelastic deformation of the timber for positive drifts. As noted previously, the increased inelastic deformation in the west connections is due to the additional axial forces applied by the loading apparatus. The additional axial force appeared to increase and decrease the moment within the connection for positive and negative drifts respectively. However, there is little difference in connection moment between the west and east connections, which may indicate that inelastic deformation has limited the capacity of the connection to some extent. The hysteretic response for Test 2 is given in Figure B.2.4b, d and f. There is little hysteretic energy dissipation at the L3 connections. Hence, most of inelastic deformation appears to have occurred during Test 1.

The moment-drift response of the Level 2 (L2) connections (see Figure B.2.4c and d) varies significantly for each connection. Intuitively, it is expected that the west connections (L2-1 and L2-2) would have a larger capacity than the east connections (L2-3 and L2-4), due to additional axial load applied by the rams, but the opposite is true. It is possible that either the strain gauge data is inaccurate or secondary effects
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have caused additional axial forces in the east bay. This secondary effect could be that axial force from L3 was transferred to the east bay on L2 via shear in the interior column. The higher axial force in the east bay (on L2) could explain the increased moment capacity. Also, each beam-column connection may have had significantly different stiffness, due to alternative fabrication techniques. Half of the columns were manufactured with the internal steel plate arrangement (see Appendix A) epoxied into position. This would have significantly increased the connection stiffness. Additional reinforcement across the BC connections has not provided significant energy dissipation to the frame system on L2 or L3. Refer to Chapter 5, for a discussion on the effectiveness of additional reinforcement.

For Test 1, the column-base connections (see Figure B.2.4e) give a pinched hysteretic response with significant strength loss, indicating that there is anchorage slip and failure of the external reinforcement. For Test 1, the external reinforcement anchorages failed for some connections (most notably B-3), as discussed in the previous section (and illustrated in Appendix C). The connection response partially explains why the frame system has low levels hysteretic damping when compared to predictions (see Chapter 5 for more detail). The exterior connections (B-1 and B-3) show significant variation in moment capacity for positive and negative drifts due variations in column axial forces caused by lateral loading. The column-base connections are more effective for Test 2 (compared to Test 1), as shown in Figure B.2.4f. This is because a pre-tension was applied to the external reinforcement and anchorages were repaired. Yet, the hysteretic energy dissipation provided by the base connections was still much less than predicted (for an elastic-plastic hysteresis), due to pinching (see Chapter 5).

The connection moment versus rotation response is shown in Figure B.2.5. For Level 3, there is much less connection rotation in the positive quadrant (which corresponds to positive drift). This is due to axial forces applied by the rams on the west external columns, which stiffen the connection, resulting in less rotation. For Level 2, the moment-rotation response is variable for each connection and indicates the complex distribution of axial load through the connections or variation in connection flexibility,
as mentioned above. Connections that used non-epoxied internal steel plates exhibit a low initial stiffness. Figure B.2.5c indicates when the ram is in compression, most of the axial force transfers to the east connections increasing the moment and reducing connection rotation. When the ram is in tension, the axial force appears to be highest in the east connections increasing moment and reducing rotation. Comparing Test 1 and 2 (without and with external reinforcement respectively), for Test 2 there is significantly less rotation for both Level 2 and 3 connections due to the presence of additional reinforcement. This is discussed further in Chapter 5.

Figure B.2.6 illustrates the proportion of the connection rotation and total drift ($\theta_{\text{con}}/\theta_D$) is highly variable (20-60%) for each BC connection. Again, this indicates a complex distribution of axial forces and variable connection stiffness throughout the frame. The experimental data indicates that for many connections there is a finite connection rotation at small interstorey drifts, before the connections have decompressed, contrary to existing precast-concrete theories (Palermo 2004; Pampanin et al. 2001). However, the precision of experimental data at such small displacements is questionable. The connection rotation of the column-base connections is approximately 80% of the interstorey drift at 2% drift.

The neutral axis depth (or compression region) is difficult to accurately define due to deformed profile of joint panel (see Figure B.2.2). However, approximate results are given in Figure B.2.7, using linear extrapolation from potentiometers at the centerline and edges of the beams. The neutral axis depth is slightly larger on Level 3 compared to Level 2. The neutral axis depth for the column-base connections is highly variable, due to the performance of the external reinforcement and significant changes axial load. There is no apparent difference in neutral axis depth, for a given connection rotation, for Test 1 and 2. For the Level 2 and 3 connections the minimum neutral axis depth equates to approximately 80mm and 160mm respectively at 2% drift. This corresponds well with observations, as shown in Figure B.2.3.
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Figure B.2.2. Shear distortion of BC joint panel

Figure B.2.3. Photograph of neutral axis for Test 1: a) Level 2 b) Level 3

The connection moments are higher than the predictions (see Appendix A) for the BC connections. Furthermore, for the prediction, the connection deformation was expected to be approximately 60% of the total drift (for Test 1 and 2). Observing the experimental data (see Figure B.2.6); the connection deformation only exceeds 60% for few connections. Hence, for a given connection rotation, the moment capacity of the connections is significantly higher than predicted, especially for Level 3 connections. In addition, the prediction did not capture the significant reduction in connection rotation when the external reinforcement was added. During testing, a pre-load was used for the
energy dissipation to ensure the bars activated under small rotations in tension. This pre-load was not accounted for in the prediction and potentially has increased the shear deformation of the joint panel region.

For the column-base connections, the predictions match relatively well with experimental data, considering that the experimental response was considerably pinched. Furthermore the predictions assumed that the elastic deformation of the column member was insignificant and, hence, the connection rotation was equal to interstorey drift. This assumption gives an error in connection rotation of approximately 20%. Discrepancy between the actual and predicted connection rotation may have also been increased by small amounts of sliding, as well as flexural and shear deformation of the column.

The neutral axis depth was predicted to be 40% and 75% of the beam depth for Level 2 and 3 respectively. The actual neutral axis depth, from experimental data, was significantly less than predicted, giving approximately 30% and 45% for Level 2 and 3 respectively. This further indicates that existing procedures for predicting the connection response (Appendix A) are inaccurate.
Figure B.2.4. Connection moments versus drift for Stage 1: a) & b) L3 BC connections, c) & d) L2 BC connections, e) & f) Column-base connections for Test 1 and 2 respectively
Figure B.2.5. Connection moments versus rotation for Stage 1: a) & b) L3 BC connections, c) & d) L2 BC connections, c) & d) Column-base connections for Test 1 and 2 respectively.
Figure B.2.6. Ratio of connection and total drift for Stage 1: a) & b) L3 BC connections, c) & d) L2 BC connections, e) & f) Column-base connections for Test 1 and 2 respectively.
Figure B.2.7. Neutral axis depth for Stage 1: a) & b) L3 BC connections, c) & d) L2 BC connections, c) & d) Column-base connections for Test 1 and 2 respectively.
**Stage 2: Test 1 and Test 2**

Figure B.2.8 shows the moment-drift response of the frame connections for Test 1 and Test 2 of Stage 2, which include a floor diaphragm. There is little hysteretic energy dissipation for L2 and L3 connections. Hence, the majority of inelastic deformation of the timber within the beam column connections has occurred during Stage 1. Generally, the moment response of the BC connections is very similar to Stage 1, with no increase in strength due to the interaction of the floor. Refer to Chapter 5, for a discussion on the influence of floor on the strength of the frame system.

The moment-drift response of each connection is similar for both Test 1 and 2, but the moment-rotation response varies significantly (as discussed above for Stage 1). Again, the external reinforcement appears to have significantly reduced the connection rotation. Refer to Chapter 5, for further discussion on the effects of energy dissipation.

The column-base connections response is highly pinched (see Figure B.2.8e and f) resulting in minor levels of hysteretic energy dissipation. This indicates that in previous tests there has been inelastic deformation of the anchorage pins embedded within the column which has created slop in the connections. This deformation (see Appendix C) has occurred at only approximately 30% of the shear capacity of the pins (640kN) according to EC 5 (EC5 1994). Using a continuous pin through the column appears to provide unsatisfactory performance for cyclic loading (see Chapter 5).

The connection rotation-drift ratio ($\theta_{\text{con}}/\theta_D$) and the neutral axis depth relationships, shown in Figure B.2.10 and Figure B.2.11, are similar to Stage 1. This further verifies that the presence of the floor diaphragm has little effect on the response of the frame. Note; the neutral axis depth for the column-base connection B-1 could not be defined due to instrument malfunction.
Figure B.2.8. Connection moments for Stage 2: a) & b) L3 BC connections, c) & d) L2 BC connections, c) & d) Column-base connections for Test 1 and 2 respectively
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Figure B.2.9. Connection moments versus rotation for Stage 2: a) & b) L3 BC connections, c) & d) L2 BC connections, e) & f) Column-base connections for Test 1 and 2 respectively
Figure B.2.10. Ratio of connection and total drift for Stage 2: a) & b) L3 BC connections, c) & d) L2 BC connections, e) & f) Column-base connections for Test 1 and 2 respectively.
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Figure B.2.11. Neutral axis depth for Stage 2: a) & b) L3 BC connections, c) & d) L2 BC connections, e) & f) Column-base connections for Test 1 and 2 respectively.
B.2.2. Uni-directional wall tests

Four uni-directional wall tests were performed during Stage 2; Test 3 through 6. The bending moment at the base of the walls was approximated by using potentiometer measurements on the north-west (NW) wall. The curvature over the gauges length is determined, and then multiplied by the elastic modulus (11GPa is assumed) and the second moment of area, giving moment. The moment was determined at two points in the lower part of the wall and projected to the base connection elevation. The moment is plotted versus control drift and connection rotation.

Figure B.2.13a and d shows the wall connection moment versus control drift for the PT only system. The wall appears to respond elastically with little hysteretic energy dissipation. This indicates the hysteretic damping, observable in the global response of the wall system, is derived mostly from friction and cracking of the concrete slab. The experimental response matches well with predictions (see Appendix A).

The moment-drift response of the base connections for the hybrid wall system (Test 4 and Test 5) is given in Figure B.2.13b and c. The moment is slightly higher for positive drifts, due additional axial loads applied by the UFP couplers. Again the connection response matches well with predictions.

Although the predicted connection moment at 2% drift matches well with experimental data, the global OTM of the building (see Figure B.1.3) is significantly under predicted. Therefore, the coupling action provided by the floor diaphragm and edge beams was significant. See Chapter 5 for more detail.

There are some inaccuracies in calculating the rotation at the base of the wall. The deformed profile of the wall section was found to be non-linear from finite element modeling (see Chapter 7) and from experimental data, as shown in Figure B.2.12. From experimental data it was discovered that, depending on which potentiometers (Pots) were used, there was a significant difference in the connection rotation and hence moment-rotation response. Using Pots on the uplifted edge of the wall over-predicted the connection rotation of the wall at the base. A finite element model of the wall found
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that Pots A and D (refer to Figure B.2.12) were most appropriate for determining the centerline rotation of the wall due to connection deformation (see Chapter 6 for more detail).

The moment-rotation response of the connections matches relatively well with predictions (see Figure B.2.14) but the connection rotation is over-predicted. The over prediction could be due to inaccuracies in the calculation of the connection rotation from the experimental response and/or due to sliding at the base of the walls.

The accuracy of the neutral axis calculation is also affected by the displacement profile at the base of the wall (see Figure B.2.12). The calculated neutral axis depth for different Pots, versus connection rotation (based on Pots AD), is plotted in Figure B.2.12c. If Pots AB or CD (depending on whether there are negative or positive rotations) are used the neutral axis depth is significantly over-estimated. If Pots AD are used, it is likely that one of the Pots will be in the compression region. This will not affect the accuracy of the result, provided that compression region of the wall has a linear strain profile (as indicated by FEM analysis in Chapter 6). Hence, for all tests Pots AD are used to determine the neutral axis depth. This approach will at least give comparative results between each test.

At 2% drift, the connection rotation is between 50% and 90% of the control drift (see Figure B.2.15). The connection rotation tends to zero for small Drifts, indicating there is a more clearly defined decompression point (compared to the BC connections). Predictions, which suggest that the connection rotation is 80% of drift, match relatively well with the experimental data. Again, the slight over prediction could be due to wall slip or inaccuracies in calculation of the connection rotation.

The experimental neutral axis depth is computed for the NW and SW walls using potentiometers at the edge of each wall, as shown in Figure B.2.16. Based on the experimental data the neutral axis depth is fairly consistent at approximately 0.17 of the wall length at 2% drift. There is a slight variation in neutral axis depth for positive and negative drifts for Test 4 and 5, where the UFP couplers are present. Because there is little variation in neutral axis depth for Test 3 and 6 (without UFPs), it can be inferred
that the wall-floor coupling result in minor variations in axial force in the walls. The predicted neutral axis depth is 0.35 of the wall length at 2% drift, which varies significantly from the experimental results (that give a neutral axis depth of approximately 0.17 of the wall lengths). It is likely that the disparity is due to inaccuracies in the analytical prediction procedure (see Appendix A).

Figure B.2.12. Interpretation of experimental data for wall connection: a) Deformed shape of wall base b) Moment-rotation using different Pots c) Neutral axis depth using different Pots
Figure B.2.13. NW Wall connection moments versus Drift: a) Test 3 b) Test 4 c) Test 5 and d) Test 6
Figure B.2.14. NW Wall connection moments versus connection rotation: a) Test 3 b) Test 4 c) Test 5 and d) Test 6
Figure B.2.15. Ratio of connection and total drift for NW wall:

a) Test 3  b) Test 4  c) Test 5 and d) Test 6
Figure B.2.16. Normalized neutral axis depth versus connection rotation for NW and SW walls: a) Test 3  b) Test 4  c) Test 5 and d) Test 6
B.2.3. Bi-directional tests

The connection response of the wall and frame connections is considered for the bi-directional test (Stage 2, Test 7). Due to the early termination of the second bi-directional test (Test 8), there is limited data available. Hence, only Test 7 is considered here.

**Frames**

The moment-drift response of the frame connections during the bi-directional test (Test 7) are shown in Figure B.2.17. The BC response was comparable to the uni-directional tests. Notably, the column-base connections provide less moment than in the uni-directional testing. Refer to Chapter 5 for further discussion.

Because 3% peak drift was applied to the structure, the connection rotation was larger than the previous uni-directional tests. With increased connection rotation, BC connections provide a more ‘flag-shaped’ hysteretic shape with significant hysteretic energy dissipation (see Figure B.2.18). However, the connection rotation is still much less than predicted at 2% drift. The moment within the base-connections are effected by in-plane and out-of-plane drift, hence, it is not possible to compute the connection moment versus rotation in the frame direction.

The ratio connection rotation versus drift and neutral axis depth, shown in Figure B.2.19 and Figure B.2.20 respectively, are similar to uni-directional results.
Figure B.2.17. Connection moments versus EW Drift for Stage 2 Test 7: a) L3 BC connections b) L2 BC connections c) Column-base connections

Figure B.2.18. Connection moments versus rotation for Stage 2, Test 7: a) L3 BC connections b) L2 BC connections
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Walls
The moment-drift response of the NW wall is shown in Figure B.2.21. The UFP couplers provide significant hysteretic damping to the moment-rotation response beyond 2% drift. The accuracy of the estimation of the connection moment appears to be affected by out-of-plane deformation of the wall. In general, the response of the wall system is not significantly affected by bi-directional loading. See Chapter 5 for more detail.

The connection rotation and neutral axis could not be accurately defined for bi-directional loading and is not presented here.
Figure B.2.21. NW Wall connection moments versus NS Drift for Stage 2 Test 7
B.2.4. Out-of-plane moments

Frames

The out-of-plane moment provided by the column-base connections is a component of the global strength of the building in the NS direction. Hence, this contribution is quantified to determine by elimination the influence of the floor system on the response of the building.

Strain-gauges have been placed on the north and south face of the columns on the north frame. The strain-gauges are used to determine the moment provided by the column-base connections, acting out-of-plane, at 2% drift. The average of the column-base moments for the north frame is calculated and plotted in Figure B.2.22 against the interstorey drift for each of the wall tests (Stage 2: Test 3 though 6). The strain-gauges lack the precision to determine the axial force in each column, with which the coupling action provided by the floor and the frame could be quantified.

The moment contribution from the base connections is relatively minor. However, the connection moment shows that there is significant coupling action between the frame and the floor system; for positive drifts the moment is significantly higher than for negative drifts, due to increased axial forces (in the north frame) due to coupling action of the floor. The OTM provided by the base connections at 2% drift can be approximated by taking the average moment for positive and negative base moment, which is approximately 10kN.m per column, and multiplying by the number of columns. Hence, approximately 60kN.m OTM is provided by the column-base connections at 2% drift.
Figure B.2.22. Average out-of-plane moment provided by the column-base connections on the north frame in Stage 2: a) Test 3 b) Test 4 c) Test 5 d) Test 6
Appendix B – Further detail on the experimental response of the test building

Walls

The out-of-plane moment provided by the walls is estimated by using potentiometers on the east and west faces of the NW wall. The moment at the base of the wall is projected from the centerline of the potentiometers assuming a linear moment profile. This moment contribution can be subtracted from the total OTM of the building in the EW direction, to determine influence of the floor diaphragm on the building strength (see Chapter 5). The total overturning moment (OTM) contribution provided by the wall acting out-of-plane is approximately 80kN.m at 2% drift.

The axial forces induced by floor-to-wall coupling could not be reliably quantified (other than by elimination) from the experimental data. This mechanism may have provided significant strength to the building, responding the EW direction.

Figure B.2.23. Out-of-plane moment provided by walls in Stage 2: a) Test 1 b) Test 2
B.3. TENDON FORCES

The tendon forces are monitored using load cells on the tendons during the uni-directional and bi-directional tests. Two load cells were used for each tendon group. The load cells were placed on the top and bottom tendons going through the frames, and the north and south tendons going through the walls. These tendons were expected to give the peak tendon force in each group.

B.3.1. Uni-directional frame tests

The tendon forces for Stage 1 (Test 1 and 2) versus drift are shown in Figure B.3.1. For Level 3 tendons, the post-tensioning (PT) force is significantly different for positive and negative drifts. This is due to additional compressive forces applied to the frame by the loading apparatus. For positive drifts, the loading apparatus applies a compressive force, which results in shortening of the frame and loss of post-tensioning force. The reverse is true for negative drifts. For Test 1 (Figure B.3.1a), there is irrecoverable tendon losses because of inelastic deformation of the BC connections adjacent to the application of lateral load. During Test 2, there is no significant loss of PT force, further verifying that there was little further inelastic deformation within the BC connections after Test 1. For the Level 2 tendons, there is no significant variation in force for positive and negative Drifts, or losses in PT force, due to the higher axial stiffness and strength of the BC connections.

For Stage 2, the tendon forces are similar of positive and negative drifts on both Level 2 and 3. This indicates that the Stage 2 loading apparatus effectively distributes lateral forces throughout the structure. Between Stage 1 and 2 there have been slight losses of PT force on Level 3 (due to creep) but effectively none of Level 2. Refer to Neale (2009) for further detail.

For the predictions, it was expected that the tendon forces on Level 2 and 3 would be approximately 110kN and 93kN respectively at 2% drift. Both predictions over-estimate the force in the tendons slightly. This may be because inaccuracies in the
Appendix B — Further detail on the experimental response of the test building

analytical predictions procedures. Namely, because the axial deformation of the members was not taken into account, the predicted force within the tendons was higher. In addition, for both Stage 1 and 2, there is no significant difference in PT force with and without external reinforcement.

![Graph showing forces in tendons for stages 1 and 2](image url)

**Figure B.3.1.** Force in the post-tensioning tendons for stage 1: a) & b) Level 3 tendons, c) & d) Level 2 tendons for Test 1 and 2 respectively
Figure B.3.2. Force in the post-tensioning tendons for stage 2: a) & b) Level 3 tendons, c) & d) Level 2 tendons for Test 1 and 2 respectively

B.3.2. Uni-directional wall tests

Figure B.3.3 shows the forces in the north and south tendons of the NW and SW walls for Tests 3 through 6. During Test 3, there are slight losses in PT force, indicating limited inelastic deformation of the timber. Losses successively reduce for subsequent tests. For the post-tensioned (PT only) walls, which correspond to Test 3 and 6, the PT forces are similar for positive and negative drifts. For Hybrid walls, Test 4 and 5, there
are variations in PT forces between corresponding tendon positions in the north and south walls. This is due to the affects of additional axial load from the UFP couplers. Furthermore to PT force remains higher during unloading for some tendons due to resistance provided by the UFP couplers. The peak PT force was predicted to be approximately 110kN at 2% drift. This corresponds well with experimental results.

Figure B.3.3. Force in the post-tensioning tendons within the NW and SW walls: a) Test 3 b) Test 4 c) Test 5 d) Test 6
B.3.3. Bi-directional test

The forces in the frame and walls tendons are plotted in Figure B.3.4 and Figure B.3.5 respectively for the bi-directional test (Stage 2, Test 7).

For the frames, there are slight losses in PT force after 2% drift, which indicates further inelastic deformation, especially on Level 3. The peak PT force is approximately 107kN, which is well within the elastic range of the tendon and significantly less than predicted (125kN).

For the walls, the PT forces are similar for all walls except the north tendon in the NE Wall. The initial PT force was higher than specified. Hence, the peak tendon force is dictated by this tendon at 140kN. This PT force is 10% less than the non-linear limit of the tendon, and 23% less than the ultimate tensile strength of the tendon.
Figure B.3.4. Force in the frame post-tensioning tendons for Test 7: a) & b) Level 3 tendons, c) & d) Level 2 tendons for North and South frames respectively.
Figure B.3.5. Force in the wall post-tensioning tendons for Test 7: a) NW Wall b) NE Wall c) SW Wall d) SE Wall
B.4. FRAME ELONGATION

The longitudinal elongation/shortening deformation of the frame was recorded using potentiometers, as shown in Figure B.4.1. The total elongation is plotted for each uni-directional frame test from Stage 1 and 2. The elongation/shortening during each test is examined, dividing the total deformation into components. In all graphs, shortening is positive and elongation is negative.

![Figure B.4.1. Potentiometers for measuring longitudinal frame deformation](image)

**Figure B.4.1. Potentiometers for measuring longitudinal frame deformation**

B.4.1. Uni-directional tests

Figure B.4.2 shows the total frame elongation/shortening of Level 2 and 3 for Stage 1, Test 1 and 2. The longitudinal deformation at Level 3 differs for positive and negative drifts, due to application of lateral load from the test apparatus. For negative drifts, the rams are in tension, resulting in increased elongation of the frame. For Test 1, inelastic deformation in the west exterior BC connection results in permanent frame shortening of 3 to 4mm. For Test 2, there is no further permanent shortening. When the ram is in tension, the maximum elongation on Level 3 is 4mm. On Level 2, there is no significant inelastic shortening. A peak elongation of 4mm occurs during Test 1. For Test 2, due to the presence of external reinforcement, the connection rotation is reduced, resulting in less elongation (approximately 2.5mm).
The longitudinal deformation for the frame during the Stage 2 tests (including the floor system) is shown Figure B.4.3. Unlike Stage 1, the longitudinal deformation at Level 3 is similar for positive and negative drifts due to the Stage 2 loading apparatus. The longitudinal deformation of both Level 2 and 3 is similar to that of Stage 1, indicating the floor system did not have a significant affect on the frame elongation. This is because the majority of frame elongation has occurred at the exterior connections which are not connected to the floor diaphragm (as discussed further below).

Differential longitudinal deformation on Level 2 and 3 may have resulted in additional axial forces in the beams. Using simple beam theory and the method of superposition, the worst-case differential elongation of 6mm will result in approximately 40kN compression and 20kN tension at Level 2 and 3 respectively. The increased compressive force would have contributed to the moment capacity of the Level 2 connections (see Figure B.2.4).
Figure B.4.2. Frame elongation for Stage 1: a) & b) Level 3, c) & d) Level 2 for Test 1 and 2 respectively
Figure B.4.3. Frame elongation for Stage 2: a) & b) Level 3, c) & d) Level 2 for Test 1 and 2 respectively.
The longitudinal deformation is deaggregated into the localized deformation around the exterior and interior BC connections, and the perpendicular-to-grain timber on Level 3, as shown in Figure B.4.1.

For Stage 1 (see Figure B.4.4) the most significant elongation on Level 2 occurs in the exterior connections for both Stage 1 and 2. There is a significantly less longitudinal deformation for Test 2 (Stage 1) due to reduced connection rotation (because of the presence of external reinforcement). On Level 3, the most significant elongation occurs within the exterior BC connections, but this is offset by the permanent compressive deformation in the perpendicular-to-grain timber within the columns. For Test 2, the permanent compressive deformation almost completely offsets the elongation due to connection rotation. There is significantly more elongation in the exterior connections for negative drifts due to forces applied by the experimental apparatus.

The elongation of the interior connections appears to have reduced for Stage 2, compared to Stage 1 on both Levels 2 and 3. On Level 3, this could be attributed to the restraint provided by the floor diaphragm, the different load apparatus for Stage 1 and 2 or inelastic deformation within the connections. However, if the average elongation of the interior connections for positive and negative from Stage 1 is taken, this value compares well with Stage 2. This would indicate the differential elongation in the interior connections is caused by the loading apparatus, and not the restraint provided by the floor. For Level 2, there appears to be slightly less elongation of the internal BC connections for Stage 2. However, any restrain does not appear to have significantly affected the moment resistance provided by the connections.
Figure B.4.4. Frame elongation for Stage 1: a) & b) Level 3, c) & d) Level 2 for Test 1 and 2 respectively.
Appendix B – Further detail on the experimental response of the test building

Figure B.4.5. Frame elongation for Stage 2: a) & b) Level 3, c) & d) Level 2 for Test 1 and 2 respectively
B.4.2. Bi-directional test

The total longitudinal deformation of the frame was monitored during the bi-directional test (Test 7), as shown in Figure B.4.6. For the uni-direction part of the bi-directional loading protocol, the elongation is similar to the previous uni-directional frame test up to 2% drift (Stage 2, Test 2). However, under bi-directional loading (during the clover part of the loading protocol) the elongation appears to slightly increase. For Level 3, bi-directional loading appears to increase in the elongation of the frame by 1mm at 2% drift and 2.5mm at 3% drift. For Level 2, the variation between uni-directional and bi-directional cycles is less with 1.5mm at 3% drift. Furthermore, for the NS uni-directional cycles there is an elongation of approximately 1.5mm and 2.5mm during the 3% drift cycle for Level 2 and 3 respectively. This indicates that elongation of the frame is slightly increased by bi-directional loading, which could be caused by bowing of the frame out-of-plane. As the load is applied to the floor diaphragm in the NS direction, it deflects in-plane (like a simply-supported beam), which results in elongation on the frame in the orthogonal direction. Using simple beam theory and assuming a crack section stiffness of 20% of the gross section stiffness for the concrete slab (from Chapter 3), the peak out-of-plane deformation of the floor system is estimated to be 2.5mm on Level 3. Converting the elongation in the NS direction to curvature, the estimated peak displacement of the floor on Level 3 is 7mm. Hence, it is feasible that significant out-of-plane floor deformations have occurred and the calculations would indicate that the stiffness of the floor diaphragm is significantly less than the gross section stiffness.

According to method of superposition, differential elongation of L2 and L3 at 3% drift of 8mm may have resulted in additional compressive and tensile axial forces in the L2 and L3 beams of approximately 50kN and 25kN respectively.
Figure B.4.6. Frame elongation for Stage 2, Test 7: a) Level 3 b) Level 2
B.5. FLOOR SLAB ELONGATION

The floor slab deformations were monitored using a calibrated ‘Demec Gauge’ and ‘Demec Points’, as shown in Figure B.5.1. The total elongation of the slab, as determined by summing all the Demec Gauge readings on one line, is given in Table B.8.1. The elongation is deaggregated into the individual readings along the EW (or X) axis of the building (see Figure B.5.1) in Figure B.5.2 and Figure B.5.3 for Test 1 and Test 7 respectively.

![Figure B.5.1. Demec points on floor slab](image)

The peak elongation of the slab over its entire length during uni-directional Test 1 at 2% drift is 1.3mm, with a peak residual deformation of the floor is 0.2mm. Even if the total deformation of the slab was concentrated in one crack, according to FEMA-356 (2000), the slab would not required repair (under immediate occupancy criteria). Observing Figure B.5.2, it is evident that there is significantly more floor slab elongation for negative drifts than positive drifts. This is due to the application of lateral load by the test apparatus. For negative Drifts, the slab is predominately in tension, exacerbating elongation. Most of the deformation that occurs for negative drifts is concentrated around the walls, where the there is deformation incompatibilities between the floor and wall systems, as the building undergoes lateral drift. The residual deformation after Test
Appendix B – Further detail on the experimental response of the test building

1 is relatively uniform, with many areas that have shortened, offsetting areas that have elongated.

Under bi-directional loading (see Table B.5.1 and Figure B.5.3), the residual elongation after 2% drift is larger than for uni-directional loading. This is likely to be due to additional cracking that has occurred due to out-of-plane loading. The most significant residual deformation is adjacent to the walls (at the edges of the floor). The peak residual elongation of the slab after 3% drift is 1.0mm. Again, even if the recorded peak deformation (over a 500mm gauge length) was localized in one crack, the crack would not require repair according to FEMA-356 (2000).

<table>
<thead>
<tr>
<th>Test</th>
<th>Description</th>
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<th>Line 2</th>
<th>Line 3</th>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 1</td>
<td>+2.0% Drift</td>
<td>-0.10</td>
<td>0.14</td>
<td>-0.05</td>
</tr>
<tr>
<td></td>
<td>-2.0% Drift</td>
<td>0.75</td>
<td>1.25</td>
<td>0.88</td>
</tr>
<tr>
<td></td>
<td>Residual</td>
<td>-0.04</td>
<td>0.15</td>
<td>0.07</td>
</tr>
<tr>
<td>Test 7</td>
<td>Residual after 2% Drift cycle</td>
<td>-0.68</td>
<td>0.69</td>
<td>-0.60</td>
</tr>
<tr>
<td></td>
<td>Residual after 3% Drift cycle</td>
<td>0.18</td>
<td>1.82</td>
<td>0.15</td>
</tr>
</tbody>
</table>
Figure B.5.2. Elongation of the concrete slab for Test 1: a) & b) Positive 2% Drift, c) & d) Negative 2% Drift, e) & f) Residual for Level 2 and 3 respectively
Figure B.5.3. Elongation of the concrete slab for Test 7: a) & b) Residual after 2% Drift, c) & d) Residual after 3% Drift, for Level 2 and 3 respectively.
B.6. COLUMN AND WALL INCLINATION

Inclinometers were placed on all of the columns and two of the walls. Each was fixed at the base and at the top of element (in line with L3). The inclinometers have relatively low precision (compared to Potentiometers) but are useful to determine if the rotation of each of the walls and columns are similar. The inclination of the columns or walls can be reduced due to base slippage, which can influence the capacity of the frame and wall systems. Conversely, the inclination of the columns can be increased due to beam elongation.

B.6.1. Uni-directional tests

The ratio of the column and wall rotations and the control Drift ($\theta/\theta_d$) for the uni-directional tests are shown in Figure B.6.1. The data precision at small rotations is poor. For larger rotations, the inclination of the columns and walls tests, on average, tends to approximately 95% of the control drift (as defined in Chapter 5). This indicates that the slippage of column bases is minor. Figure B.6.3 shows the column-base slippage for four of the columns for uni-directional tests in the frame and wall direction. The peak slip is approximately 2mm at 2% drift, which equates to 0.05% drift. Hence, column-base slip had little effect on the inclination of the columns. Furthermore, there is no discernible difference between the inclination of the east and west columns. Hence, the outer columns have not splayed due to beam elongation, further indicating that beam elongation was negligible.

For the walls tests, there appears to more variation between the inclination of the walls and the control drift, as illustrated in Figure B.6.1c and d. The variation may be more significant for Test 4, which includes the UFP couplers, due to wall slip at base. It was observed (see Appendix C) that the walls tended to splay at the base due to wedging actions created by the UFP couplers.

To investigate further, the control drift and the inclination of both the columns and walls for the 2% drift cycles are shown in Figure B.6.2. Figure B.6.2a and b. Again, the...
Inclinations of each column for the uni-directional frame tests are similar to the control drift. There is slightly more instability in the inclination readings for the wall tests but generally good agreement with the control drift. Hence, splaying at the base of the walls does not appear to have been significant.

Figure B.6.1. Wall and column inclination versus Drift of the control column (every 40th data point): a) Stage 1, Test 1 b) Stage 2, Test 1 c) Stage 2, Test 3 d) Stage 2, Test 4
Figure B.6.2. Wall and column inclination for 2% Drift cycles: a) Stage 1, Test 1 b) Stage 2, Test 1 c) Stage 2, Test 3 d) Stage 2, Test 4 (refer to Figure B.6.1 for legend)
Figure B.6.3. Column-base slip for uni-directional tests a) Stage 1, Test b) Stage 2, Test 1c) Stage 2, Test 3 (NS direction) d) Stage 2, Test 4 (NS direction)
B.6.2. Bi-directional tests

For the bi-directional tests, the inclination of the columns and walls is in general similar to the control drift. While there is imprecision in inclination recordings, the ratio of the column and wall rotation and the control drift ($\theta/\theta_D$) tend to unity for both EW and NS loading, as shown in Figure B.6.4. Observing the inclination for columns and walls over the 3% drift cycle (see Figure B.6.5), there is no significant deviation from the control drift, except for the SE column. This is because target drifts were not imposed to the SE column in the loading methodology (refer to Chapter 5). The inclinometer on the SW Wall for EW loading was determined to be faulty and hence does not appear in the figures below.

![Figure B.6.4. Wall and column inclination versus control Drift for Stage 2, Test 7 a) EW direction b) NS direction](image-url)

Figure B.6.4. Wall and column inclination versus control Drift for Stage 2, Test 7 a) EW direction b) NS direction
Figure B.6.5. Wall and column inclination for 3% Drift cycles for Stage 2, Test 7: a) EW direction b) NS direction (refer to Figure B.6.4 for legend)
B.7. INVESTIGATION OF TIMBER PROPERTIES

Within this section, experimental data is considered in an attempt to better characterise the material properties of the timber. Key parameters that are investigated are the parallel and perpendicular-to-grain elastic modulus and the shear modulus.

B.7.1. Parallel-to-grain timber stiffness

The elastic modulus of timber parallel-to-grain was characterized using factory test data (Banks 2010), but can be verified by considering experimental data. This is done by considering the axial elastic deformation of the beams and walls that occurred during the application of post-tensioning, as expressed below:

\[ E = \frac{F_{pt} L_g}{\Delta_i A} \]

Where:
- \( F_{pt} \) = the axial force applied by the post-tensioning
- \( A \) = the area of the beam (84800mm\(^2\)) or wall (91200mm\(^2\))
- \( L_g \) = the gauge length of the Pots
- \( \Delta_i \) = the longitudinal displacement of each Pot

Deformation data is provided by potentiometers that are placed longitudinally in the centerline of the beams in the south frame and down either side of the NW wall (see the appended compact disc). The post-tensioning force is determined by using data from load cell placed on the some of the outer tendons and assuming the inner tendons forces are similar.

The experimental data indicates that the elastic modulus of the timber is approximately 12GPa (see Table B.7.1). The data gives values between 10 and 16GPa, indicating that there are some experimental inaccuracies. Hence, for calculation of moments, in the preceding sections, the manufacture specified elastic modulus of 11GPa was used, which is within 15% of the calculated average.
Table B.7.1. Estimated parallel-to-grain Elastic Modulus from experimental data

<table>
<thead>
<tr>
<th>Location</th>
<th>Pots</th>
<th>$A$ (mm$^2$)</th>
<th>$L_g$ (mm)</th>
<th>$\Delta l$ (mm)</th>
<th>$F$ (kN)</th>
<th>$E$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NW Wall</td>
<td>17,18,20,21</td>
<td>91200</td>
<td>595</td>
<td>0.210</td>
<td>444</td>
<td>13794</td>
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<tr>
<td>SW Wall</td>
<td>12,13</td>
<td>91200</td>
<td>598</td>
<td>0.239</td>
<td>453</td>
<td>12428</td>
</tr>
<tr>
<td>L2-W Beam</td>
<td>163</td>
<td>84800</td>
<td>3390</td>
<td>1.442</td>
<td>371</td>
<td>10285</td>
</tr>
<tr>
<td>L2-E Beam</td>
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<td>84800</td>
<td>3401</td>
<td>1.259</td>
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<td>11818</td>
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<tr>
<td>L3-W Beam</td>
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<td>0.953</td>
<td>368</td>
<td>16074</td>
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<td>L3-E Beam</td>
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<td>1.351</td>
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<td><strong>Average</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td><strong>12618</strong></td>
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</table>

B.7.2. Perpendicular-to-grain timber stiffness

The perpendicular-to-grain (perp-to-grain) elastic modulus is difficult to accurately define using experimental data from the test building. While the Level 3 columns were subjected to compression perp-to-grain, the column fibers at the top and bottom of the beam are subjected to tension, which increases the bearing stiffness. Furthermore, the diffusion of compressive stresses into the column and the presence of screw reinforcement may have affected the perceived perp-to-grain elastic modulus of the column. Newcombe (2008), Davies and Fragiacomo (2008), provide some detail on the actual perp-to-grain elastic modulus, without bearing effects at the edges. Newcombe (2008) suggests the perp-to-grain elastic modulus is as low as 300MPa. This is considered as a lower bound, for comparison with experimental data.

The above points notwithstanding, the experimental perp-to-grain stiffness of the column is still important for modeling the frame response (see Chapter 6). The average calculated perp-to-grain elastic modulus is approximately 660MPa. Hence, the combined effect of edge-bearing, diffusion of stresses and screw reinforcement appears to have effectively increased the perp-to-grain stiffness of the timber, according to Newcombe (2008), by approximately 100%.
<table>
<thead>
<tr>
<th>Location</th>
<th>Pots</th>
<th>A  (mm$^2$)</th>
<th>$L_g$ (mm)</th>
<th>$\Delta l$ (mm)</th>
<th>F  (kN)</th>
<th>E  (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L3-SW Col</td>
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<td>84800</td>
<td>184</td>
<td>0.996</td>
<td>368</td>
<td>802</td>
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</tbody>
</table>

Average: 659

B.7.3. Shear stiffness

Similar to the parallel-to-grain elastic modulus, the shear modulus may be larger than the value specified by the manufacture (Futurebuild 2010). For the LVL in the test building the manufacturers specify a shear modulus of 660MPa. Here an attempt is made to verify the manufacture specified shear modulus by using experimental data from potentiometers placed on the walls, as shown in Figure B.7.1 below.

![Shear deflection of wall](image)

The shear distortion, $\gamma$, can be calculated for the displacement of Pots 1 and 2, $\delta_1$ and $\delta_2$, and the angle of the Pots, $\theta$, as follows:

$$\gamma = \frac{\delta_1 - \delta_2}{2L_g} \left( \tan \theta - \frac{1}{\tan \theta} \right)$$

The shear modulus can be defined as:

$$G = \frac{V}{\gamma A_s}$$
Applying simple beam theory, the shear area, $A_s$, of the wall sections have been calculated as a function of the total area, $A_{tot}$. Hence:

$$A_s = 0.85 A_{tot}$$

The wall shear force, $V$, is difficult to accurately define from the experimental data. Some shear resistance will be provided by the columns, acting out of plane. However, the previous estimations of the out-of-plane moment capacity of the frame system indicate the shear resistance provided by the columns is relatively small (approximately 20kN in total) which is approximately 5% of the base shear of the building at 2% drift. Furthermore, due to coupling of the wall and floor system, it is possible that there will be differential shear force in each adjacent wall. To account for the variable in shear resistance provided by each wall the shear deformation of both the NW and SW walls was considered. The shear modulus is then calculated using an average shear force. Taking an average of the calculated shear modulus of the NW and SW wall should provide a good estimate of the actual shear modulus.

The shear modulus is calculated for Test 3 (Stage 2) for all drift values and is plotted in Figure B.7.2. While the precision of the data is relatively poor, the shear modulus appears to be slightly larger than the value specified by LVL producers (Futurebuild 2010) of 660MPa. Taking into account the shear capacity provided by the columns, the shear modulus appears to be between 690MPa and 920MPa. At higher drifts, the accuracy of the calculated shear modulus is expected to be affected by the disturbed strain region at the base of the walls. Hence, for modeling (see Chapter 6 and 7) the shear modulus will be taken as 660MPa.
Figure B.7.2. Estimated shear modulus versus Drift (Stage 2, Test 3)
B.8. FLOOR SAGGING DEFORMATION

The floor deformations were monitored using a laser measurement device until the end of testing to ensure the loading apparatus was in alignment and that floor had not buckled due to the in-plane loading. A description of the measurement points for the laser is given in the appended compact disc.

The out-of-plane deformations of the floors are given in Table B.8.1. The seismic tests caused up to +/- 5mm deformation of the floor. The maximum sag for the 1st and the 2nd floor was 14mm and 15mm respectively, which is approximately equal to the span over 300. These values are slightly larger than the (10mm) predicted value by effective modulus approach in Appendix A. Hence, some of the floor distortions are like to be due to the seismic testing and loads applied by the testing apparatus.
Table B.8.1. Floor deflection measurements

<table>
<thead>
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<th>Days after pour</th>
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<th>Joint 2</th>
<th>Joint 3</th>
<th>Joint 4</th>
<th>Joint 5</th>
<th>Joint 6</th>
<th>Joint 7</th>
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Notes:
- PB = Floor 1 and 2, holes drilled in floor.
- PR = Floor 3 to 10.
- N/A = Not applicable.

Steel plates and square hollow sections on floors.
- Splitter beams and rams added.
- Last cycle in frame test at 2.0% drift.
- Last cycle in frame test at 0.5% drift.
- After frame test.
- After final test up to 3.0% bidirectional.
APPENDIX C
EXPERIMENTAL OBSERVATIONS FOR THE TEST BUILDING

In this appendix experimental observations of the test building during each earthquake simulation are recorded using photographs and illustrations. The observations are categorized by those pertaining to the frames, walls, gravity system and floor slab.

C.1. FRAME SYSTEM

C.1.1. Stage 1: Test 1

The frame response at 1% drift was essentially elastic. The only observable inelastic deformation occurred at the west beam column connections on Level 3. At 1.5% drift, the connections appeared to remain within the elastic range. However, localized bearing deformations were observed on the Level 3 connections, which may or may not have been elastic.

At 2% drift, still the frame responded in an essentially elastic manner. However, for the west-exterior beam-column connections on Level 3, bearing failure within the columns (loaded perpendicular-to-gain) became pronounced. Timber crushing and bulging was observed, for positive 2% drift (when the rams were in compression).
Figure C.1.1. Frame at 1.0% Drift a) 3D view b) S column

Figure C.1.2. Frame connections at 1.5% Drift a) S column L2 b) S column L3 c) SE column L2 d) SE column L3
Appendix C – Experimental observations for the Test Building

Figure C.1.3. Frame at 2.0% Drift a) 3D view b) S column

Figure C.1.4. Exterior columns at 2.0% Drift a) SW b) SE
The beam-column connection gap openings were larger on Level 2 than Level 3. At 2% drift the gap opening was approximately 5mm and between 0 to 2mm for Level 2 and 3 respectively.

During Test 1, the anchorage of the external reinforcement at the column-bases failed at some locations. Welds between steel foundation and anchorage plates fractured, causing a significant reduction in the strength of the frame (see Appendix B).

Screws that were used to provide temporary fixing between the beams and columns during construction were left in-place during testing. The ‘Type 17’ screws accommodated the deformations at the beam-column connections without fracturing. Hence, these fasteners could also be used for de-construction.
Appendix C – Experimental observations for the Test Building

Figure C.1.6. BC connection gap opening at 2% Drift a) SW column L2 b) NE column L3 c) NW column L3 d) N column

Figure C.1.7. Failure of column-base reinforcement anchorages
C.1.2. Stage 1: Test 2

For Test 2, external reinforcement was connected between the beams and columns. To minimize slop in the connections, the reinforcement was pre-loaded. Hence, the outer nuts were fastened first, to apply a pre-tension (of approximately 120N.m of torque), and then the inner nuts were tightened.

At 2% drift the frame deformation appeared similar to Test 1. On Level 2, the external reinforcement was subjected to elongations in the order of 2 to 5mm. On Level 3, the elongation of the reinforcement appeared to be negligible due to little gap opening.
Appendix C – Experimental observations for the Test Building

As shown in figures below, the Level 2 connections deformed differently. Columns that were manufactured by Hunter Laminates Nelson Ltd had the internal steel plate reinforcement that was epoxied in-place, while MacIntosh Timber Laminates Ltd did not. Hence, internal steel plate reinforcement that was not expoxied in-place within the column rotated relative to the column.

Figure C.1.10. Frame at 2.0% Drift a) 3D view b) S column

Figure C.1.11. L2 BC connection gap opening at 2% Drift a) SW column b) S column
Figure C.1.12. L3 BC connection deformation at 2% Drift a) & b) S column c) & d) SE column

Figure C.1.13. Damage to west columns on L3 at 2% Drift due to loading apparatus a) NW column b) SW column
Appendix C – Experimental observations for the Test Building

Figure C.1.14. Columns at negative 2% Drift a) S column b) SW column

Figure C.1.15. BC connection deformation at negative 2% Drift a) SW column L2 b) SE column L2 c) NE column L3 d) SW column L3
At 2% drift, the column-base connections appeared to uplift by approximately 10mm at the unloaded edge of the column. The failure of the external reinforcement anchorages (discussed for Test 1) was noticed during Test 2. Hence, the anchorages were repaired and Test 2 was repeated.

Figure C.1.16. Column-base connections at negative 2% Drift a) NE column b) SW column
C.1.3. Stage 2: Test 1

The frame deformation was similar with the timber-concrete composite floor (Stage 2). No further inelastic deformation appeared to occur within the beam-column connections and gap openings were similar to Stage 1. Note: all external reinforcement shown in the figures below is loose and hence did not contribute the response of the frames.

![Figure C.1.17. Frame at 2.0% Drift a) 3D view b) S column](image1)

Figure C.1.17. Frame at 2.0% Drift a) 3D view b) S column

![Figure C.1.18. L2 BC connection gap opening at 2% Drift a) S column b) S column, east side](image2)

Figure C.1.18. L2 BC connection gap opening at 2% Drift a) S column b) S column, east side
Figure C.1.19. L3 BC connection deformation at 2% Drift for west exterior columns a) SW column, looking up b) SW column, looking down c) NW column, top d) NW column, bottom
Appendix C – Experimental observations for the Test Building

Figure C.1.20. L3 BC connection deformation at 2% Drift a) N column b) NE column

Figure C.1.21. BC connection deformation at negative 2% Drift a) S column, L2 b) SE column, L2 c) S column, L3 d) SW column, L3
The deformation of the edge-joists (used to connect the floor diaphragm to the beams) was monitored during testing. It was shown that there was no significant deformation between the edge-joist and the beam. The position of the edge-joist, relative to the beam, was marked at 1.0% drift. At negative 2% drift the edge joist remained in the same location.
C.1.4. Stage 2: Test 2

The observed response of the frame with external reinforcement and floor diaphragm was similar to Test 2 from Stage 1. Hence, the floor did not have a noticeable effect.

Figure C.1.24. Frame at 2.0% Drift a) 3D view b) S column

Figure C.1.25. External BC connection reinforcement a) S column, L2 b) S column, L3
C.1.5. Stage 2: Test 7

Under the bi-directional loading protocol the frames were subjected to a peak drift of 3% in both the NS and EW direction. The frame is shown in the figures below at the peak bi-directional drift, which is slightly less than 3% drift in both the EW and NS directions.

The beam-column connections appear to respond purely in-plane with the frame. Hence, the connections do not appear to be affected by out-of-plane loading. The gap openings in the beam-column connections were more pronounced (than at 2% drift), resulting in significantly reduce neutral axis depths on Level 3.

The south column fractured at the Level 3 beam-column connection during the second clover at approximately negative 3% drift in the NS and EW direction. The fracture continued to propagate under reversed loading (see Appendix B and Chapter 5 for further discussion).

Cracking was also observed on south column at Level 2. This was caused by the steel pins within the column. The pins were connected to the internal steel plate reinforcing within the column. As the steel plates rotated relative to the column timber, the pins applied high shear stresses to the timber resulting in a localized shear cracking.
Figure C.1.26. Frame and wall at peak bi-directional Drift a) 3D view b) Close 3D view
Figure C.1.27. BC connection deformation at peak bi-directional Drift a) S column, L2 b) NE column, L3 c) N column, L3 d) NW column, L3
Figure C.1.28. S column failure at 3% Drift: a) & b) Looking north c) Looking south

Figure C.1.29. S column failure at negative 3% Drift
The gap opening of the unloaded edge of the column base at 3% drift was approximately 14mm. The steel plate shear keys at the base of the columns performed as designed. The plates bent above the weld allowing the column to rotate. The perpendicular-to-grain compressive strength of the timber was exceeded, causing minor amounts of crushing.

Significant inelastic deformation of the column-base anchorage was observed during testing. Based on experimental data, it is apparent that this deformation also occurred.
Appendix C – Experimental observations for the Test Building

during the uni-directional tests (see Appendix B). This was not expected because the design axial capacity of the pins was much larger than required (see Appendix A). It appears that uneven tensile forces in the exterior reinforcement resulted in exceedence of the embedment strength at each edge of the pin holes, progressively increasing the slop in the connection. Under bi-direction loading, this mechanism would have been exacerbated, further increasing slop within the base connections.

Figure C.1.32. Column-base external reinforcement anchorage pin deformation at 3% Drift: a) & b) SE column
C.1.6. Stage 2: Test 8

For Test 8 the 3% drift cycle was attempted, without external reinforcement across the beam-column connections. However, only two complete clovers in the loading protocol were achieved until the column-base connections on the SE column failed (see Appendix B for further details). No further damage to the frame was apparent.

The response of the beam-column connections appeared to be similar with and without external reinforcement.

During this test it was noticed that the flexural deformation of the column out-of-plane was significant.

The external reinforcement at the column-based failed due to low cycle fatigue after being subjected to eight tests up to 2% drift and one test up to 3% drift. For all of these tests the strain within the steel did not exceed 5%.

Figure C.1.33. Frame and wall at peak bi-directional Drift
Figure C.1.34. BC connection deformation at peak bi-directional Drift a) S column, L2 b) NE column, L3 c) NE column, L3, close up d) NW column, L3
Figure C.1.35. Column deformation at peak bi-directional Drift a) S column, in-plane b) NE column, out-of-plane
Figure C.1.36. Column-base external reinforcement failure on SE column
C.1.7. The frame after testing

The damage to column was investigated further after testing was complete. The pin-holes at the column-base had elongated from a diameter of 60mm to 64mm, accounting for the connection slop (apparent in hysteresis from Appendix B).

The column failure appeared to initiate around the heads of the screws used as diagonal and horizontal reinforcement. The diagonal screws appeared to be relatively ineffective at reinforcing the timber against forces induced by the external reinforcement anchorage pin.
Figure C.1.37. Column damage after Testing: a) Pin-holes at column-base for external reinforcement b) Column fracture, looking up c) Column fracture, pulling apart d) Column fracture, looking down
C.2. WALL SYSTEM

C.2.1. Stage 2: Test 3

The observed wall response was almost entirely elastic at 2% drift.

There was some minor perpendicular-to-grain crushing at the wall-base due to shear restraint applied by the shear keys.

The edge beams supports, required for construction (and gravity loading), were subjected to significant compressive loads. Because the edge beam is fixed to each wall, at one end the beam uplifted off the column support, while at the other reacted against the column support. In some cases, the cantilevered steel plate gravity supports began to lever off the edge beam. The uplifted end pulled Type 17 screws from the corbel.

The uplift at the unloaded edge of the base of the wall was in the order of 10mm at 2% drift.

![Figure C.2.1. Wall at 2.0% Drift](image_url)

Figure C.2.1. Wall at 2.0% Drift a) 3D view b) E walls
Appendix C – Experimental observations for the Test Building

Figure C.2.2. Edge beam supports at 2.0% Drift a) NE support b) NE support c) SW support d) NW support

Figure C.2.3. Wall at 2.0% Drift a) West walls b) Wall rotation relative to slab
Figure C.2.4. Wall-base at 2.0% Drift a) SE wall, south edge b) NE wall, south edge

Figure C.2.5. Edge beam supports at negative 2.0% Drift a) SE support b) NW support

Figure C.2.6. Rotation of edge beam relative to wall a) Looking north b) Looking east
C.2.2. Stage 2: Test 4

For Test 4, UFP couplers were placed between the walls. The observed response of the wall elements appeared to be similar to Test 3. The gap opening at the unloaded edge of the wall-base is approximately 10mm at 2% drift.

The relative displacement of the anchorage plates at each side of the UFP coupler is approximately 15mm, which includes the gap opening plus compressive deformation of each wall. There was no apparent slip of the anchorage connections. The relative displacement at the centerline of UFP coupler was approximately 6mm, which indicates the UFP couplers did not deform strictly as anticipated.

The deformable wall elements allowed the UFP couplers to deform so that a constant radius was not maintained (as shown below). Hence, the UFP couplers did not yield (or maintain uniform plastic strain) along the entire semi-circle, reducing the effectiveness of the devices to dissipate energy and increasing the yield drift. Also, because the devices were only fixed on the inside of the wall element the elastic deformation of the UFPs would have increased, again reducing the effectiveness of the devices. In
addition, wedging action created by the UFP couplers caused the walls to splay (by 2 to 3mm) at the base. Again, this limited the strain along the UFP plates.

Figure C.2.8. Observed UFP coupler deformation

Figure C.2.9. Wall uplift at 2.0% Drift a) SE wall, south edge b) SE wall, north edge
Appendix C – Experimental observations for the Test Building

Figure C.2.10. UFP couplers at 2.0% Drift a) E walls, first UFP b) E Wall, second UFP

Figure C.2.11. Relative wall movements at 2.0% Drift
Figure C.2.12. Deformation of UFP couplers at 2.0% Drift

Figure C.2.13. Wall spreading due to UFP couplers at 2.0% Drift
C.2.3. Stage 2: Test 5

Test 5 was similar to Test 4, except the edge beam supports were removed. This allowed the edge beams to drop below the previous support elevation and avoided reaction forces between the edge beam and column.

Figure C.2.14. Wall at 2.0% Drift

Figure C.2.15. Wall spreading due to UFP couplers at 2.0% Drift
Figure C.2.16. Edge beams at 2.0% Drift a) South b) North
C.2.4. Stage 2: Test 6

Test 6 was similar to Test 3 but excluded edge beam supports. Further perpendicular-to-grain crushing was evident around the shear keys at the base of the walls. This could have occurred during Test 4 and 5 and may have been exacerbated by the wall splaying induced by the UFP couplers.

Figure C.2.17. West walls at 2.0% Drift
C.2.5. After the uni-directional tests

After the uni-directional walls tests the screws connecting the edge beams to the walls were checked. None of the screws had fractured, and all remained effectively straight with no noticeable signs of inelastic deformation.
C.2.6. Stage 2: Test 7

Under bi-directional loading up to 3% drift there was no further significant damage to wall system. The displacement incompatibility between the edge-beams and the out-of-plane frames increased. The uplift at the base of the walls increased to approximately 15mm.

The UFP couplers performed in a similar fashion to the uni-directional tests. There were still not signs of movement of the UFP anchorage plates relative to the wall elements.

Figure C.2.20. Wall at peak bi-directional Drift
Figure C.2.21. Tilt of edge beams relative to floor a) Level 3 b) Level 2

Figure C.2.22. SE Wall gap opening
Figure C.2.23. Deformation of UFP couplers  
a) West walls, UFP 3 and 4  
b) West walls, UFP 2  
c) East walls, UFP 2
C.2.7. Stage 2: Test 8

No further damage to the wall system was noted during Test 8.

Figure C.2.24. Test 8 a) West walls b) Removed UFP, west side c) Removed UFP, east side
C.2.8. The wall after testing

During de-construction rounding of the wall bases was noticed. A peak permanent inelastic deformation of 1 to 2mm was recorded at the base of the walls.

Figure C.2.25. Rounding at the wall bases: a) Concrete dust showing the rounding at the end
b) & c) With a straight edge
C.3. GRAVITY SYSTEMS

C.3.1. Uni-directional frame tests

Considering both Test 1 and 2 of Stage 2, the gravity system (joists and primary beam) performed well up to 2% drift, with no apparent damage.

The top hung support detail incorporated a gap between the supporting elements and the joist. This allowed rotation between the joist and the supporting elements, avoided contact and ensured purely pinned supports (which did not contribute to the lateral strength of the building).

The joists on Level 3 and the primary beam remained fixed to the floor slab, remaining essentially flat as the column rotated.

Figure C.3.1. Floor joist supports at 2% Drift: a) West edge b) East edge
Figure C.3.2. Floor joist supports at negative 2% Drift: a) West edge b) West side of primary beam c) East side of primary beam d) East edge
Figure C.3.3. Floor joist support rotation relative to columns at 2% Drift: a) L3 central joist, looking south b) L3 central joist, looking north
C.3.2. Uni-directional wall tests

Considering Test 3 through 6 of Stage 2, again the gravity system (joists and primary beam) performed well up to 2% drift, with no apparent damage.

Figure C.3.4. Floor joist supports at negative 2% Drift: a) South edge b) North edge
Figure C.3.5. Primary beam supports at 2% Drift: a) South edge b) North edge

Figure C.3.6. Level 3 joist rotation relative to frame at 2% Drift: a) Bottom b) Top
C.3.3. Bi-directional test

Considering Test 7, again the gravity system exhibited no apparent damage. There was sufficient gap to accommodate relative rotation between the joists and supporting elements for up to 3% Drift.

Figure C.3.7. Level 3 joist rotation relative to frame at 3% Drift: a) Bottom b) Top
C.4. FLOOR SLAB

The cracking in the floor was monitored during Stage 2 experimentation. This was done using by sketching the crack propagation on a plan view of each level and using photographs.

C.4.1. Crack patterns

The propagation of cracks across the floor slab was recorded during all tests, except Test 2, 4 and 6. For Tests 2, 4 and 6, the cracks were marked at the end of each test (in black).

Different colors were used to distinguish between cracks that occurred during each test and under positive or negative drifts (as described in the table below).

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<th>Line colour</th>
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<td>Pre-test cracking</td>
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<tr>
<td>Red</td>
<td>Positive drift in the EW direction</td>
</tr>
<tr>
<td>Blue</td>
<td>Negative drift in the EW direction</td>
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<tr>
<td>Black</td>
<td>Cracks that occurred during Test 2, 4 and 6</td>
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<tr>
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<td>Positive drift in the NS direction</td>
</tr>
<tr>
<td>Green</td>
<td>Negative drift in the NS direction</td>
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<tr>
<td>Orange</td>
<td>Bi-directional testing</td>
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Pre-test cracks were created in the slab by the attachment of the loading apparatus to the floor. The maximum crack width was 0.2mm.

During Test 1, almost all cracks were localized in the corners of the slab around the walls. These cracks were caused by the displacement incompatibility that occurred between the wall system and the floor. The edge beams rotated with walls (up to 2% drift) but the floor slab adjacent to the beams remained horizontal. Because the slab was fixed to the edge beams via notched composite connections (to transfer in plane loads),
the slab cracked. The largest crack of 0.8mm occurred on the SE edge of the slab on Level 3.

Other cracking was observed for negative drifts that extended across the entire slab in the NS direction. This was caused by the loading apparatus. Under negative drifts, the slab is subjected to tension. To transfer this tensile force to the east bay, the concrete cracked, activating the slab reinforcement (see Appendix A). Hence, this cracking was not induced by frame elongation.

For Test 2, the same crack pattern was maintained, with only a few additional cracks being observed.

During Test 3, when the building was subjected to NS loading, again there was a displacement incompatibility between the wall system and floor. The edge beams remained effectively fixed to the wall system, causing uplift of the corners of the floor slab at one end of the edge beam. Adjacent to the beam (from the frame) the slab remained in-line with the top of the beam (due to the composite connections) causing cracking that extended in either the NS or EW direction. Corbels on the columns forced the edge beams to bend, allowing the slab to remain at the same elevation as the top of the beam. This reduced cracking. The maximum crack width of 0.2mm was recorded during testing.

After Test 4, very little additional cracking was observed in the floor slab.

For Test 5, once the edge beam support had been removed, the slab was able to drop below the top of beam (with the edge beam) at the corners. This significantly increased the number of cracks in the slab adjacent to the walls. The maximum crack was similar to Test 3.

There were very few additional cracks during Test 6.

During Test 7, in general the crack pattern remained similar to the previous unidirectional tests but the number and size the cracks increased. The maximum residual crack width was approximately 1mm. There were several more cracks surrounding the loading apparatus, due to increased lateral forces that were applied to the structure.
Figure C.4.1. Crack pattern after Test 1: Level 2
Appendix C – Experimental observations for the Test Building

Figure C.4.2. Crack pattern after Test 1: Level 3
Figure C.4.3. Crack pattern after Test 4: Level 2
Appendix C – Experimental observations for the Test Building

Figure C.4.4. Crack pattern after Test 3: Level 3
Figure C.4.5. Crack pattern after Test 3: Level 2
Figure C.4.6. Crack pattern after Test 5: Level 3
Figure C.4.7. Crack pattern after Test 5: Level 2
Figure C.4.8. Crack pattern after Test 6: Level 3
Figure C.4.9. Crack pattern after Test 6: Level 2
Figure C.4.10. Crack pattern after Test 7: Level 3
Figure C.4.11. Crack pattern after Test 7: Level 2
C.4.2. Localized damage

Pre-test cracking

Figure C.4.12. Pre-test cracking due to loading apparatus on Level 3: a) & b) SW bay c) NW bay d) SE bay
Test 1
During Test 1, cracking was observed in the corners of the floor slab (as discussed above).

Figure C.4.13. Cracking around edge beams at 0.75% Drift on Level 3: a) & b) NE corner

Figure C.4.14. Cracking around edge beams at negative 0.75% Drift: a) Level 2, SW corner
b) Level 3, NW corner
Appendix C – Experimental observations for the Test Building

Appendix C – Experimental observations for the Test Building

Figure C.4.15. Cracking induced by loading apparatus on Level 3 at negative 1.5% Drift: a) SE bay b) NE bay

Figure C.4.16. Cracking around edge beams over a 500mm grid at 2.0% Drift: a) NE bay b) SE bay

Additional cracking was induced by the loading apparatus. Due to eccentricities between the slab and the application of load, a partial pull-out failure of a bolted connection was observed at the NW corner of the slab on Level 3.
Figure C.4.17. Cracking due to loading apparatus hold-down at 2.0% Drift

Figure C.4.18. Cracking around edge beams at negative 2.0% Drift: a) SW bay b) NW bay
Test 2

Few additional cracks were observed during Test 2. However, the pull-out failure due to the loading apparatus grew slightly.

Figure C.4.19. Further cracking due to loading apparatus hold-down at 2.0% Drift
**Test 3**

During Test 3, cracking was observed in the corners of the floor slab (as discussed above).

![Figure C.4.20. Cracking due to bending of the slab on Level 2 at 2.0% Drift: a) & b) SE bay](image)

![Figure C.4.21. Cracking due to bending of the slab on Level 2 at negative 2.0% Drift: a) & b) SE bay](image)
Figure C.4.22. Cracking due to bending of the slab on Level 3 at negative 2.0% Drift in the SE bay

Foam spacers were effective at preventing crushing and damage to the floor slab at the walls rotated.

Figure C.4.23. Deformation around wall edges: a) Position of flexible foam spacer b) Deformation around spacer at 2% Drift

The frame system rotated out-of-plane relative to the floor slab. This resulted in the gap opening between the beam and the slab, avoiding cracking of the slab. However, this deformation would have caused bending in the coach screws that were used for the timber-concrete composite diaphragm connection.
Figure C.4.24. Gap opening between slab and beam 2.0% Drift: a) NE bay b) NW bay

Figure C.4.25. Slab uplift relative to the beam at negative 2.0% Drift: a) & b) NE bay
**Test 4**

During test 4, floor cracking slightly increased, but generally in the same locations as Test 3.

![Further cracking during Test 4](image)

Figure C.4.26. Further cracking during Test 4: a) & b) East bay, Level 3 c) & d) West bay, Level 3
Test 5

The cracking pattern changed slightly due to the absence of the edge beam supports. The slab was able to drop below the beams, resulting in more visible cracking. Crack around the supporting corbel were created when the slab depressed with the edge beam.

Figure C.4.27. Slab depression relative to the beam at 2.0% Drift on Level 3: a) NE bay b) NW bay

Figure C.4.28. Slab uplift relative to the beam at 2.0% Drift on Level 3: a) NE bay b) NW bay
Appendix C – Experimental observations for the Test Building

There were no significant changes in the crack patterns during Test 6. Some cracking was noticed between edge-beams and concrete slab. Similar cracking was not observed on Level 3, where the in-plane forces are double that at Level 2. This indicates that the cracking was caused by restraint of the edge of the floor slab as the edge beam dropped below the height of the beam, which acted to delaminate the slab from the formwork.

Figure C.4.29. Slab uplift relative to the beam at negative 2.0% Drift on Level 3: a) NW bay b) NE bay

Test 6

Figure C.4.30. Slab cracking around edge beam connection at SE corner on Level 2
Test 7

For the bi-directional testing, cracking patterns were similar to the uni-directional tests. Generally cracks increased in length, became more numerous in certain areas and increased in width. No new trends in crack development were noted.

Figure C.4.31. Slab distortion at peak bi-directional Drift: a) Level 3 b) Level 2

Figure C.4.32. Further cracking during Test 4: a) & b) East bay, L3 c) & d) West bay, L3
APPENDIX D

MODELLING FRAME SYSTEMS

This appendix focuses on the modeling the response of post-tensioned timber frames. The predicted response of the frames is compared with complex finite element models (FEM) and experimental data.

D.1. COLUMN DEFORMATION USING FEM

In the section, the analytical predictions of the column deformation are compared with the results of finite element model (FEM) created in SAP 2000 (2005). For the numerical model, it was not possible to separate the individual components of column deformation into joint panel, column deformation, as considered for analytical modeling. Instead, a combination of joint panel and column deformation was considered, as illustrated in Figure D.1.2. A small sensitivity study is performed in which several different parameters are varied, to ensure the accuracy of the analytical equations.

D.1.1. The finite element model

The model is a two-dimensional representation of the column which consists of orthotropic elastic shell elements. A fine mesh was required to achieve convergence of displacements from model, as shown in Figure D.1.1. A benchmark model was specified, and key parameters were altered to determine their effect on the accuracy of the analytical predictions. The material properties and dimensions for the benchmark model are given in Table D.1.1.
The boundary conditions are pin supports at the top and bottom of the column. The nodes at the top and bottom edges of the column are constrained to the same rotation and can not deform transversely; hence, a linear strain profile is enforced.

For the benchmark model, the applied loads are linearly distributed, and provide a connection moment of 150kN.m. The distance between extreme top and bottom point loads is the beam depth. The beam shear is calculated for a 6m bay length and is simulated by using uniformly distributed point loads within the simulated compression region (or neutral axis depth).

Figure D.1.1. Boundary conditions and applied loads for the benchmark FEM of the column
Table D.1.1 – Material properties and geometry of the benchmark column model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Units</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus – Parallel to grain</td>
<td>$E_{para}$</td>
<td>MPa</td>
<td>13200</td>
</tr>
<tr>
<td>Elastic modulus – Perp. to grain</td>
<td>$E_{perp}$</td>
<td>MPa</td>
<td>660</td>
</tr>
<tr>
<td>Poisson’s ratio*</td>
<td>$\nu$</td>
<td>-</td>
<td>0.3</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>$G$</td>
<td>MPa</td>
<td>600</td>
</tr>
<tr>
<td>Column height</td>
<td>$H$</td>
<td>m</td>
<td>3.2</td>
</tr>
<tr>
<td>Column depth</td>
<td>$h_c$</td>
<td>mm</td>
<td>600</td>
</tr>
<tr>
<td>Column width</td>
<td>$b_c$</td>
<td>mm</td>
<td>380</td>
</tr>
<tr>
<td>Beam depth</td>
<td>$h_b$</td>
<td>mm</td>
<td>600</td>
</tr>
<tr>
<td>Bay length</td>
<td>$L_b$</td>
<td>mm</td>
<td>6000</td>
</tr>
</tbody>
</table>

*Based on research performed on Pine by Murray (2007).

D.1.2. Comparison of the model and the analytical equations

The FEM deformation can be compared with the analytical model, to a limited extent. Because the boundary conditions for the FEM are different to those assumed for the analytical equations, it is not possible to simply compare the sum of joint and column deformation, as defined in Chapter 6. This is illustrated in Figure D.1.2.
The total rotation of the FEM, as shown in Figure D.1.2, is:

\[ \theta_{\text{tot,m}} = \theta_{\text{col}} + \theta_{j,m} = \frac{\Delta_x - \Delta_y}{h_b} \]

To allow comparison, \( \theta_{\text{tot,m}} \) must be defined with analytical equations. The column rotation, \( \theta_c \), was defined in Chapter 6 as:

\[ \theta_c = \frac{\phi_c}{H} \left( \frac{(H-h_b)^2}{6} + \frac{E_i h_c^2}{G/4} \right) \]

Where:

\[ \phi_c = \frac{M_{\text{con}} L_b (H - h_b)}{E_i I_c H (L_b - h_c)} \].

The rotation due to joint panel rotation, \( \theta_{j,m} \), can be derived using simple trigonometry as:

\[ \theta_{j,m} = \frac{H - h_b}{H} \gamma \]

Where:

\[ \gamma = \frac{V_{jh}}{G A_{sh}} \]

\[ V_{jh} = 2 \frac{M_{\text{con}}}{h_b} - V_{\text{col}}; \]

\[ A_{sh} = b_i h_c. \]

Hence, for the analytical comparison:

\[ \theta_{\text{tot,m}} = \theta_{\text{col}} + \theta_{j,m} = \theta_{\text{col}} + \frac{H - h_b}{H} \gamma \]

D.1.3. Results from the sensitivity study

In this section, the results of the sensitivity study on the column deformation are considered. For each FEM permutation, key findings are presented and the deformation of the column is compared with the analytical predictions in Table D.1.2.
**FEM 1 (Benchmark)**

The deformed shape, transverse, longitudinal and shear stresses are shown in Figure D.1.3. The shear, transverse and longitudinal stresses range from -3 to 5, -11 and 0 and -35 and 11 respectively. In Figure D.1.3, the minimum and maximum values correspond to purple and blue respectively.

The analytical prediction for the benchmark model was 9% greater than that of the FEM, as shown in Table D.1.2. It is preferable that the equations are slightly conservative, as is the case here, because the connection rotation and therefore the design strength of the frame is conservative.

**FEM 2**

For FEM 2, the only alteration from the benchmark models was the neutral axis depth, which was halved. This modification had no significant effect on the accuracy of the analytical predictions (which was 8% larger than the deformation of the FEM).

The shear stresses within the joint panel are shown in Figure D.1.4, over the same range as the benchmark model. The angle of the compression strut does not appear to significantly affect the accuracy of the predictions. The shear stresses due to the applied loads are concentrated mainly with the joint. However, there is some diffusion of shear stress to outside the joint panel, which may account for the slight inaccuracy of the predictions.

**FEM 3**

For FEM 3, the only alteration from the benchmark models was the stress distribution. Instead of a linear stress distribution, a uniform stress distribution was applied, which gave the same connection moment as the benchmark model. This modification had no significant effect on the accuracy of the predictions.

**FEM 4**

For FEM 4, the connection moment is doubled and the neutral axis depth is halved from the benchmark model. These modifications had no significant effect on the accuracy of the predictions.
**FEM 5**
For FEM 5, the neutral axis depth is halved and the column depth is reduced by one-third from the benchmark model. In this case, the prediction improved in accuracy but slightly underestimated the column deformation by 5%. Due to the increased aspect ratio of the joint panel, it is expected that the contribution of flexural deformation increased. Flexural deformations are not considered in the analytical predictions.

**FEM 6**
For FEM 6, the neutral axis depth is halved and the column depth is increased by two-thirds from the benchmark model. The prediction significantly over-estimates the column deformation by 44%. There is significant diffusion of stresses into the column, outside the joint panel zone. The diffusion of stresses results in a less pronounced compression strut and less overall column deformation, and consequently, conservative predictions.

**FEM 7**
For FEM 7, the neutral axis depth is halved and the beam depth is reduced by one third from the benchmark model. The prediction is 23% larger than the deformation of the FEM. Similar to FEM 6, there is significant stress diffusion and a less defined compression strut, which appears to have increased the joint panel stiffness. The effect of stress diffusion is not as significant as FEM 6, because the neutral axis depth is larger relative to the height of the beam.

**FEM 8**
For FEM 8, the neutral axis depth is halved and the beam depth is increased by two-thirds from the benchmark model. The prediction is only 1% larger than the deformation of the FEM.
Figure D.1.3. Results of benchmark FEM: a) Deformed shape b) Shear stresses c) Transverse stresses d) Longitudinal stresses
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe

Figure D.1.4. Shear stresses in joint panel: a) Benchmark b) FEM 2

Figure D.1.5. Shear stresses in joint panel: a) Benchmark b) FEM 3
Figure D.1.6. Shear stresses in joint panel: a) Benchmark b) FEM 4

Figure D.1.7. Shear stresses in joint panel: a) Benchmark b) FEM 5
Figure D.1.8. Shear stresses in joint panel: a) Benchmark b) FEM 6

Figure D.1.9. Shear stresses in joint panel: a) Benchmark b) FEM 7
Figure D.1.10. Shear stresses in joint panel: a) Benchmark b) FEM 8
Summary of the sensitivity study

Based-on the results in Table D.1.2, the analytical predictions are sufficiently accurate. For joint panels that have an aspect ratio \((h_b/h_c)\) of less than one, the analytical predictions will be conservative for design.

<table>
<thead>
<tr>
<th>FEM Model</th>
<th>Change from benchmark</th>
<th>Eqn. (\theta_c (\text{rad}))</th>
<th>Eqn. (\gamma (\text{rad}))</th>
<th>Eqn. (\theta_{tot,m} (\text{rad}))</th>
<th>FEM</th>
<th>Accuracy*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Benchmark</td>
<td>0.00132</td>
<td>0.00299</td>
<td>0.00374</td>
<td>0.00351</td>
<td>7 %</td>
</tr>
<tr>
<td>2</td>
<td>Half neutral axis</td>
<td>0.00132</td>
<td>0.00299</td>
<td>0.00374</td>
<td>0.00345</td>
<td>8 %</td>
</tr>
<tr>
<td>3</td>
<td>Uniform stress</td>
<td>0.00132</td>
<td>0.00299</td>
<td>0.00374</td>
<td>0.00347</td>
<td>8 %</td>
</tr>
<tr>
<td>4</td>
<td>Double (M_{con}) Half neutral axis</td>
<td>0.00263</td>
<td>0.00597</td>
<td>0.00749</td>
<td>0.00690</td>
<td>9 %</td>
</tr>
<tr>
<td>5</td>
<td>Half neutral axis 2/3 column depth</td>
<td>0.00287</td>
<td>0.00448</td>
<td>0.00651</td>
<td>0.00687</td>
<td>-5 %</td>
</tr>
<tr>
<td>6</td>
<td>Half neutral axis 5/3 column depth</td>
<td>0.00061</td>
<td>0.00179</td>
<td>0.00206</td>
<td>0.00143</td>
<td>44%</td>
</tr>
<tr>
<td>7</td>
<td>Half neutral axis 2/3 beam depth</td>
<td>0.00150</td>
<td>0.00482</td>
<td>0.00572</td>
<td>0.00464</td>
<td>23 %</td>
</tr>
<tr>
<td>8</td>
<td>Half neutral axis 5/3 beam depth</td>
<td>0.00100</td>
<td>0.00152</td>
<td>0.00204</td>
<td>0.00202</td>
<td>1 %</td>
</tr>
</tbody>
</table>

*Positive percentages indicate that the column deformation is over predicted.
D.2. JOINT PANEL DEFORMATION USING EXPERIMENTAL DATA

For further verification, the predicted joint panel zone deformation is compared with experimental data from subassembly tests. The joint panel deformation is determined using diagonal potentiometers placed on the joint panel, as shown in Figure D.2.1. The joint panel shear distortion, \( \gamma \), is calculated as:

\[
\gamma = \frac{\Delta_2 - \Delta_1}{2L_g} \left( \tan \theta + \frac{1}{\tan \theta} \right)
\]

Where: \( \Delta_1 \) and \( \Delta_2 \) is the displacement of Pot 1 and 2;
\( L_g \) is the gauge length of the potentiometers.

However, caution should be exercised when interpreting the joint panel deformation from experimental data for timber frames. There is significant perpendicular-to-grain strain (\( \varepsilon_z \)) that occurs near the compression regions. This has the effect of artificially increasing the measured joint distortion, \( \gamma \), as illustrated in Figure D.2.2. To minimize inaccuracies in measurement (due to the transversal strains in the column), the potentiometers should be located at the center of joint panel and have a short gauge length. Alternatively, additional potentiometers should be placed transversely, at the top and bottom of the joint panel. This will enable the transverse deformation of the column and the resultant rotation, \( \gamma_c \), to be quantified, and then subtracted from the measured joint panel deformation, \( \gamma_{tot} \). Therefore:
\[
\gamma = \gamma_\text{tot} - \gamma_c = \frac{\Delta_2 - \Delta_1}{2L_g} \left( \tan \theta + \frac{1}{\tan \theta} \right) \delta_c \frac{h_b}{h_b}
\]

Where: \( \delta_c \) is the difference in displacement between the top and bottom transverse potentiometers.

To-date no experimental tests have adequately taken into account the interaction of shear and transversal deformation, and hence, placed potentiometers away for the compression regions. Therefore, the comparison of the FEM and the analytical model is likely to be the most robust for now.

Persevering with experimental data, an approximate evaluation of the accuracy of the analytical model can be made by considering external and internal beam-column subassembly tests (Cusiel 2010; Green 2010; Iqbal et al. 2010). Experimental data from the test building, as discussed in Chapter 5, is not as reliable due to the approximate means of calculating the connection moments (using strain gauges). A summary of the subassembly tests considered is given in Table D.2.1 and Figure D.2.3. In Table D.2.1, the area of beam, \( A_b \), the force in the post-tensioning tendons, \( T_{pt} \), and the initial stress applied the column, \( f_i \), is also given.

The experimental data from the subassembly tests is compared with analytical predictions in Figure D.2.4. For the predictions, as shear modulus of 660MPa is assumed. For subassembly 1 (Sub 1) from Green (2010), two pair of diagonal potentiometers are used to measure the joint panel deformation (see Figure D.2.3a).
This arrangement effectively reduces the contribution of transverse deformation to the shear distortion, by average the joint distortion between the two diagonal pairs of potentiometers. Because the perpendicular-to-grain stresses are relatively low for Test 1A and 3 (see Table D.2.1), the transverse deformation is further reduced. Consequently, the predictions match relatively well with the experimental data, as shown in Figure D.2.4a and b.

For Sub 2 (Cusiel et al. 2010), the predicted panel stiffness is significantly higher than indicated by experimental data. This is expected due to the proximity of the diagonal potentiometers to large transverse deformations. Furthermore, inelastic deformation is apparent in Figure D.2.4c, associated with the transverse deformations, which are actually linked to the rocking connection response. Therefore, contrary to recent research (Cusiel et al. 2010), the joint panel deformation should not provide any additional damping to the frame system and should be considered as part of the rocking connection response.

For Sub 3 (Iqbal et al. 2010), top and bottom potentiometers were present in the joint panel region (see Figure D.2.4c and d). This meant that the joint deformation could be approximately decoupled into transversal and shear distortion, as shown in Figure D.2.4d. The resultant joint deformation, $\gamma$, matches well with the analytical prediction. While there appears to be some hysteresis with the pure shear distortion, it is likely that this is due to inconsistent zero positions for the horizontal and diagonal potentiometers.

<table>
<thead>
<tr>
<th>Sub.</th>
<th>Test</th>
<th>Description</th>
<th>$A_b$ (mm$^2$)</th>
<th>$T_{pt}$ (kN)</th>
<th>$f_i$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Test 1A</td>
<td>External - No armouring</td>
<td>72000</td>
<td>62</td>
<td>0.9</td>
</tr>
<tr>
<td>2</td>
<td>Test 1</td>
<td>External - No armouring</td>
<td>72000</td>
<td>186</td>
<td>2.6</td>
</tr>
<tr>
<td>3</td>
<td>Test 3</td>
<td>Internal - No armouring</td>
<td>55800</td>
<td>240</td>
<td>4.4</td>
</tr>
<tr>
<td>3</td>
<td>Test 3</td>
<td>External - Armoured</td>
<td>223000</td>
<td>870</td>
<td>3.9</td>
</tr>
</tbody>
</table>
Figure D.2.3. Subassemby tests considered for joint panel moment-rotation response: a) Sub 1 (Green 2010) b) Sub 2 (Cusiel et al. 2010) c) and d) Sub 3 (Iqbal et al. 2010)
Figure D.2.4. Experimental and predicted joint response for subassembly tests: a) Sub 1 - Test 1 b) Sub 1 - Test 3 c) Sub 2 - Test 1 d) Sub 3 - Test 3
D.3. CALIBRATION OF THE PASTERNAK MODEL

The Pasternak Model, as applied to timber by Tanahashi et al (2006), is proposed for defining the connection response of non-armoured post-tensioned timber frames (see Chapter 6). The Pasternak Model accounts for shear deformation of the column adjacent to beam-column interface (see Figure D.3.1), termed henceforth as ‘edge-effect’. To do this, the parameter, $\alpha$, must be calibrated for the specific material properties of the timber. The parameter $\alpha$ is related to $\Phi$ and, in turn, $\eta$.

The parameters $\eta$ and $\Phi$, define the strain profile into the column depth, as shown in Figure D.3.1. A two-dimensional finite element model is created in SAP 2000 (2005) to determine $\eta$ and $\Phi$ for Laminated Veneer Lumber (LVL), with the same material properties as shown in Table D.1.1. A sensitivity study of the FEM models is performed and the results are summarized in Table D.3.1.

To determine $\eta$ and $\Phi$, the displacement profile for the Pasternak model is calibrated to the displacement profile from the FEM. From Tanahashi et al (2006), the displacement profile is:


\[ W(z) = W_0 \frac{\sinh(\eta \left( 1 - \frac{z}{h_c} \right))}{\sinh \eta} \]

Where: \( W_0 \) is the surface displacement.

And:

\[ \Phi = \eta \sqrt{\frac{\sinh \eta \cosh \eta + \eta}{\sinh \eta \cosh \eta - \eta}} \]

The shape of the displacement profile, outside the compression region is related to the parameter, \( \Phi \), according to the equation:

\[ \alpha = \frac{E_{\text{perp}}}{\sqrt{(1 - \nu_{LT} \nu_{TL}) G h_c}} \Phi \]

Where: \( \nu_{LT} \) and \( \nu_{TL} \) are the Poissons Ratios in the longitudinal and transversal directions;

\( G \) is the shear modulus (in the longitudinal and transverse directions).

For this study the poisons ratios, \( \nu_{LT} \) and \( \nu_{TL} \), are assumed to be 0.3, based on research performed on Radiata Pine by Murray (2007).

**D.3.1. Results of the sensitivity study**

The sensitivity study is performed by creating a benchmark model and modifying one parameter at a time within realistic bounds. The material properties of the timber were assumed to remain fixed. Hence, this study is only applicable to Laminated Veneer Lumber, with the properties given in Table D.1.1. In this section, key results from the sensitivity study are provided.

**FEM 1 (Benchmark)**

The deformed shape of the model is illustrated in Figure D.3.2a. The displacement profile at the centreline of the FEM and the calibrated Pasternak Model representation is shown in Figure D.3.2b.
**FEM 2**

The support conditions were altered from FEM 1. Instead of a rigid support on the left hand side of the column, supports were provided at the top and bottom of the column, as shown in Figure D.3.3a. This resulted in a different displacement profile (see Figure D.3.3b), that could not be accurately represented by the Pasternak Model. Therefore, the interaction of longitudinal bending stresses result in an altered displacement profile in the transverse direction. This is not considered by Tanahashi et al (2006). Possibly, because the bending stresses are equal and opposite above and below the centreline of the beam, the support conditions of the benchmark model are appropriate for calibration of the Pasternak Model. Further research is required.

**FEM 3**

In this model, the compression area is halved. This altered the displacement profile slightly, as shown in Figure D.3.4.

**FEM 4**

In this model, the column depth is halved. The displacement profile is almost linear. Hence, the model is tending towards a state of full compression and the displacement profile in the centre is essentially unaffected by the diffusion of stresses above and below the compression region.

**FEM 5**

In this model, the column depth is doubled. The displacement profile is similar to FEM 3.
Appendix D – Modeling frame systems

Figure D.3.2. Deformation of benchmark model: a) FEM b) The calibrated Pasternak Model and FEM displacement profile

Figure D.3.3. Deformation of FEM 2: a) FEM b) The calibrated Pasternak Model and FEM displacement profile
Figure D.3.4. Deformation of FEM 3: a) FEM b) The calibrated Pasternak Model and FEM displacement profile

Figure D.3.5. Deformation of FEM 4: a) FEM b) The calibrated Pasternak Model and FEM displacement profile
D.3.2. Summary of the sensitivity study

The results of the sensitivity study are shown below. Firstly, it is evident that the aspect ratio of the column depth to the length of the compression area affects the parameter, \( \eta \) and \( \Phi \). For a rocking connection, the depth of the compression region will change as the connection rotation increases, but the depth of the column is constant. To minimise variation of \( \Phi \), a constant ratio of \( \alpha \cdot h_c \) of 2.0 is considered. The implication of choice on the moment-rotation response is examined in Chapter 6.

<table>
<thead>
<tr>
<th>FEM Model</th>
<th>Change from benchmark</th>
<th>( \eta )</th>
<th>( \Phi )</th>
<th>( \alpha )</th>
<th>( \alpha \cdot h_c )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Benchmark</td>
<td>1.0</td>
<td>1.86</td>
<td>0.0034</td>
<td>2.04</td>
</tr>
<tr>
<td>2</td>
<td>Altered support conditions</td>
<td>2.0</td>
<td>2.32</td>
<td>0.0043</td>
<td>2.55</td>
</tr>
<tr>
<td>3</td>
<td>Halved compression area</td>
<td>1.6</td>
<td>2.09</td>
<td>0.0038</td>
<td>2.30</td>
</tr>
<tr>
<td>4</td>
<td>Halved column depth</td>
<td>0.4</td>
<td>1.75</td>
<td>0.0064</td>
<td>1.92</td>
</tr>
<tr>
<td>5</td>
<td>Double column depth</td>
<td>1.6</td>
<td>2.09</td>
<td>0.0019</td>
<td>2.30</td>
</tr>
</tbody>
</table>
D.4. BEAM-COLUMN CONNECTION RESPONSE USING FEM

Here the accuracy of analytical relationships, derived in Chapter 6, for modeling non-armoured PT beam-column connections are evaluated. Firstly, the proposed relationship between the neutral axis depth, c, and the imposed connection rotation, $\theta_{imp}$, is considered. This expression is repeated below:

$$\theta_{imp} = \frac{T_{pt} h_t}{P_{perp} b_h \left( \frac{e^2 + 2c}{\alpha} \right)}$$

Where:
- $T_{pt}$ is the force applied by the post-tensioning;
- $\alpha$ defines the displacement profile on the column face (see Chapter 6);

Secondly, the stresses distribution applied to the column face are investigated. It is proposed that the peak stress applied to the column is approximately:

$$f_t = \frac{2T_{pt}}{cb_t}$$

Analytical expressions are compared with the results of sensitivity study using finite element models created in SAP 2000 (2005). This model is an extension of the column model from section D.1 and includes beam elements, as shown in Figure D.4.1. The sensitivity study considers variation in several parameters, to ensure a robust evaluation of the accuracy of the analytical connection rotation versus neutral axis depth ($\theta_{imp}$-c) predictions and peak stresses. This study is limited to the elastic response of the timber within the beam-column connections.

D.4.1. The finite element model

Again, a benchmark beam-column subassembly model was specified, and key parameters were altered to determine their effect on the accuracy of the $\theta_{imp}$-c predictions and the peak stresses. The material properties and dimensions for the benchmark model were similar to the column model in section D.1, given in Table
D.1.1. The beam and the column have identical material properties and section size, but the beam was orientated at 90 degrees to the column. The bay length was 6 meters.

The boundary conditions consist of a pin support at the bottom of the column and vertical restraint at the end of the beams. Lateral loads are applied to the top of the column, which create a known connection moment. Axial loads, of 1020kN for the benchmark model, are applied to the end of the beams to simulate the compression provided by the post-tensioning.

To simulate the rocking beam-column connection, any shell elements in tension at the connection interface were deleted, as shown in Figure D.4.1. This was an iterative process, which required shell elements to be deleted or added until the transverse stress at the edge of the neutral axis was approximately zero. The fineness of the mesh was enhanced around the beam-column interface to improve the accuracy of the model and to avoid instability due to the interaction of shell elements which represent the parallel and perpendicular-to-grain timber.

Figure D.4.1. Boundary conditions and applied loads for the benchmark FEM of the beam-column subassembly
D.4.2. Comparison of the model and the analytical equations

To evaluate the accuracy of the analytical $\theta_{\text{imp}}$-$c$ relationship, the results from the FEM must be interpreted accurately. The neutral axis depth can be readily interpreted by observing the transverse stresses, but the imposed connection rotation is more problematic. This is because the joint panel deformation and the connection deformation are difficult to decouple in the FEM.

One approach (Approach 1) is to deduce the connection rotation by subtracted the member deformation from the total deformation, $\Delta_{\text{tot}}$, of the subassembly (measured at the top of the column). It was shown in section D.1 that the member deformation could be predicted with sufficient accuracy using analytical equations (from Chapter 6).

Hence:

$$\theta_{\text{con}} = \frac{\Delta_{\text{tot}}}{H} - (\theta_{b} + \theta_{c} + \theta_{j})$$

And:

$$\theta_{\text{imp}} = \frac{\theta_{\text{con}}}{1 - \frac{h_{c}}{L_{b}}}$$

Furthermore, using this approach evaluates the accuracy of the overall analytical frame modeling procedure, because it incorporates the member and connection deformations.

An alternative approach (Approach 2) is to determine the imposed rotation by taking the difference between average rotation of the column centerline and the rotation of the column surface under compression (see Figure D.4.2). This approach becomes less accurate as the neutral axis depth becomes smaller. When the neutral axis depth is small, the displacement profile on the surface of the column is affected by shear distortion of the joint panel and the axial deformation of the beam.
Both approaches are considered to evaluate the imposed rotation from the FEM analyses.

The stresses applied to the column face in the FEM analyses were determined by recording the element joint forces. To avoid numerical instabilities created by the interaction of parallel and perpendicular-to-grain elements, the joint forces were recorded at 12.5mm into the column.
D.4.3. Results from the sensitivity study

The sensitivity study considered many parametric variations. Hence, only key results are presented. A summary of the study is given in Table D.4.1.

**Benchmark response**

The benchmark response for a connection moment of 150kN.m is shown below in terms of deformation, horizontal stresses, vertical stresses and shear stresses. The displacement profile of the column-face appears to correspond well with that assumed by the Pasternak Model (see Figure D.4.3). Furthermore, the diffusion of transverse stresses is shown in Figure D.4.4.

![Figure D.4.3. Results of benchmark FEM: Deformed shape](image)
Figure D.4.4. Results of benchmark FEM: a) Horizontal stresses b) Vertical stresses c) Shear stresses
The $\theta_{\text{imp}}$-$c$ relationship

Three models were subjected to several different moment demands; model 1, 2 and 3 from Table D.4.1. This enabled the relationship between the neutral axis depth and the imposed connection rotation to be established for each of these models, as shown in Figure D.4.5. The $\theta_{\text{imp}}$-$c$ equation appears to give a slightly larger neutral axis depth for a given rotation, and therefore, is conservative when predicting the moment provided by the connections.

Figure D.4.5. Normalized neutral axis depth versus connection rotation: a) FEM 1 b) FEM 2 and c) FEM 3
The accuracy of the analytical $\theta_{imp}$-$c$ relationship was evaluated for different transverse (or perpendicular-to-grain) elastic moduli in the column. It was determined that there were significant discrepancies when the transverse elastic modulus increased and tended to the parallel-to-grain elastic modulus. This is because the assumed surface displacement profile, from the Pasternak Model, is not appropriate for high transverse elastic moduli.

The column deformation, magnified by ten, is shown in Figure D.4.6 for four different transverse moduli. It can be observed that there is a less shear distortion adjacent to the compression edge of the beam (or edge-effect) as the transverse elastic modulus increases. Or more specifically, $\alpha$ (as defined in Chapter 6), appears to be larger. Furthermore, Figure D.4.7 shows that there is less diffusion of the transverse stresses as the elastic modulus increases.

Figure D.4.6. Connection deformation for different transverse elastic moduli ($E_{\text{perp}}$): a) 400 MPa b) 1320 MPa c) 8000 MPa d) 13200 MPa
Therefore, it is proposed that the edge-effect, accounted for in the Pasternak Model, can be ignored if the transverse timber in the column is aligned parallel-to-grain. Therefore:

\[ \theta_{\text{imp}} = \frac{T_p h_c}{E_{\text{perp}} b h c^2} \]

However, when the transverse elastic modulus in the column tends toward the parallel-to-grain elastic modulus in the beam, it is no longer reasonable to assume the beam is effectively rigid compared to the column, as assumed in Chapter 6 for the Pasternak Model. The contribution of the beam to the flexibility of the connection is difficult to accurately define or interpret from FEM. The flexibility is derived from the compressive strains at the beam end, which are in excess of longitudinal strains due to flexure. Therefore, several different FEM analyses are run with a parallel-to-grain (13200MPa) transverse elastic modulus to calibrate the appropriate axial stiffness of the connection. The \( \theta_{\text{imp-c}} \) relationship is shown in Figure D.4.8 (and compared with the
For the $\theta_{\text{imp}}$ prediction, it is tentatively proposed that the axial stiffness of the beam should be one third of the axial stiffness of the column, and hence:

$$\theta_{\text{imp}} = \frac{3}{2} \frac{T_p h_c}{E_{\text{perp}} b_c c^2}$$

From Figure D.4.8, it is noted that there are larger discrepancies between Approach 1 and 2 for calculation of imposed rotation from the FEM. This is because the beam-end and column-face have a similar axial stiffness. Deformation of the beam end adds to the imposed rotation, but is not accounted for by Approach 2. Furthermore, the deformations of the beam end are non-linear, further complicating the computation of the imposed rotation. Hence, Approach 1 is considered more robust for these analyses.

Figure D.4.8. Normalized neutral axis depth versus connection rotation: FEM 4
The addition of a steel plate to the column face effectively increases edge-effects, distributing deformation further outside the beam-column connection. This is shown in Figure D.4.9, by analyzing the column section with uniform transverse displacement at the beam position, as done in section D.3. Similar observations are made for the beam-column FEM model in Figure D.4.10. The increased edge-effect is also evident when observing the transverse stresses in the column, which are distributed further above and below the connection (see Figure D.4.11). This results in significantly reduced perpendicular-to-grain stresses.

![Figure D.4.9. Displacement profile for the Pasternak Model with a steel armouring plate: a) Benchmark b) With a 30mm steel plate](image)

![Figure D.4.10. Connection deformation (magnified by 10) with a steel armouring plate: a) Benchmark b) With a 30mm steel plate](image)
To adequately account for steel plate armoring using the Pasternak Model, the displacement profile at the surface of the column needs to be defined. This will depend on the thickness of the steel plate and the distance that the plate extrudes outside of the beam column connection. An example of how the displacement profile may look is given in Figure D.4.13. A conservative design approach is to ignore the presence of the steel plate, for the calculation of the $\theta_{\text{imp-c}}$ relationship. The consequence of this assumption is examined in Figure D.4.12, for a 30mm thick plate with 100mm extrusion at the top and bottom of the beam (FEM 16). In Figure D.4.12, the benchmark prediction ignores the presence of the plate. It is apparent that the imposed rotation is significantly over-predicted for a given neutral axis depth.

Again, it is noted that Approach 2, for determination of the imposed rotation, is inaccurate due to deformation at the end of the beam. Furthermore, the accuracy of Approach 1 may be affected by the addition of the steel plates in the FEM. In the model, the steel plates are fully connected to the column and will contribute to the flexural and shear stiffness of the column. This will decrease column deformation and result in an underestimation of the imposed connection rotation using Approach 1. In reality, the steel plates are able to slide on the column face and can detach from the column surface. Hence, further analysis is required to characterize the $\theta_{\text{imp-c}}$ relationship with steel plate armouring.
If the plate is designed to be effectively rigid, the displacement profile in the column can be readily defined, as shown in Figure D.4.13. Following the same procedure from Chapter 6, the $\theta_{imp}$-c relationship can be re-defined as:

$$\theta_{imp} = \frac{T_{pl} h_c}{E_{perp} b_h \left( c + e_p \left( c + e_p + \frac{2}{\alpha} \right) \right)}$$

Where: $e_p$ is distance that the plate extrudes outside of the beam-column connection.

The above $\theta_{imp}$-c relationship is compared with the results of FEM 16 in Figure D.4.12. The accuracy of the prediction appears to have improved significantly, but remains conservative due to the inaccuracies of calculating the imposed rotation from the FEM.
Figure D.4.13. Displacement profile of column face with steel armouring
Three FEM analyses (FEM 17) investigated whether the $\theta_{imp}$-c relationship varied for an external beam-column connection. One of the beams from the benchmark model was removed. A tendon anchorage was represented using a 30mm thick steel plate on the face of the column, as shown in Figure D.4.14a.

The predicted $\theta_{imp}$-c relationship, which is identical to the benchmark subassembly, matched well with the FEM analysis results (see Figure D.4.14b). It is noted that Approach 1 for interpreting the FEM results is less accurate for an external beam-column subassembly. This is because the deformation at the top of the column, due to joint panel deformation cannot be accurately defined. However, for Type 1 (unarmoured) connections, the FEM results for the benchmark model according to Approach 1 and 2 are similar. Therefore, it can be inferred that Approach 2 is fairly accurate for the external subassembly. Furthermore, the computed joint panel deformation is relatively small.

The modification of beam depth, post-tensioning force, column depth and shear modulus did not appear to affect the accuracy of the proposed $\theta_{imp}$-c relationship. Perhaps the lack of sensitivity to the shear modulus is counter-intuitive. It would be expected that as the shear modulus increased that the diffusion of transverse stresses
into the column would be more significant. However, it is inferred that the perpendicular-to-grain elastic modulus is the most important parameter in defining the column displacement profile for the Pasternak Model. A more comprehensive sensitivity analysis is required in future research. These studies should focus more heavily on the connection response for armoured columns.

The applied stress distribution

As introduced in Chapter 6, the applied stress distribution at the column face is not equal to the average transverse stress distribution within the column. The Pasternak Model inherently assumes a shear resistant layer which distributes applies stress to the column, to satisfy a certain displacement profile. If the applied stress distribution were linear, the peak stress at the extreme fiber of the beam is:

\[ f_t = \frac{y}{c} \frac{2T_{pl}}{cb_h} \]

Where: \( y \) is distance from the neutral axis position into the compression region.

The above expression is compared with the results of the FEM sensitivity study, as shown in Figure D.4.15. For the unarmoured connections (FEM 1, 2 and 3), the stress distribution appears to be non-linear. Hence, the peak stresses are higher than predicted toward the extreme compression fiber and lower than predicted towards the neutral axis position. This is likely to be result of interaction with the shear distortion of the joint panel region, which also has a non-linear deformed shape. However, the predictions still provide reasonable estimates of the peak stress. Furthermore, under cyclic loading the extreme compression fibers will yield, causing redistribution of stresses, which will result in a more linear stress profile. Finally as the timber strain increases, an elastoplastic stress distribution may be appropriate. Also, the non-linearity of the stresses may have been exacerbated by inaccuracies in the FEM model. The finite elements on the extreme compression edge of the beam have half the stiffness of elements within the section, resulting in a reduction of peak stress at the extreme fiber (see Figure D.4.15).
For connections with parallel-to-grain timber running transversely through the column, there is no significant affect on the shape of the stress distribution, as shown in Figure D.4.15c for FEM 4. However, armouring will steel plates significantly reduces the non-linearity and magnitude of the applied stress. The steel plate is analogous to the shear layer used in the Pasternak Model, and distributes applied stresses throughout the column. If the steel plate is thick enough, the parallel-to-grain timber in the beam yields before the perpendicular-to-grain timber in the column. This was the case for FEM 16.

Figure D.4.15. Stresses applied to column face: a) Benchmark (FEM 1) b) FEM 2 c) FEM 4 d) FEM 16
Summary of the sensitivity study

The $\theta_{\text{imp}}$-$c$ relationship predicted by Pasternak Model is in good agreement with the FEM sensitivity study. Three analytical equations are used for the sensitivity study.

For traditional PT timber frames (where compression is applied to perpendicular-to-grain timber without armouring):

$$\theta_{\text{imp}} = \frac{T_p h_c}{E_{\text{perp}} b_c \left(c^2 + 2c \frac{t}{\alpha}\right)}$$

If parallel-to-grain timber is aligned transversely within the column:

$$\theta_{\text{imp}} = \frac{3}{2} \frac{T_p h_c}{E_{\text{perp}} b_c c^2}$$

If an effectively rigid steel plate is used to armour the column face:

$$\theta_{\text{imp}} = \frac{T_p h_c}{E_{\text{perp}} b_h \left(c + e_p \left(c + e_p + \frac{2}{\alpha}\right)\right)}$$

The predicted imposed rotation from the above equations is compared with the results of the sensitivity study in Figure D.4.16.
The stresses applied to the column face are predicted reasonably accurately by the analytical expression, repeated below.

\[
f_i = \frac{y \cdot 2T_{ps}}{c \cdot cb_s}
\]

For timber-to-timber connections peak stresses at the extreme fiber are slightly underestimated. For connections with steel armouring the applied stresses are slightly underestimated.
### Table D.4.1 – Summary beam-column FEM

<table>
<thead>
<tr>
<th>FEM Model</th>
<th>Change from benchmark</th>
<th>$M_{\text{con}}$ (kN.m)</th>
<th>$c/h_b$</th>
<th>$\theta_{\text{imp}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Benchmark</td>
<td>100</td>
<td>1.30</td>
<td>0.0021   0.0023 0.0023</td>
</tr>
<tr>
<td></td>
<td></td>
<td>125</td>
<td>0.92</td>
<td>0.0027   0.0029 0.0039</td>
</tr>
<tr>
<td></td>
<td></td>
<td>150</td>
<td>0.73</td>
<td>0.0035   0.0039 0.0054</td>
</tr>
<tr>
<td></td>
<td></td>
<td>175</td>
<td>0.67</td>
<td>0.0051   0.0053 0.0061</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200</td>
<td>0.48</td>
<td>0.0076   0.0074 0.0096</td>
</tr>
<tr>
<td></td>
<td></td>
<td>225</td>
<td>0.33</td>
<td>0.0131   0.0108 0.0153</td>
</tr>
<tr>
<td>2</td>
<td>Beam height reduced to 400mm.</td>
<td>160</td>
<td>0.25</td>
<td>0.0305   0.0243 0.0350</td>
</tr>
<tr>
<td></td>
<td></td>
<td>145</td>
<td>0.41</td>
<td>0.0190   0.0167 0.0198</td>
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<td></td>
<td>135</td>
<td>0.44</td>
<td>0.0165   0.0151 0.0181</td>
</tr>
<tr>
<td></td>
<td></td>
<td>125</td>
<td>0.50</td>
<td>0.0122   0.0118 0.0153</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100</td>
<td>0.56</td>
<td>0.0092   0.0095 0.0132</td>
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<td></td>
<td></td>
<td>80</td>
<td>0.81</td>
<td>0.0055   0.0064 0.0082</td>
</tr>
<tr>
<td>3</td>
<td>Half of the post-tensioning force.</td>
<td>125</td>
<td>1.00</td>
<td>0.0042   0.0051 0.0061</td>
</tr>
<tr>
<td></td>
<td></td>
<td>120</td>
<td>0.17</td>
<td>0.0146   0.0107 0.0175</td>
</tr>
<tr>
<td></td>
<td></td>
<td>115</td>
<td>0.23</td>
<td>0.0101   0.0074 0.0121</td>
</tr>
<tr>
<td></td>
<td></td>
<td>100</td>
<td>0.27</td>
<td>0.0072   0.0057 0.0099</td>
</tr>
<tr>
<td></td>
<td></td>
<td>80</td>
<td>0.44</td>
<td>0.0036   0.0032 0.0054</td>
</tr>
<tr>
<td></td>
<td></td>
<td>70</td>
<td>0.67</td>
<td>0.0020   0.0020 0.0031</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50</td>
<td>0.71</td>
<td>0.0014   0.0015 0.0028</td>
</tr>
<tr>
<td>4</td>
<td>$E_{\text{per}}$ of the column 13200MPa (aligned parallel-to-grain).</td>
<td>100</td>
<td>0.83</td>
<td>0.0008   0.0009 0.0006</td>
</tr>
<tr>
<td></td>
<td></td>
<td>150</td>
<td>0.92</td>
<td>0.0005   0.0004 0.0009</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200</td>
<td>0.75</td>
<td>0.0011   0.0007 0.0022</td>
</tr>
<tr>
<td></td>
<td></td>
<td>225</td>
<td>0.48</td>
<td>0.0021   0.0012 0.0052</td>
</tr>
<tr>
<td></td>
<td></td>
<td>250</td>
<td>0.31</td>
<td>0.0037   0.0008 0.0145</td>
</tr>
<tr>
<td>5</td>
<td>Beam height increased to 800mm.</td>
<td>150</td>
<td>0.19</td>
<td>0.0104   0.0019 0.0025</td>
</tr>
<tr>
<td>6</td>
<td>Col. height decreased to 400mm.</td>
<td>150</td>
<td>0.92</td>
<td>0.0017   0.0016 0.0041</td>
</tr>
<tr>
<td>7</td>
<td>Col. height increased to 1000mm.</td>
<td>150</td>
<td>0.77</td>
<td>0.0038   0.0031 0.0048</td>
</tr>
<tr>
<td>8</td>
<td>$E_{\text{per}}$ of the column 400MPa.</td>
<td>150</td>
<td>0.92</td>
<td>0.0032   0.0039 0.0086</td>
</tr>
<tr>
<td>9</td>
<td>$E_{\text{per}}$ of the column 1320MPa.</td>
<td>150</td>
<td>0.75</td>
<td>0.0049   0.0054 0.0025</td>
</tr>
<tr>
<td>10</td>
<td>$E_{\text{per}}$ of the column 8000MPa.</td>
<td>150</td>
<td>0.77</td>
<td>0.0024   0.0026 0.0015</td>
</tr>
<tr>
<td>11</td>
<td>$E_{\text{per}}$ of col. and beam 8000MPa.</td>
<td>150</td>
<td>0.75</td>
<td>0.0012   0.0005 0.0052</td>
</tr>
<tr>
<td>12</td>
<td>G of col. and beam 400 MPa.</td>
<td>150</td>
<td>0.75</td>
<td>0.0038   0.0037 0.0054</td>
</tr>
<tr>
<td>13</td>
<td>G = 1400MPa, $E_{\text{per}}$ = 2000MPa.</td>
<td>150</td>
<td>0.73</td>
<td>0.0042   0.0039 0.0016</td>
</tr>
<tr>
<td>14</td>
<td>70% of the post-tensioning force.</td>
<td>150</td>
<td>0.77</td>
<td>0.0016   0.0014 0.0086</td>
</tr>
<tr>
<td>15</td>
<td>150% of the post-tensioning force.</td>
<td>175</td>
<td>0.40</td>
<td>0.0071   0.0078 0.0054</td>
</tr>
<tr>
<td>16</td>
<td>Steel plate armouring: plate thickness 30mm and 100mm extrusions.</td>
<td>200</td>
<td>0.94</td>
<td>0.0013   0.0025 0.0052</td>
</tr>
<tr>
<td></td>
<td></td>
<td>225</td>
<td>0.58</td>
<td>0.0029   0.0030 0.0074</td>
</tr>
<tr>
<td></td>
<td></td>
<td>250</td>
<td>0.42</td>
<td>0.0047   0.0034 0.0115</td>
</tr>
<tr>
<td>17</td>
<td>External beam-column connection</td>
<td>125</td>
<td>0.92</td>
<td>0.0044   0.0034 0.0039</td>
</tr>
<tr>
<td></td>
<td></td>
<td>150</td>
<td>0.73</td>
<td>0.0055   0.0042 0.0054</td>
</tr>
<tr>
<td></td>
<td></td>
<td>225</td>
<td>0.33</td>
<td>0.0167   0.0122 0.0153</td>
</tr>
</tbody>
</table>

* FEM 1 and 2 are the imposed rotations obtained by Approach 1 and 2 respectively.
D.5. VERIFICATION OF THE CONNECTION PROCEEDURE

In Chapter 6, analytical modeling approaches are proposed for post-tensioned timber beam-column connections. Within this section, the predictions are compared with experimental data for several subassembly tests. The input parameters used for each subassembly are given in Table D.5.1.

D.5.1. Axial stiffness corrections for the post-tensioning

As discussed in Chapter 6, compression of the perpendicular-to-grain timber within the column reduces the strain, and therefore, the force in post-tensioning tendons. For the predictions, this was taken into account by increasing the apparent unbonded length of the post-tensioning tendons. If the connections remain elastic the following expression can applied to estimate the corrected unbonded length:

\[
I_{pt,c} \approx E_{pt} A_{pt} \left( \frac{1}{K_{beam}} + \frac{1}{K_{col}} + \frac{1}{K_{pt}} \right)
\]

Where: \( K_{beam}, K_{col}, K_{pt} \) is the axial stiffness of the beam, column and post-tensioning respectively;
\( E_{pt} \) is the elastic modulus of the tendon;
\( A_{pt} \) is the area of the tendon.

If the connection goes into the inelastic range, the above expressions will underestimate the flexibility of the column. Therefore, if the timber is inelastic the unbonded length is calibrated to approximately match the tendon force from the experimental data.

D.5.2. Experimental-Analytical Comparison

Several frame subassembly tests are considered to verify the accuracy of the proposed connection design procedure. Due to lack of experimental data, only unarmoured beam-column connections are considered. All the post-tensioned frame subassemblies
constructed to date, have been subjected to multiple quasi-static tests, which have resulted in inelastic deformation of the timber. Because of stiffness degradation, the first test on a given subassembly is most appropriate for comparison with the design procedure.

**Subassembly 1**

Subassembly 1 is an approximate two-third scale external beam-column joint. Two tests are considered; Test 1 and 3.

For Test 1, there were significant losses in tendon forces. Hence, an upper and lower bound prediction were made with an initial post-tensioning force of 31kN and 23kN per tendon respectively. The elastic modulus of the perpendicular-to-grain timber was 600MPa. The perpendicular-to-grain compressive strength of the timber was taken as 6MPa. Due to axial deformation of the timber, the unbonded length of the tendons was increased to 3000mm, to match with experimental data.

The connection moment, neutral axis depth and tendon forces are shown in Figure D.5.1. The connection moment and post-tensioning force matches well with the experimental data. The upper bound prediction appears to overestimate the moment provided by the connection at small rotations. This may be because the yield strain of the timber, according to the analytical procedure, is exceeded at a rotation of only 0.005 rad. This may have caused significant losses of post-tensioning force at even small displacement cycles. The prediction with the lower-bound post-tensioning force appears to match the initial stiffness well. As discussed in Appendix B, it is difficult to obtain an accurate neutral axis depth relationship from experimental data and therefore, inaccuracies in the experimental neutral axis depth are likely. This is verified by observing Figure D.5.1d, which shows the connection interface at 4% connection rotation. The inferred neutral axis depth from this figure is approximately 18% of the beam depth, which provides good agreement with the analytical predictions.

For Test 3, the post-tensioning force was increased to 93kN per tendon. Observing Figure D.5.2a, the connection moment is over predicted at small rotations. This is due to strength and stiffness degradation that has occurred in previous tests. Loss of post-
tensioning force (for the tendon within the neutral axis) also indicates that permanent deformation of the column face has occurred (see Figure D.5.2b). The analytical predictions indicate the yielding of timber may have begun at as low as 0.0025rad.

Figure D.5.1. Experimental – analytical comparison for Subassembly 1, Test 1A: a) Connection moment b) Tendon forces c) Neutral axis depth d) Neutral axis depth at 4% rotation
Figure D.5.2. Experimental – analytical comparison for Subassembly 1, Test 3: a) Connection moment b) Tendon forces c) Neutral axis depth d) Neutral axis depth at 4% rotation
Subassembly 2

Subassembly 2 is an approximate two-thirds scale internal beam column joint. One test is considered. The connection moment, neutral axis depth and tendon forces are shown in Figure D.5.3. The connection moment is over predicted. It is likely that this is due to significant inelastic deformation of the timber and losses in the post-tensioning force. The analytical prediction indicates that the yield stress of the timber is exceeded at only 0.0025rad. The bi-linear stress-strain law, used for the timber in compression, may need refinement when there is significant inelastic deformation. For the prediction procedure a bi-linear factor of 0.1 is assumed. To illustrate the effect of the timber stress-strain law another prediction is made with a bi-linear factor of 0, which is plotted in Figure D.5.3a. This prediction matches much more closely with the experimental data.
Figure D.5.3. Experimental – analytical comparison for Subassembly 2: a) Connection moment b) Tendon forces c) Neutral axis depth d) Beam-column connection
**Subassembly 3**

Subassembly 3 is an approximate two-thirds scale external frame. For more information refer to Palermo et al (2005a). The connection moment and tendon force is shown in Figure D.5.4. Due to errors in data acquisition, it is not possible to show the connection rotation or the neutral axis depth. Hence, the total frame drift, rather than connection rotation, is considered. Hence, the elastic deformation of the column, beam and joint panel is taken into account. There is good agreement between the analytical predictions and experimental data.

![Figure D.5.4. Experimental – analytical comparison for Subassembly 4: a) Connection moment b) Tendon forces](image)

**Figure D.5.4.** Experimental – analytical comparison for Subassembly 4: a) Connection moment b) Tendon forces
Table D.5.1 – Summary of frame subassembly tests

<table>
<thead>
<tr>
<th>Sub.</th>
<th>Test</th>
<th>Date (m/y)</th>
<th>Lb (mm)</th>
<th>H (mm)</th>
<th>h_b (mm)</th>
<th>b_b (mm)</th>
<th>h_c (mm)</th>
<th>b_c (mm)</th>
<th>E_perp (MPa)</th>
<th>f_y,t (MPa)</th>
<th>T_pt (kN)</th>
<th>E_pt (MPa)</th>
<th>I_sub,c (mm)</th>
<th>y_pt (mm)</th>
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<tbody>
<tr>
<td>1</td>
<td>1A Exterior</td>
<td>01/10</td>
<td>3000</td>
<td>2000</td>
<td>300</td>
<td>255</td>
<td>195</td>
<td>255</td>
<td>600</td>
<td>8</td>
<td>31;31</td>
<td>190000</td>
<td>3000</td>
<td>120;180</td>
</tr>
<tr>
<td>3</td>
<td>Exterior</td>
<td>01/10</td>
<td>3000</td>
<td>2000</td>
<td>300</td>
<td>255</td>
<td>195</td>
<td>255</td>
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<td>8</td>
<td>93;93</td>
<td>190000</td>
<td>3000</td>
<td>120;180</td>
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<td>1 Interior</td>
<td>01/10</td>
<td>3100</td>
<td>2000</td>
<td>300</td>
<td>216</td>
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<td>3000</td>
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<td>300</td>
<td>255</td>
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<td>600</td>
<td>8</td>
<td>60</td>
<td>190000</td>
<td>3000</td>
<td>150</td>
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</table>
APPENDIX E

MODELLING WALL SYSTEMS

This appendix concentrates on modeling the connection response of post-tensioned timber walls. Analytical predictions, that are proposed in Chapter 7 are compared with complex finite element models (FEM) and experimental data.

E.1. WALL-BASE CONNECTION RESPONSE USING FEM

Here the accuracy of analytical relationships, derived in Chapter 7, for modeling PT wall-base connections are evaluated. Firstly, the proposed relationship between the neutral axis depth, c, and the imposed connection rotation, \( \theta_{\text{imp}} \), is considered. This expression is repeated below:

\[
c = \left( \frac{2L_e \sum N}{\theta_{\text{imp}} E_{\text{para}} t_w} \right)^{0.5}
\]

Where:
- \( \sum N = T_{pt} + N \pm V_{cg} \) is the sum of the axial force from the post-tensioning, axial load and coupling elements;
- \( E_{\text{para}} \) is the parallel-to-grain elastic modulus of the timber;
- \( t_w \) is the thickness of the wall.

The effective length, \( L_e \), was calibrated as:

\[
L_e \approx 65 \left( \frac{t_w}{c} - 1 \right) \quad (\text{mm})
\]

The peak stress at the toe of the wall is:

\[
f_t = \frac{2 \sum N}{c b_h}
\]
Analytical expressions are compared with the results of sensitivity study using finite element models created in SAP 2000 (2005). The sensitivity study considers variation in several parameters, to ensure a robust evaluation of the accuracy of the analytical ($\theta_{imp-c}$) predictions and peak stresses. This study is limited to the elastic response of the wall-base connections.

**E.1.1. The finite element model**

A benchmark wall subassembly model was created, as shown in Figure D.4.1, and key parameters were altered to determine their effect on the accuracy of the $\theta_{imp-c}$ predictions and the peak stresses. The material properties and dimensions for the benchmark model are given in Table E.1.1. The geometry of the benchmark model was actually based on column dimensions. This allows a more gradual change in neutral axis depth with connection rotation, and hence, more accurate calibration.

The boundary conditions consist of a pin supports at the base of the wall, which resist only longitudinal compression and shear forces. Hence, support nodes are deleted if tension forces occur, which is an iterative process. A lateral load is applied to the top of the wall, which create a known connection moment. This lateral load is distributed evenly throughout the top of the wall section. Axial loads (730kN for the benchmark model) are applied to the top of the wall to simulate the compression provided by the post-tensioning and gravity loads. This is kept constant throughout the analysis.
Figure E.1.1. Boundary conditions and applied loads for the benchmark FEM of the wall

Table E.1.1 – Material properties and geometry of the benchmark wall model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Units</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic modulus – Parallel to grain</td>
<td>$E_{\text{para}}$</td>
<td>MPa</td>
<td>13200</td>
</tr>
<tr>
<td>Elastic modulus – Perp. to grain</td>
<td>$E_{\text{perp}}$</td>
<td>MPa</td>
<td>660</td>
</tr>
<tr>
<td>Poisson’s ratio*</td>
<td>$\nu$</td>
<td>-</td>
<td>0.3</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>$G$</td>
<td>MPa</td>
<td>600</td>
</tr>
<tr>
<td>Wall height</td>
<td>$H$</td>
<td>m</td>
<td>2.7</td>
</tr>
<tr>
<td>Wall length</td>
<td>$l_w$</td>
<td>mm</td>
<td>600</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>$t_w$</td>
<td>mm</td>
<td>380</td>
</tr>
</tbody>
</table>

*Based on research performed on Pine by Murray (2007).
E.1.2. Comparison of the model and the analytical equations

To evaluate the accuracy of the analytical $\theta_{\text{imp}}$-c relationship, the results from the FEM must be interpreted accurately. The neutral axis depth can be readily interpreted by observing the longitudinal stresses, but the imposed connection rotation is slightly more problematic. This is because the base of the wall does not remain linear. This is also reported from experimental results in Appendix B.

The connection rotation is deduced by taking the total deformation of the wall, $\Delta_{\text{tot}}$, and subtracted the member deformation. Hence:

$$\theta_{\text{imp}} = \frac{\Delta_{\text{tot}}}{H} - (\theta_f + \theta_s)$$

The flexural and shear deformation of the wall is approximated using simple beam theory. Therefore:

$$\theta_f = \frac{\Delta_f}{H} = \frac{FH^2}{3E_{\text{para}}I_w} \quad \& \quad \theta_s = \frac{\Delta_s}{H} = \frac{F}{GA_{\text{sw}}}$$

Where: $F$ is the lateral force applied to the model; $I_w$ is the second moment of inertia of the wall; $A_{\text{sw}} = \frac{2}{3}I_w t_w$ is the shear area of the wall.

Models are created with and without shear deformation. This is because the shear area used in simple beam theory is an only approximation, which can affect accuracy of the computed imposed rotation. If shear deformation is ignored, the imposed rotation at the connection can be determined more accurately. However, if the shear modulus significantly affects neutral axis depth, the accuracy of the proposed $\theta_{\text{imp}}$-c relationship may be affected. This is investigated as part of the sensitivity study.

E.1.3. Results from the sensitivity study

The sensitivity study considered many parametric variations. Only key results are presented. A summary of the study is given in Table D.4.1.


Benchmark response

The benchmark response, for a connection moment of 150kN.m, is shown below in terms of deformation, horizontal stresses, vertical stresses and shear stresses.

Figure E.1.2. Results of benchmark upside-down FEM wall: Deformed shape (magnified by 20)
The compressive stress distribution at the connection interface is shown in Figure E.1.4. This is determined by dividing the force applied by the pinned supports by the area of the each shell element. The analytical expression (shown above) for timber stresses is compared with the results of the benchmark FEM. Assuming that applied stress distribution is linear the stress in the timber can be re-written as:

\[ f_i = \frac{y}{c} \frac{2}{cb_y} \sum N \]

Where: \( y \) is distance from the neutral axis position into the compression region.

Observing Figure E.1.4 it is evident that the timber stresses are effectively linear. Hence, it is reasonable to assume both linear stress and strain in the connection design procedure.
The $\theta_{\text{imp}}$-c relationship
Several models were created (see Table D.4.1) to compare the proposed relationship for the neutral axis depth with FEM results. To gauge the accuracy of the proposed $\theta_{\text{imp}}$-c relationship, the predicted normalised neutral axis depth is plotted against that obtained from FEM (for a given connection rotation), as shown in Figure E.1.5.

In Figure E.1.5, the data is divided into models that do and do not consider shear deformation, and models that vary the material properties of the timber. The proposed neutral axis depth formulation matches well with the FEM data. The same trend is apparent with and without shear deformation in the model. Hence, the shear modulus does not significantly affect the $\theta_{\text{imp}}$-c relationship. However, there is more dispersion in the results when shear deformation is considered. It is likely that this due to inaccuracy in calculating the shear deformation, which leads to inaccuracy in interpreting the connection rotation. Changing the material properties of the timber appears to slightly influence the accuracy of the proposed $\theta_{\text{imp}}$-c relationship. Further research is necessary to determine if this is actually the case.
The proposed $\theta_{imp}$-c relationship is plotted for FEM model 7 (see Table D.4.1) and compared with three FEM data points in Figure E.1.6. The proposed $\theta_{imp}$-c relationship appears reasonable and matches well with the FEM results.
### Table E.1.2 – Summary wall FEM

<table>
<thead>
<tr>
<th>FEM Model</th>
<th>Change from benchmark</th>
<th>$M_{con}$ (kN.m)</th>
<th>Shear def. (Y/N)</th>
<th>$c/h_b$</th>
<th>$\theta_{imp}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Benchmark</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.79</td>
<td>0.00003</td>
</tr>
<tr>
<td>2 Lw = 400mm</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.44</td>
<td>0.00165</td>
</tr>
<tr>
<td>3 Lw = 400mm, 150% Tpt</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.78</td>
<td>0.00009</td>
</tr>
<tr>
<td>4 Lw = 400mm, Lcant = 5400mm</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.41</td>
<td>0.00180</td>
</tr>
<tr>
<td>5 80% Tpt</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.63</td>
<td>0.00048</td>
</tr>
<tr>
<td>6 60% Tpt</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.29</td>
<td>0.00150</td>
</tr>
<tr>
<td>7 Lw = 1000mm</td>
<td></td>
<td>225</td>
<td>N</td>
<td>0.85</td>
<td>0.00001</td>
</tr>
<tr>
<td>8 Lw = 1000mm, 60% Tpt</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.73</td>
<td>0.00001</td>
</tr>
<tr>
<td>9 Lw = 1000mm, 40% Tpt</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.38</td>
<td>0.00019</td>
</tr>
<tr>
<td>10 Lw = 1000mm, 35% Tpt</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.23</td>
<td>0.00048</td>
</tr>
<tr>
<td>11 Lw = 1000mm, 35% Tpt, tw = 180mm</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.23</td>
<td>0.00095</td>
</tr>
<tr>
<td>12 Lw = 1000mm, Lcant = 6400mm</td>
<td></td>
<td>451</td>
<td>N</td>
<td>0.20</td>
<td>0.00316</td>
</tr>
<tr>
<td>13 Lcant = 2200mm</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.79</td>
<td>0.00003</td>
</tr>
<tr>
<td>14 Lcant = 2200mm, 80% Tpt</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.58</td>
<td>0.00020</td>
</tr>
<tr>
<td>15 Lcant = 3200mm</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.79</td>
<td>0.00004</td>
</tr>
<tr>
<td>16 Lcant = 3200mm, 80% Tpt</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.63</td>
<td>0.00021</td>
</tr>
<tr>
<td>17 Lcant = 6400mm</td>
<td></td>
<td>270</td>
<td>N</td>
<td>0.21</td>
<td>0.00023</td>
</tr>
<tr>
<td>18 Lw = 400mm</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.44</td>
<td>0.00276</td>
</tr>
<tr>
<td>19 Lw = 400mm, 150% Tpt</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.75</td>
<td>0.00009</td>
</tr>
<tr>
<td>20 Lw = 400mm, Lcant = 5400mm</td>
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<td>Y</td>
<td>0.41</td>
<td>0.00311</td>
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<tr>
<td>21 67% Tpt</td>
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<td>Y</td>
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<td>0.00108</td>
</tr>
<tr>
<td>22 Lw = 1000mm</td>
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<td>300</td>
<td>Y</td>
<td>0.63</td>
<td>0.00008</td>
</tr>
<tr>
<td>23 Lw = 1000mm, 40% Tpt</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.40</td>
<td>0.00029</td>
</tr>
<tr>
<td>24 Lw = 1000mm, 35% Tpt</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.25</td>
<td>0.00077</td>
</tr>
<tr>
<td>25 Lw = 1000mm, 60% Tpt, tw = 180mm</td>
<td></td>
<td>266</td>
<td>Y</td>
<td>0.15</td>
<td>0.00688</td>
</tr>
<tr>
<td>26 Lw = 1000mm, Lcant = 6400m</td>
<td></td>
<td>451</td>
<td>Y</td>
<td>0.20</td>
<td>0.00508</td>
</tr>
<tr>
<td>27 Lw = 600mm, Lcant = 2200mm, 80% Tpt</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.58</td>
<td>0.00027</td>
</tr>
<tr>
<td>28 Lcant = 3200mm</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.75</td>
<td>0.00001</td>
</tr>
<tr>
<td>29 Lcant = 3200mm, 80% Tpt</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.58</td>
<td>0.00035</td>
</tr>
<tr>
<td>30 Lcant = 6400mm</td>
<td></td>
<td>281</td>
<td>Y</td>
<td>0.21</td>
<td>0.00947</td>
</tr>
<tr>
<td>31 Eperp = 8000MPa</td>
<td></td>
<td>225</td>
<td>N</td>
<td>0.46</td>
<td>0.00050</td>
</tr>
<tr>
<td>32 Epara = 8000MPa</td>
<td></td>
<td>225</td>
<td>N</td>
<td>0.46</td>
<td>0.00138</td>
</tr>
<tr>
<td>33 Lw = 400mm, Lcant = 5400mm, Eperp = 8000MPa</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.41</td>
<td>0.00089</td>
</tr>
<tr>
<td>34 Lw = 400mm, Lcant = 5400mm, Epara = 8000MPa</td>
<td></td>
<td>150</td>
<td>N</td>
<td>0.41</td>
<td>0.00261</td>
</tr>
<tr>
<td>35 Lw = 400mm, Lcant = 5400mm, G = 400MPa</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.41</td>
<td>0.00355</td>
</tr>
<tr>
<td>36 Lw = 400mm; Lcant = 5400mm, Isotropic E=13200</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.41</td>
<td>0.00116</td>
</tr>
<tr>
<td>37 Lw = 400mm, Lcant = 5400mm, Isotropic E=8000</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.41</td>
<td>0.00191</td>
</tr>
<tr>
<td>38 Lw = 400mm, Lcant = 5400mm, Isotropic E = 5000</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.41</td>
<td>0.00306</td>
</tr>
<tr>
<td>39 Lw = 400mm, Lcant = 5400mm, Isotropic, $\nu = 0$</td>
<td></td>
<td>150</td>
<td>Y</td>
<td>0.41</td>
<td>0.00124</td>
</tr>
</tbody>
</table>
E.2. VERIFICATION OF THE CONNECTION PROCEEDURE

In Chapter 7, analytical modeling approaches are proposed for post-tensioned timber wall-base connections. Within this section, the predictions are compared with experimental data from subassembly tests (Iqbal et al. 2007; Smith et al. 2007). Rather than considering the connection rotation, which is difficult to accurately define from experimental data (see Appendix B), the total drift of the subassembly is considered. Hence, the analytical predictions consider the flexural and shear deformation of the wall elements. The input parameters used for each subassembly are given in Table E.2.1.

Subassembly 1

Subassembly 1 is an approximate two-third scale post-tensioned wall. The connection moment, neutral axis depth and tendon forces are shown in Figure E.2.1. The connection moment and post-tensioning force matches well with the experimental data. As discussed in Appendix B, it is difficult to obtain an accurate neutral axis depth relationship from experimental data due to the non-linear displacement at the base of the wall. Therefore, there are likely to be inaccuracies in the experimental neutral axis depth. Furthermore, due to experimental calibration error the neutral axis depth could only be calculated for positive drifts. However, the accuracy of the predicted neutral axis depth is verified by observing Figure D.5.1d, which shows the connection interface at 2.5% drift. The inferred neutral axis depth from this figure is approximately 10% of the wall length, which is in agreement with the analytical predictions.

At 2.5% drift, the peak stress is only 60% of the yield stress of the timber parallel-to-grain. Hence, in many situations it may be possible to avoid any yielding in the wall elements.
Figure E.2.1. Experimental – analytical comparison for Subassembly 1: a) Connection moment b) Tendon forces c) Neutral axis depth d) Neutral axis depth at 2.5% drift
Subassembly 2

Subassembly 2 is another two-third scale post-tensioned wall. The connection moment and tendon forces are shown in Figure E.2.2. The neutral axis depth is not compared, due to errors associated with interpreting the neutral axis depth from experimental data (see Appendix B) and experimental calibration errors. The connection moment and post-tensioning force matches well with the experimental data. There was some wall sliding during the experiment, which accounts for the differences in connection moment between the experimental data and the prediction at low rotations.

![Figure E.2.2. Experimental – analytical comparison for Subassembly 1: a) Connection moment b) Tendon forces](image)

**Figure E.2.2. Experimental – analytical comparison for Subassembly 1: a) Connection moment b) Tendon forces**

<table>
<thead>
<tr>
<th>Sub.</th>
<th>Test</th>
<th>Date</th>
<th>l_w</th>
<th>t_w</th>
<th>H</th>
<th>E_{para}</th>
<th>G</th>
<th>f_{y,t}</th>
<th>T_{pt}</th>
<th>E_{pt}</th>
<th>l_{ub,c}</th>
<th>y_{pt}</th>
</tr>
</thead>
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<td>1</td>
<td>8</td>
<td>10/06</td>
<td>790</td>
<td>195</td>
<td>2000</td>
<td>10.5</td>
<td>600</td>
<td>45</td>
<td>43;43</td>
<td>190</td>
<td>3200</td>
<td>160;640</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>08/06</td>
<td>790</td>
<td>195</td>
<td>2000</td>
<td>10.5</td>
<td>600</td>
<td>45</td>
<td>43;43</td>
<td>190</td>
<td>3400</td>
<td>160;640</td>
</tr>
</tbody>
</table>
This appendix provides a summary of the earthquake records used within thesis for numerical time-history analysis. Details are provided on location, distance to source and the employed scaling approaches.

**F.1. EARTHQUAKE SET 1**

Earthquake set 1 consists of 15 earthquakes, with a mixture near field and far field records. The characteristics of the earthquake records are provided in Table F.1.1. The records were chosen to satisfy, and scaled according to, New Zealand Standards criteria (NZS1170.5 2004) within a period range of 0.64 to 3s. This was to match with case study building used in Chapter 8 to determine the effects of floor flexibility. The scaling factors for 2% damping and 1/500 year, Wellington City, Soil type C, earthquake intensity are shown in Table F.1.1. For 2% damping the spectral response varies significantly and therefore the limitations on scaling (0.3 to 3) according to NZS1170.5 (2004) were violated for 3 earthquakes. The goodness-of-fit criteria according to NZS1170.5 (2004), which specify that the normalized sum of the lognormal difference between the earthquake spectra and the design spectra must be less than $\log_{10}(1.5)$, were satisfied for all earthquakes. The scaled displacement and acceleration spectra for 2% of critical damping are shown in Figure D.1.1, Figure F.1.2 and Figure F.1.3.
Table F.1.1 – Properties of earthquake set 1

<table>
<thead>
<tr>
<th>EQ Number</th>
<th>Near field/ Far field</th>
<th>Location</th>
<th>Distance to source (km)</th>
<th>Soil type (NEHRP)</th>
<th>Component</th>
<th>PGA (g)</th>
<th>Scale factor ξel = 2% 1/500 year</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>NF</td>
<td>Northridge, Los Angeles Dam</td>
<td>5.92</td>
<td>A</td>
<td>334</td>
<td>0.35</td>
<td>1.2</td>
</tr>
<tr>
<td>2</td>
<td>NF</td>
<td>Northridge, Sylmar - Olive view MedCtr</td>
<td>5.3</td>
<td>D</td>
<td>360</td>
<td>0.84</td>
<td>0.6</td>
</tr>
<tr>
<td>3</td>
<td>FF</td>
<td>Tabas, Iran (BOS_L1)</td>
<td>26.1</td>
<td>D</td>
<td>LP1 (P)</td>
<td>0.107</td>
<td>4.49</td>
</tr>
<tr>
<td>4</td>
<td>FF</td>
<td>Northridge, N Hollywood – Coldwater Can</td>
<td>14.6</td>
<td>C</td>
<td>270</td>
<td>0.271</td>
<td>2.18</td>
</tr>
<tr>
<td>5</td>
<td>FF</td>
<td>Northridge, Canoga Park – Topanga Clan</td>
<td>15.8</td>
<td>D</td>
<td>106</td>
<td>0.356</td>
<td>1.82</td>
</tr>
<tr>
<td>6*</td>
<td>N/A</td>
<td>Artificial record</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.53</td>
<td>1.22</td>
</tr>
<tr>
<td>7</td>
<td>FF</td>
<td>Superstition Hills, El Centro Imp. Co. Cent</td>
<td>13.9</td>
<td>D</td>
<td>0</td>
<td>0.2899</td>
<td>2.16</td>
</tr>
<tr>
<td>8</td>
<td>FF</td>
<td>Superstition Hills, Plaster City</td>
<td>21</td>
<td>D</td>
<td>135</td>
<td>0.155</td>
<td>4.49</td>
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<tr>
<td>9</td>
<td>FF</td>
<td>Cape Mendocino, Fortuna Fortuna Blvd</td>
<td>23.6</td>
<td>C</td>
<td>0</td>
<td>0.116</td>
<td>2.68</td>
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<tr>
<td>10</td>
<td>NF</td>
<td>Loma Prieta, Los Gatos Pres Center</td>
<td>3.88</td>
<td>A</td>
<td>0</td>
<td>0.563</td>
<td>0.55</td>
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<td>11</td>
<td>FF</td>
<td>Northridge, LA – N Faring Rd</td>
<td>23.9</td>
<td>D</td>
<td>0</td>
<td>0.273</td>
<td>3.22</td>
</tr>
<tr>
<td>12</td>
<td>NF</td>
<td>Chi Chi, TCU068</td>
<td>9.96</td>
<td>D</td>
<td>270</td>
<td>0.57</td>
<td>0.56</td>
</tr>
<tr>
<td>13</td>
<td>FF</td>
<td>Landers, Yemo Fire Station</td>
<td>24.9</td>
<td>D</td>
<td>0</td>
<td>0.2095</td>
<td>2.65</td>
</tr>
<tr>
<td>14</td>
<td>FF</td>
<td>Loma Prieta, Hollister Diff. Army</td>
<td>25.8</td>
<td>D</td>
<td>0</td>
<td>0.2762</td>
<td>1.29</td>
</tr>
<tr>
<td>15</td>
<td>NF</td>
<td>Tabas, Iran</td>
<td>2</td>
<td>D</td>
<td>TR</td>
<td>0.852</td>
<td>0.85</td>
</tr>
</tbody>
</table>

*Earthquake 6 is an artificial record using SIMQKE
Appendix F – Earthquake records

Figure F.1.1. Set 1 spectra for EQ 1 to 5: a) Acceleration b) Displacement

Figure F.1.2. Set 1 spectra for EQ 6 to 10: a) Acceleration b) Displacement

Figure F.1.3. Set 1 spectra for EQ 11 to 15: a) Acceleration b) Displacement
F.2. EARTHQUAKE SET 2

Earthquake set 2 consists of 15 earthquakes records that are a mixture near field and far events. Several earthquakes from Set 1 were also used in Set 2. However, earthquake Set 1 under represented the design spectral demand at low periods. Hence, both far field and near field records were added which increase the spectral demand at low periods. This is to ensure that higher mode amplification factors, developed in Chapter 10, are conservative. The characteristics of the additional five earthquakes are provided in Table F.2.1.

The records were chosen to satisfy, and scaled according to, New Zealand Standards criteria (NZS1170.5 2004) within a period range of 0.2 to 3.5s. This range was expected to encompass the modal periods that would significantly contribute to the response of the frame and wall models from Chapter 10 and 11. The scaling factors for 2% and 5% damping and 1/500 year earthquake, Wellington City, Soil type C, intensity are shown in Table F.2.2. The displacement and acceleration spectra for each record, for 5% of critical damping, are shown in Figure F.2.1, Figure F.2.2 and Figure F.2.3.
# Appendix F – Earthquake records

## Table F.2.1 – Properties of earthquake set 2

<table>
<thead>
<tr>
<th>EQ Number</th>
<th>Near field/ Far field</th>
<th>Location</th>
<th>Distance to source (km)</th>
<th>Soil type (NEHRP)</th>
<th>Component</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>NF</td>
<td>Northridge, Los Angeles Dam</td>
<td>5.92</td>
<td>A</td>
<td>334</td>
<td>0.35</td>
</tr>
<tr>
<td>2</td>
<td>NF</td>
<td>Northridge, Sylmar - Olive view Med Ctr</td>
<td>5.3</td>
<td>D</td>
<td>360</td>
<td>0.84</td>
</tr>
<tr>
<td>3</td>
<td>FF</td>
<td>Tabas, Iran (BOS_L1)</td>
<td>26.1</td>
<td>D</td>
<td>LP1 (P)</td>
<td>0.107</td>
</tr>
<tr>
<td>4</td>
<td>FF</td>
<td>Northridge, N Hollywood – Coldwater Can</td>
<td>14.6</td>
<td>C</td>
<td>270</td>
<td>0.271</td>
</tr>
<tr>
<td>5</td>
<td>FF</td>
<td>Northridge, Canoga Park – Topanga Clan</td>
<td>15.8</td>
<td>D</td>
<td>106</td>
<td>0.356</td>
</tr>
<tr>
<td>6*</td>
<td>N/A</td>
<td>Artificial record</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.53</td>
</tr>
<tr>
<td>7</td>
<td>FF</td>
<td>Superstition Hills, El Centro Imp. Co. Cent</td>
<td>13.9</td>
<td>D</td>
<td>0</td>
<td>0.2899</td>
</tr>
<tr>
<td>8</td>
<td>FF</td>
<td>Superstition Hills, Plaster City</td>
<td>21</td>
<td>D</td>
<td>135</td>
<td>0.155</td>
</tr>
<tr>
<td>9</td>
<td>FF</td>
<td>Northridge, LA – N Faring Rd</td>
<td>23.9</td>
<td>D</td>
<td>0</td>
<td>0.273</td>
</tr>
<tr>
<td>10</td>
<td>FF</td>
<td>Landers, Yemo Fire Station</td>
<td>24.9</td>
<td>D</td>
<td>0</td>
<td>0.2095</td>
</tr>
<tr>
<td>11</td>
<td>NF</td>
<td>Tabas, Iran</td>
<td>2</td>
<td>D</td>
<td>TR</td>
<td>0.852</td>
</tr>
<tr>
<td>12</td>
<td>FF</td>
<td>Superstition Hills, Brawley</td>
<td>18.2</td>
<td>D</td>
<td>-</td>
<td>0.134</td>
</tr>
<tr>
<td>13</td>
<td>NF</td>
<td>Dinar, Turkey</td>
<td>3.4</td>
<td>C</td>
<td>180</td>
<td>0.285</td>
</tr>
<tr>
<td>14</td>
<td>FF</td>
<td>LA - Hollywood Stor FF</td>
<td>25.5</td>
<td>D</td>
<td>90</td>
<td>0.231</td>
</tr>
<tr>
<td>15</td>
<td>FF</td>
<td>El Cent. Imp. Val. Niland Fire Station</td>
<td>35.6</td>
<td>D</td>
<td>90</td>
<td>0.087</td>
</tr>
</tbody>
</table>

*Earthquake 6 is an artificial record using SIMQKE
Figure F.2.1. Set 2 spectra for EQ 1 to 5 for 5% damping: a) Acceleration b) Displacement

Figure F.2.2. Set 2 spectra for EQ 6 to 10 for 5% damping: a) Acceleration b) Displacement

Figure F.2.3. Set 2 spectra for EQ 11 to 15 for 5% damping: a) Acceleration b) Displacement
## Table F.2.2 – Scale factors for earthquake set 2

<table>
<thead>
<tr>
<th>EQ Number</th>
<th>Scale factor ( \xi_{\text{el}} = 5% ) 1/500 year</th>
<th>Scale factor ( \xi_{\text{el}} = 2% ) 1/500 year</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.93</td>
<td>1.23</td>
</tr>
<tr>
<td>2</td>
<td>0.47</td>
<td>0.62</td>
</tr>
<tr>
<td>3</td>
<td>3.46</td>
<td>4.58</td>
</tr>
<tr>
<td>4</td>
<td>1.63</td>
<td>2.15</td>
</tr>
<tr>
<td>5</td>
<td>1.40</td>
<td>1.85</td>
</tr>
<tr>
<td>6</td>
<td>0.95</td>
<td>1.25</td>
</tr>
<tr>
<td>7</td>
<td>1.68</td>
<td>2.22</td>
</tr>
<tr>
<td>8</td>
<td>3.46</td>
<td>4.58</td>
</tr>
<tr>
<td>9</td>
<td>2.57</td>
<td>3.41</td>
</tr>
<tr>
<td>10</td>
<td>2.04</td>
<td>2.69</td>
</tr>
<tr>
<td>11</td>
<td>0.59</td>
<td>0.79</td>
</tr>
<tr>
<td>12</td>
<td>3.65</td>
<td>4.83</td>
</tr>
<tr>
<td>13</td>
<td>1.51</td>
<td>2.00</td>
</tr>
<tr>
<td>14</td>
<td>2.37</td>
<td>3.14</td>
</tr>
<tr>
<td>15</td>
<td>4.67</td>
<td>6.00</td>
</tr>
</tbody>
</table>
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe
APPENDIX G

DESIGNS FOR TIME-HISTORY ANALYSIS VERIFICATION

Within this appendix the design of post-tensioned timber frame and wall buildings from Chapter 10 and 11 is documented. These designs are divided post-tensioned timber frames, cantilever walls and coupled walls. The lateral force design, member design and connection design are provided.

G.1. FRAMES

As described in Chapter 10, three frame geometries were considered, termed frame 1, 2 and 3. The frames are designed using lateral force design procedures from Chapter 9 and frame design procedures from Chapter 6.

G.1.1. Lateral force design

The lateral force design procedure is shown below for frame 1. Because a similar approach was followed for frame 2 and 3 for brevity only the results are provided herein.
Frame 1

Step 1: The design displacement $\Delta_d$, the effective mass $m_e$ and effective height $H_e$ is determined.

The peak design displacement for the SDOF representation:

$$\Delta_d = \frac{\sum_{i=1}^{n}(m_i \Delta_i^2)}{\sum_{i=1}^{n}(m_i \Delta_i)}$$

Where:

$$\Delta_i = \delta_i \frac{\Delta_i}{\delta_i}$$

And:

$$\frac{\delta_i}{\delta_i} = \frac{H_i}{H_n}$$ is the mode shape (linear displacement profile)

So:

$$\Delta_i = \frac{H_i \Delta_i}{H_n}$$

And:

$$\Delta_i = H_i \cdot \theta_d = 3.81 \times 0.02 = 0.0762m$$

The effective mass:

$$m_e = \frac{\sum_{i=1}^{n}(m_i \Delta_i)}{\Delta_d}$$

The effective height:

$$H_e = \frac{\sum_{i=1}^{n}(m_i \Delta_i, H_i)}{\sum_{i=1}^{n}(m_i \Delta_i)}$$

Table G.1.1 – DBD calculations

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Height, Hi (m)</th>
<th>Weight, wi (KN)</th>
<th>Mass, mi (tonnes)</th>
<th>$\Delta_i$ (m)</th>
<th>$m_i \Delta_i$</th>
<th>$m_i \Delta_i^2$</th>
<th>$m_i \Delta_i^* H_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>11.43</td>
<td>630</td>
<td>64.2</td>
<td>0.229</td>
<td>14.7</td>
<td>3.36</td>
<td>167.8</td>
</tr>
<tr>
<td>2</td>
<td>7.62</td>
<td>693</td>
<td>70.6</td>
<td>0.152</td>
<td>10.8</td>
<td>1.64</td>
<td>82.0</td>
</tr>
<tr>
<td>1</td>
<td>3.81</td>
<td>693</td>
<td>70.6</td>
<td>0.076</td>
<td>5.4</td>
<td>0.41</td>
<td>20.5</td>
</tr>
<tr>
<td>Sum</td>
<td>3360</td>
<td>205.4</td>
<td></td>
<td>30.9</td>
<td>5.41</td>
<td>270.3</td>
<td></td>
</tr>
</tbody>
</table>

AR-G-2
Therefore:
\[ \Delta_d = \sum_{i=1}^{n} \left( m_i \Delta_i \right)^2 / \sum_{i=1}^{n} (m_i \Delta_i) = \frac{5.41}{30.9} = 0.175m \]

\[ m_e = \sum_{i=1}^{n} (m_i \Delta_i) / \Delta_d = \frac{30.9}{0.175} = 176\text{tonne} \quad (85\% \text{ of the total mass}) \]

\[ H_e = \sum_{i=1}^{n} (m_i \Delta_i H_i) / \sum_{i=1}^{n} (m_i \Delta_i) = \frac{270.3}{30.9} = 8.77m \quad (77\% \text{ of the total height}) \]

**Step 2:** Determine the system damping

An elastic damping, \( \xi_{el} \), of 5\% of critical damping is assumed, based on findings from Chapter 9.

\[ \xi_{eq} = \xi_{el} + \xi_{hyst} = 5\% + 0 = 5\% \]

**Step 3:** Determine the effective period from the design displacement spectrum

By entering the displacement spectrum with the design displacement the effective period is obtained: \( T_e = 1.37s \).
**Step 4:** Obtain the equivalent lateral stiffness

\[ K_e = \frac{4\pi^2 m_e}{T_e^2} = 4\pi^2 \frac{176}{1.37^2} = 3691 \text{ kN/m} \]

**Step 5:** Determine the base shear

\[ V_b = K_e \Delta_d = 3691 \times 0.175 = 647 \text{ kN} \]

**Step 6:** Distribute the base shear up the structure

The base shear is distributed up the structure as equivalent lateral forces, which are proportional to the mass and displacement at each floor. Priestley *et al.* (2007) suggests that an additional allocation of force shall be added to roof, to account of higher modes of vibration. However, this is only suggested for taller frames. This measure is said to be conservative for frames below 10 storeys. To be consistent with the specified displacement profiles (see Chapter 9), an additional 10% of the base shear is added to the roof level for frames over 4 storeys. Therefore, for Frame 1:

\[ F_i = V_b \left( m_i \Delta_i \right) / \sum_{i=1}^{n} (m_i \Delta_i) \]

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Floor Force (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>308</td>
</tr>
<tr>
<td>2</td>
<td>226</td>
</tr>
<tr>
<td>1</td>
<td>113</td>
</tr>
<tr>
<td><strong>Sum (Vb)</strong></td>
<td><strong>647</strong></td>
</tr>
</tbody>
</table>
Frame 2
The resultant base shear and distributed lateral forces from the displacement-based design for Frame 2 are given Table G.1.3. The frame has effective period of 2.03s.

Table G.1.3 – Design lateral forces for Frame 2

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Floor Force (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>288</td>
</tr>
<tr>
<td>5</td>
<td>191</td>
</tr>
<tr>
<td>4</td>
<td>161</td>
</tr>
<tr>
<td>3</td>
<td>127</td>
</tr>
<tr>
<td>2</td>
<td>89</td>
</tr>
<tr>
<td>1</td>
<td>46</td>
</tr>
<tr>
<td><strong>Sum (Vb)</strong></td>
<td><strong>901</strong></td>
</tr>
</tbody>
</table>

Frame 3
The resultant base shear and distributed lateral forces from the displacement-based design for Frame 2 are given Table G.1.4. The frame has effective period of 3.26s, which is above the corner period from NZS1170.5. To determine the effective period, a linear projection of the displacement spectrum below the corner period was considered, as discussed in Chapter 9.

Table G.1.4 – Design lateral forces for Frame 3

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Floor Force (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>218</td>
</tr>
<tr>
<td>9</td>
<td>129</td>
</tr>
<tr>
<td>8</td>
<td>118</td>
</tr>
<tr>
<td>7</td>
<td>107</td>
</tr>
<tr>
<td>6</td>
<td>94</td>
</tr>
<tr>
<td>5</td>
<td>81</td>
</tr>
<tr>
<td>4</td>
<td>67</td>
</tr>
<tr>
<td>3</td>
<td>51</td>
</tr>
<tr>
<td>2</td>
<td>35</td>
</tr>
<tr>
<td>1</td>
<td>18</td>
</tr>
<tr>
<td><strong>Sum (Vb)</strong></td>
<td><strong>918</strong></td>
</tr>
</tbody>
</table>
G.1.2. Determination of frame actions

The moments and shears throughout the frame must be determined. To do this an equilibrium-based approach (Priestley et al. 2007) is used. As for the lateral force design, the procedure for determining the frame actions is only shown for Frame 1. Key results are provided for Frame 2 and 3.

Frame 1

Step 1: Determine the total overturning moment (OTM)

\[ OTM = \sum_{i=1}^{n} F_i H_i \]

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Hi (m)</th>
<th>Fi (kN)</th>
<th>Fi.Hi (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>11.43</td>
<td>308</td>
<td>3524</td>
</tr>
<tr>
<td>2</td>
<td>7.62</td>
<td>226</td>
<td>1723</td>
</tr>
<tr>
<td>1</td>
<td>3.81</td>
<td>113</td>
<td>431</td>
</tr>
<tr>
<td>Sum</td>
<td>V_b=647</td>
<td>OTM=5677</td>
<td></td>
</tr>
</tbody>
</table>

Step 2: Decide how much of the OTM will be taken by the column bases:

\[ OTM = \sum_{j=1}^{n} M_{c,j} + TL_{base} \]

Where: \( \sum_{j=1}^{n} M_{c,j} \) = the sum all column-base moments

\( T \) = the tension force induced by lateral load in the exterior column

\( L_{base} \) = the total frame length (to column centrelines)

Hence, the sum of the column base moments must be decided. As mentioned in Chapter 10, it is decided that the base connections have a contra-flexure point at approximately 60% the first interstorey height.
Hence:

$$\sum_{j=1}^{n} M_{c,j} = 0.6V_bH_1 = 1479kN.m$$

**Step 3:** Determine the remaining tension and compression force in the exterior columns:

$$T = C = \frac{OTM - \sum_{j=1}^{n} M_{c,j}}{L_{\text{base}}} = \frac{5677 - 1479}{35} = 120.0kN$$

**Step 4:** Proportion the seismic axial forces to each beam up the height of the building

A rational way to proportion the seismic axial forces into beam shears, is to use the total shear force diagram (Priestley et al. 2007). This will ensure that the assumed displacement profile is maintained. Hence:

$$V_{B,i} = T \frac{V_{S,i}}{\sum_{i=1}^{n} V_{S,i}}$$

Where: $V_{B,i}$ = the beam shear at the $i^{th}$ floor

<table>
<thead>
<tr>
<th>Storey, $i$</th>
<th>$F_i$ (kN)</th>
<th>$V_{si}$ (kN)</th>
<th>$V_{Bi}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>308</td>
<td>308</td>
<td>24.8</td>
</tr>
<tr>
<td>2</td>
<td>226</td>
<td>534</td>
<td>43.0</td>
</tr>
<tr>
<td>1</td>
<td>113</td>
<td>647</td>
<td>52.1</td>
</tr>
<tr>
<td><strong>Sum</strong></td>
<td>$V_b=647$</td>
<td>$\sum V_{si}=1489$</td>
<td>$\sum V_{Bi}=120.0$</td>
</tr>
</tbody>
</table>

**Step 5:** Calculate beam design moments

$$M_{B,j} = V_{B,j} \frac{L_b}{2} \text{ (At the column centerlines)}$$

Where $L_b$ = the length of the bay from column centerline to column centerline (7m)
The beam-column connection design moments at the column face are:

\[ M_{\text{con},j} = M_{B,j} \frac{L_n - h_c}{L_h} \]

Where \( h_c \) is the column width (600mm assumed).

**Table G.1.7 – Calculation of beam moments**

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>VBi (kN)</th>
<th>MBi (kN.m)</th>
<th>Mcon (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>24.8</td>
<td>86.8</td>
<td>79.4</td>
</tr>
<tr>
<td>2</td>
<td>43.0</td>
<td>151</td>
<td>138</td>
</tr>
<tr>
<td>1</td>
<td>52.1</td>
<td>182</td>
<td>167</td>
</tr>
</tbody>
</table>

**Step 6: Calculate column design moments**

The sum of the column moments above and below a given floor must equal the sum of the beam moments.

\[ \sum M_{c,j,\text{above}} + \sum M_{c,j,\text{below}} = \sum M_{B,j} \]

It is reasonable to assume that the column moments, immediately above and below the \( i^{th} \) floor, are equal. Making this assumption, the following equations are appropriate:

For the 1\(^{st}\) and 2\(^{nd}\) storey:

\[ \sum_{j=1}^{n_b} M_{C,j} = n_b M_{B,j} \]

For the 3\(^{rd}\) storey (the Roof):

\[ \sum_{j=1}^{n_b} M_{C,j} = 2n_b M_{B,i} \]

Where \( n_b \) is the number of bays.

**Table G.1.8 – Calculation of total column moments**

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>MBi (kN.m)</th>
<th>( \sum M_c ) (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>79.4</td>
<td>868</td>
</tr>
<tr>
<td>2</td>
<td>138</td>
<td>753</td>
</tr>
<tr>
<td>1</td>
<td>167</td>
<td>912</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>1479</td>
</tr>
</tbody>
</table>
The worst-case column moment is at the base of the frame. By equilibrium, the moments induced in the interior and exterior columns can be determined:

\[
M_{c,\text{int}} = \sum_{j=1}^{n} M_{C,j} / n_b
\]

\[
M_{c,\text{ext}} = \sum_{j=1}^{n} M_{C,j} / 2n_b
\]

| Table G.1.9 – Calculation of interior and exterior column moments |
|---------------------|-----------------|------------------|------------------|
| Storey, i | \( M_{c,\text{ext}} \) (kN.m) | \( M_{c,\text{int}} \) (kN.m) |
| 3 | 86.8 | 174 |
| 2 | 75.2 | 151 |
| 1 | 92.2 | 184 |
| 0 | 148 | 296 |

**Frame 2**

The distributed actions for Frame 2 are given Table G.1.10. It is assumed that the column depth is 700mm.

| Table G.1.10 – Design actions for Frame 2 |
|---------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Storey, i | \( V_{Bi} \) (kN) | \( M_{Bi} \) (kN.m) | \( M_{con} \) (kN.m) | \( \sum M_c \) (kN.m) | \( M_{c,\text{ext}} \) (kN.m) | \( M_{c,\text{int}} \) (kN.m) |
| 6 | 27.0 | 94.5 | 85.0 | 945 | 94.5 | 189 |
| 5 | 44.9 | 157 | 142 | 786 | 78.6 | 157 |
| 4 | 60.0 | 210 | 189 | 1051 | 105 | 210 |
| 3 | 71.9 | 252 | 227 | 1259 | 126 | 252 |
| 2 | 80.3 | 281 | 253 | 1405 | 140 | 281 |
| 1 | 84.6 | 296 | 266 | 1481 | 148 | 296 |
| 0 | - | - | - | 2060 | 412 | 206 |
Frame 3

The distributed actions for Frame 3 are given Table G.1.10. Again, it is assumed that the column depth is 700mm.

Table G.1.11 – Design actions for Frame 3

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>VBi (kN)</th>
<th>MBi (kN.m)</th>
<th>Mcon (kN.m)</th>
<th>$\sum$Mc (kN.m)</th>
<th>Mc,ext (kN.m)</th>
<th>Mc,int (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>21.7</td>
<td>75.9</td>
<td>68.3</td>
<td>759</td>
<td>75.9</td>
<td>152</td>
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<tr>
<td>9</td>
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<td>604</td>
<td>60.4</td>
<td>121</td>
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<td>146</td>
<td>811</td>
<td>81.1</td>
<td>162</td>
</tr>
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<td>7</td>
<td>57.0</td>
<td>199</td>
<td>179</td>
<td>997</td>
<td>99.7</td>
<td>199</td>
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<td>1161</td>
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<td>232</td>
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<tr>
<td>5</td>
<td>74.4</td>
<td>260</td>
<td>234</td>
<td>1302</td>
<td>130</td>
<td>260</td>
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<td>4</td>
<td>81.0</td>
<td>284</td>
<td>255</td>
<td>1418</td>
<td>142</td>
<td>284</td>
</tr>
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<td>3</td>
<td>86.1</td>
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<td>271</td>
<td>1507</td>
<td>151</td>
<td>302</td>
</tr>
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<td>2</td>
<td>89.6</td>
<td>314</td>
<td>282</td>
<td>1569</td>
<td>157</td>
<td>314</td>
</tr>
<tr>
<td>1</td>
<td>91.4</td>
<td>320</td>
<td>288</td>
<td>1600</td>
<td>160</td>
<td>320</td>
</tr>
<tr>
<td>0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>2099</td>
<td>420</td>
<td>210</td>
</tr>
</tbody>
</table>
G.1.3. Frame and post-tensioning design

As for the lateral force design, only the design of the Frame 1 is described in full. Key results are provided for Frame 2 and 3.

Frame 1

Step 1: Estimate frame geometry

As mentioned in Chapter 10, it is assumed that the frame geometry is constant up the entire height of the building. A solid section of 300mm wide by 600mm deep is assumed for both the beam and column.

Step 2: Evaluate member deformation

The deformation of the beam, column and joint panel at every level can be determined using equations from Chapter 6. The resulting member deformation of each level is given in Table G.1.12.

For the beam and column deformation for each level is:

\[
\theta_b = \phi_b \left( \frac{(L_b - h_c)^2}{6} + \frac{E_t}{G} \frac{h_b^2}{4} \right)
\]

\[
\theta_c = \phi_c \left( \frac{(H - h_b)^2}{6} + \frac{E_t}{G} \frac{h_c^2}{4} \right)
\]

Where: \(E_t\) and \(G\) are the bending and shear elastic modulus respectively;
\(L_b\) is the length of the bay;
\(H\) is interstorey height;
\(h_b\) and \(h_c\) is the depth of the beam and column respectively.

The bending and shear modulus is taken as 11000 MPa and 600MPa respectively. The curvature in the beam and column is a function of the applied moment in the connection \((M_{con})\) and the frame geometry:

\[
\phi_b = \frac{M_{con}}{E_t I_b} \quad & \quad \phi_c = \frac{M_{con}}{E_t I_c} \left( \frac{L_b}{H} - \frac{h_b}{h_c} \right)
\]

G-11
For joint panel deformation:

\[ \theta_j = \frac{V_{jh}}{GA_{sh}} \left( 1 - \frac{h_c}{L_b} - \frac{h_b}{H} \right) \]

Where:

\[ V_{jh} = \frac{2M_{con}}{h_b} - V_{col} ; \]
\[ A_{sh} = b_ch_b . \]

Note that at the roof level, the column moment is approximately double the connection moment but only half an interstorey height is deforming. Hence, the above equation for column deformation is also appropriate for the roof level, provided that the column curvature is also calculated as shown above. However, this assumes that the contraflexure point is at the half height of the column.

Therefore, the total member deformation is:

\[ \theta_{mem} = \theta_c + \theta_b + \theta_j \]

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>( \theta_b ) (rad)</th>
<th>( \theta_c ) (rad)</th>
<th>( \theta_j ) (rad)</th>
<th>( \theta_{mem} ) (rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.0016</td>
<td>0.0022</td>
<td>0.0015</td>
<td>0.0053</td>
</tr>
<tr>
<td>2</td>
<td>0.0028</td>
<td>0.0019</td>
<td>0.0027</td>
<td>0.0074</td>
</tr>
<tr>
<td>1</td>
<td>0.0034</td>
<td>0.0023</td>
<td>0.0032</td>
<td>0.0089</td>
</tr>
</tbody>
</table>

**Step 3: Determine the allowable connection rotation**

The difference between the allowable drift per floor and member deformation, gives the allowable connection rotation, which is used for connection design.

\[ \theta_{imp} \leq \frac{\theta_d - (\theta_b + \theta_c + \theta_j)}{\left( 1 - \frac{h_c}{L_b} \right)} \]
Step 4: Evaluate connection moment capacity

The connection moment is evaluated, using modeling procedures from Chapter 6, repeated here as Figure G.1.2. The connections are designed with half of the section comprised of parallel-to-grain timber armouring. One post-tensioning tendon group is positioned at the half height of the beam section. The connections were designed to remain essentially elastic until the design drift of 2%. A bi-linear stress-strain law is applied for the timber in compression, with a yield stress of 45MPa.
The moment-rotation curves for the beam-column connection on each level are shown in Figure G.1.3. Each curve satisfies the design connection moment and rotation from above. The post-tensioning area and force from each analysis is given in Table G.1.14.

![Graph of moment-rotation curves for each floor](image)

**Figure G.1.3. Connection response for frame 1**

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Apt (mm$^2$)</th>
<th>Tpt (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>693</td>
<td>638</td>
</tr>
<tr>
<td>2</td>
<td>495</td>
<td>494</td>
</tr>
<tr>
<td>3</td>
<td>297</td>
<td>246</td>
</tr>
</tbody>
</table>

**Step 5: Detailed design**

The flexural and shear capacity of the beam and column is checked for ultimate limit state loading. Because this is not a critical step for defining the frame models for time-history analysis, the detailed design is not shown here. In general, the deflection criteria are more critical than the ultimate limit state strength demands.
Frame 2

For frame 2, the same process (shown above for Frame 1) is repeated. The design results in a solid section of 300mm wide by 700mm deep for both the beam and column. Again, the beam-column connections are armoured with parallel-to-grain timber, which is half the width of the column. Further information is given in Figure G.1.4 and Table G.1.15.

![Figure G.1.4. Connection response for frame 2](image-url)

### Figure G.1.4. Connection response for frame 2

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>( \theta_{\text{imp}} ) (rad)</th>
<th>Apt (mm(^2))</th>
<th>Tpt (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>0.0179</td>
<td>297</td>
<td>196</td>
</tr>
<tr>
<td>5</td>
<td>0.0165</td>
<td>396</td>
<td>389</td>
</tr>
<tr>
<td>4</td>
<td>0.0146</td>
<td>594</td>
<td>565</td>
</tr>
<tr>
<td>3</td>
<td>0.0131</td>
<td>792</td>
<td>717</td>
</tr>
<tr>
<td>2</td>
<td>0.0121</td>
<td>891</td>
<td>862</td>
</tr>
<tr>
<td>1</td>
<td>0.0115</td>
<td>891</td>
<td>931</td>
</tr>
</tbody>
</table>
Frame 3

For Frame 3, a solid section 300mm wide by 700mm deep is required for both the beam and column. Again, the beam-column connections are armoured with parallel-to-grain timber, which is half the width of the column. Further information is given in Figure G.1.5 and Table G.1.16.
G.1.4. Column-base connection design

As discussed in Chapter 10, the column-base connections are designed as PT connections, without additional reinforcement. The internal column-base connections are considered for design. The design process is similar to that for the frames with horizontal post-tensioning but additional gravity axial loads are taken into account. Under the earthquake combination (from NZS1170.5:2004) the gravity induced axial forces on the interior columns are 416kN, 832kN and 1387kN for Frame 1, 2 and 3 respectively. The exterior columns have half the axial force of the interior columns.

The exterior columns are assumed to provide half the moment of the interior columns, under gravity axial loads. As discussed in Chapter 10, the effects tension and compression axial forces induced by seismic loading are ignored, as it has little effect on the average moment provided by the exterior columns.

Frame 1

Step 1: Defined geometry

The column is a solid section of 300mm wide by 600mm deep.

Step 2: Evaluate member deformation

The deformation of the column can be estimated by considering the assumed contra-flexure point at 60% of the first interstorey height.

\[
\theta_c = \frac{\phi_{c,b}}{0.6H_1} \left( \frac{(0.6H_1)^2}{3} + \frac{E_i h_c^2}{G} \right)
\]

The curvature at the column-base is:

\[
\phi_{c,b} = \frac{M_{int,b}}{E_i I_c} = \frac{0.6V_h H_1}{n_b} \frac{1}{E_i I_c} = 4.98 \times 10^{-6} \frac{1}{mm}
\]
Therefore, the total column deformation is:

\[
\theta_c = \frac{4.98 \times 10^{-6}}{226} \left( \frac{2286^2}{3} + \frac{11}{0.6} \frac{600^2}{8} \right) = 0.0038 + 0.0018 = 0.0056
\]

**Step 3:** Determine the allowable connection rotation

The difference between the allowable drift per floor and member deformation, gives the allowable connection rotation, which is used for connection design.

\[
\theta_{imp} \leq \theta_D - \theta_c = 0.0144
\]

**Step 4:** Evaluate connection moment capacity

The connection moment is evaluated, using modeling procedures from Chapter 7, repeated here as Figure G.1.6. The post-tensioning tendons are positioned 50mm outside the column section, on both sides (see Chapter 10). The connections were designed to remain essentially elastic until the design drift of 2%. A bi-linear stress-strain law is applied for the timber in compression, with a yield stress of 45MPa.

The moment-rotation curve for the interior column-base connections is shown in Figure G.1.7. The curve satisfies the design connection moment and rotation from above. The same process is repeated for the exterior column-base connection. The required area and force for each tendon is given in Table G.1.17.
Appendix G – Designs for time-history analysis verification

Figure G.1.6. Connection response procedure for walls

Figure G.1.7. Column-base connection response for frame 1

Table G.1.17 – Post-tensioning area and force for Frame 1 column-base connections

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Apt (mm²)</th>
<th>Tpt (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>495</td>
<td>401</td>
</tr>
<tr>
<td>Exterior</td>
<td>297</td>
<td>185</td>
</tr>
</tbody>
</table>
Step 5: Detailed design

The flexural and shear capacity of the column is checked for ultimate limit state loading. Because this is not a critical step for defining the frame models for time-history analysis, the detailed design is not shown here. In general, the deflection criteria are more critical than the ultimate limit state strength demands.

Frame 2

For column-based connections of frame 2, the same process, shown above for Frame 1, is repeated. The imposed rotation in the connection is 0.0145 and 0.0173 for the interior and exterior columns respectively. The moment-rotation curve is shown in Figure G.1.8. The post-tensioning area and force is given in Table G.1.18.

![Figure G.1.8. Column-base connection response for frame 2](image)

Table G.1.18 – Post-tensioning area and force for Frame 2 column-base connections

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Apt (mm²)</th>
<th>Tpt (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>396</td>
<td>309</td>
</tr>
<tr>
<td>Exterior</td>
<td>198</td>
<td>136</td>
</tr>
</tbody>
</table>
Frame 3

For column-based connections of frame 3, the imposed rotation in the connection is 0.0144 and 0.0172 for the interior and exterior column respectively. The moment-rotation curve is shown in Figure G.1.9. The post-tensioning area and force is given in Table G.1.19. Note; the gravity induced axial force for the exterior columns is so large that post-tensioning is not unnecessary. Furthermore, this is only possible if the seismic induced tension force (648kN) is less than the gravity induced axial force (694kN), which is the case here. For actual design, non-stresses reinforcement should be provided to ensure structural robustness for larger earthquake events.

![Figure G.1.9. Column-base connection response for frame 3](image)

Table G.1.19 – Post-tensioning area and force for Frame 3 column-base connections

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Apt (mm²)</th>
<th>Tpt (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>396</td>
<td>309</td>
</tr>
<tr>
<td>Exterior</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
G.1.5. Frame design with pinned column-base connections

As discussed in Chapter 9, a designer may choose to pin the column-base connections to minimizing foundation demands and/or to potentially reduce construction costs. In Chapter 9, it is suggested that an essentially linear displacement profile can be maintained. This is provided that the stiffness of the first floor is increased to compensate for the increased flexibility of the columns between the basement and the first floor. Within this section, a brief description of the design process necessary to ensure a linear displacement profile is presented. Frame 2 from Chapter 10 is considered.

**Determination of frame actions**

Again, an equilibrium-based approach (Priestley et al. 2007) is used to distribute strength throughout the frame. However, because the column-base connections are pinned the moment demand at the top of the columns is defined. The first floor moment demand must also increase to ensure a consistent bending moment diagram up the height of the frame.

**Step 1:** Determine the total overturning moment (OTM)

\[ OTM = \sum_{j=1}^{n} F_j H_j \]

**Step 2:** Decide how much of the OTM will be taken by the column bases:

\[ OTM = \sum_{j=1}^{n} M_{c,j} + T_{L_{base}} \]

The sum of the column base is zero for pinned-base columns. Hence:

\[ \sum_{j=1}^{n} M_{c,j} = 0 \]
Step 3: Determine the tension and compression force in the exterior columns:

\[ T = C = \frac{OTM - \sum_{j=1}^{n} M_{r,j}}{L_{\text{base}}} \]

Step 4: Determine the design moment first floor beams

A rational approach is to make the first floor beam moment and the column moments equal. Hence:

\[ M_{b1} = \frac{V_{b1}H_{1}}{n_b} \]

This gives a beam shear of:

\[ V_{b1} = \frac{2M_{b1}}{L_b} \]

Step 5: Proportion the seismic axial forces to each beam up the height of the building

The beam shear for the first floor has already been defined. A rational way to proportion the remaining seismic axial forces into beam shears, is to use the shear force diagram (Priestley et al. 2007). This will ensure that the assumed displacement profile is maintained. Hence:

\[ V_{B,i} = (T - V_{b1}) \frac{V_{S,j}}{\sum_{i=2}^{n} V_{S,i}} \]

Step 6: Calculate the remaining beam design moments

\[ M_{B,j} = V_{B,j} \frac{L_b}{2} \text{ (At the column centerlines)} \]

The beam-column connection design moments at the column face are:

\[ M_{\text{con},j} = M_{B,j} \frac{L_b - h_c}{L_b} \]
Step 7: Calculate column design moments

For all storeys, other than the roof:

\[ \sum_{j=1}^{n} M_{C,j} = n_B M_{B,j} \]

For the roof:

\[ \sum_{j=1}^{n} M_{C,j} = 2n_B M_{B,j} \]

For Frame 2, a summary of the frame actions are given in Table G.1.20. Note that for the lateral force design of this frame a linear displacement profile is assumed.

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>VBi (kN)</th>
<th>MBi (kN.m)</th>
<th>Mcon (kN.m)</th>
<th>( \sum M_c ) (kN.m)</th>
<th>Mc,ext (kN.m)</th>
<th>Mc,int (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>14.2</td>
<td>49.5</td>
<td>45.3</td>
<td>495</td>
<td>49.5</td>
<td>99.1</td>
</tr>
<tr>
<td>5</td>
<td>27.1</td>
<td>94.9</td>
<td>86.8</td>
<td>475</td>
<td>47.5</td>
<td>94.9</td>
</tr>
<tr>
<td>4</td>
<td>37.5</td>
<td>131</td>
<td>120</td>
<td>656</td>
<td>65.6</td>
<td>131</td>
</tr>
<tr>
<td>3</td>
<td>45.3</td>
<td>159</td>
<td>145</td>
<td>793</td>
<td>79.3</td>
<td>159</td>
</tr>
<tr>
<td>2</td>
<td>50.5</td>
<td>177</td>
<td>162</td>
<td>883</td>
<td>88.3</td>
<td>177</td>
</tr>
<tr>
<td>1</td>
<td>153</td>
<td>534</td>
<td>465</td>
<td>2670</td>
<td>267</td>
<td>534</td>
</tr>
<tr>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Due to the pinned column-base connections, the moment demand on the first floor beams and columns is significantly higher than the other levels. Hence, it is likely with such a design that the section size of the beams and columns at the first floor will need to be increased.
Frame and post-tensioning design

A similar process is followed for both pinned and moment resisting column-based connections. However, for the connection design at the first floor, the elastic deformation of the column over the entire first floor interstorey height and half the second floor interstorey height is considered.

**Step 1:** Estimate frame geometry

**Step 2:** Evaluate member deformation

The deformation of the beam, column and joint panel at every level other than the first level can be determined using equations from Chapter 6 (see above). For the first floor, the elastic deformation of the beam and column is:

\[
\theta_b = \frac{\phi_b}{L_b} \left( \frac{(L_b - h_c)^2}{6} + \frac{E_r h_b^2}{G} \right) \quad \text{(as before)}
\]

\[
\theta_c = \frac{\phi_c}{H} \left( \frac{2H - h_b)^2}{12} + \frac{(H - h_b)^2}{G} \right)
\]

Where:

\[\phi_b = \frac{M_{con}}{E_r I_b} \quad \text{and} \quad \phi_c \approx \frac{M_{con}}{E_r I_c} \frac{L_b (1.5H - h_b)}{1.5H (L_b - h_c)}\]

For joint panel deformation:

\[
\theta_j \approx \frac{V_{jh}}{G A_{sh}} \left( 1 - \frac{h_c}{L_b} - \frac{h_b}{1.5H} \right)
\]

Therefore, the total member deformation is:

\[
\theta_{\text{mem}} = \theta_c + \theta_b + \theta_j
\]

**Step 3-5:** See above. The remainder of the design procedure is similar to frames with moment-resisting column-base connections.
The above design procedure results in a solid section of 300mm wide by 900mm deep for the beams and columns on the first floor. For the remainder of the frame, the beams and columns are 300mm wide by 600mm deep. Further information is given in Figure G.1.4 and Table G.1.15.

![Connection response for frame 2 with pinned column-bases](image)

**Figure G.1.10.** Connection response for frame 2 with pinned column-bases

**Table G.1.21 – Design parameters for Frame 2 with pinned column-bases**

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>θ_{imp} (rad)</th>
<th>A_{pt} (mm²)</th>
<th>T_{pt} (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>0.0185</td>
<td>198</td>
<td>117</td>
</tr>
<tr>
<td>5</td>
<td>0.0168</td>
<td>297</td>
<td>273</td>
</tr>
<tr>
<td>4</td>
<td>0.0149</td>
<td>495</td>
<td>402</td>
</tr>
<tr>
<td>3</td>
<td>0.0134</td>
<td>594</td>
<td>528</td>
</tr>
<tr>
<td>2</td>
<td>0.0124</td>
<td>693</td>
<td>616</td>
</tr>
<tr>
<td>1</td>
<td>0.0117</td>
<td>1386</td>
<td>1254</td>
</tr>
</tbody>
</table>
G.2. WALLS

As described in Chapter 11, six wall geometries are considered, termed wall 1, 2, 3, 1c, 2c and 3c, where the ‘c’ stands for coupled. The walls are designed using lateral force design procedures from Chapter 9 and wall design procedures from Chapter 7. The lateral force design, wall design and connection design for each of these walls is documented below.

G.2.1. Lateral force design

The lateral force design procedure is similar to frames (see section G.1). Therefore, only the key results for the lateral force design of each wall are given here. For all designs a linear displacement profile is assumed. The design actions for the coupled walls are simply double that of the cantilever walls. A hysteretic damping of 8% is specified for the hybrid coupled wall systems.

Wall 1

The resultant base shear and distributed lateral forces from the displacement-based design for Wall 1 are given Table G.2.1. The wall has effective period of 1.10s.

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Floor Force, Fi (KN)</th>
<th>Shear, Vsi (kN)</th>
<th>Moment, Msi (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>115</td>
<td>115</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>84</td>
<td>199</td>
<td>438</td>
</tr>
<tr>
<td>1</td>
<td>42</td>
<td>241</td>
<td>1196</td>
</tr>
<tr>
<td>0</td>
<td>(Vb =) 241</td>
<td>(OTM =) 2115</td>
<td></td>
</tr>
</tbody>
</table>

Table G.2.1 – Design lateral forces for Wall 1
Wall 2

The resultant base shear and distributed lateral forces from the displacement-based design for Wall 2 are given Table G.2.2. The wall has effective period of 1.88s.

Table G.2.2 – Design lateral forces for Wall 2

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Floor Force, Fi (KN)</th>
<th>Shear, Vsi (kN)</th>
<th>Moment, Msi (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>78</td>
<td>78</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>71</td>
<td>149</td>
<td>297</td>
</tr>
<tr>
<td>4</td>
<td>57</td>
<td>206</td>
<td>865</td>
</tr>
<tr>
<td>3</td>
<td>43</td>
<td>249</td>
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<td>14</td>
<td>292</td>
<td>3658</td>
</tr>
<tr>
<td>0</td>
<td>(Vb =) 292</td>
<td>(OTM=) 4770</td>
<td></td>
</tr>
</tbody>
</table>

Wall 3

The resultant base shear and distributed lateral forces from the displacement-based design for Wall 3 are given Table G.2.3. The wall has effective period of 3.05s. A linear projection of the displacement spectrum beyond the corner period is assumed, as discussed in Chapter 9.

Table G.2.3 – Design lateral forces for Wall 3

<table>
<thead>
<tr>
<th>Storey, i</th>
<th>Floor Force, Fi (KN)</th>
<th>Shear, Vsi (kN)</th>
<th>Moment, Msi (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>49</td>
<td>49</td>
<td>0</td>
</tr>
<tr>
<td>9</td>
<td>49</td>
<td>98</td>
<td>188</td>
</tr>
<tr>
<td>8</td>
<td>43</td>
<td>142</td>
<td>563</td>
</tr>
<tr>
<td>7</td>
<td>38</td>
<td>180</td>
<td>1103</td>
</tr>
<tr>
<td>6</td>
<td>33</td>
<td>212</td>
<td>1789</td>
</tr>
<tr>
<td>5</td>
<td>27</td>
<td>240</td>
<td>2598</td>
</tr>
<tr>
<td>4</td>
<td>22</td>
<td>261</td>
<td>3511</td>
</tr>
<tr>
<td>3</td>
<td>16</td>
<td>278</td>
<td>4507</td>
</tr>
<tr>
<td>2</td>
<td>11</td>
<td>289</td>
<td>5565</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>294</td>
<td>6664</td>
</tr>
<tr>
<td>0</td>
<td>(Vb =) 294</td>
<td>(OTM=) 7785</td>
<td></td>
</tr>
</tbody>
</table>
Appendix G – Designs for time-history analysis verification

**Wall 1c**

The resultant base shear and distributed lateral forces from the displacement-based design for Wall 1c are given Table G.2.4. The moment applied due to UFP couplers and the wall elements is determined by assuming a $\beta_{\text{CE}}$-value of 0.4. For the moment applied to the wall elements, the absolute maximum moment at a given floor is considered. The wall has effective period of 2.20s.

<table>
<thead>
<tr>
<th>Storey $i$</th>
<th>Floor Force $F_i$ (kN)</th>
<th>Shear $V_{si}$ (kN)</th>
<th>Moment $M_{si}$ (kN.m)</th>
<th>Moment $M_{ufp}$ (kN.m)</th>
<th>Moment $M_w$ (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>57</td>
<td>57</td>
<td>0</td>
<td>104</td>
<td>104</td>
</tr>
<tr>
<td>2</td>
<td>42</td>
<td>99</td>
<td>216</td>
<td>207</td>
<td>320</td>
</tr>
<tr>
<td>1</td>
<td>21</td>
<td>120</td>
<td>593</td>
<td>311</td>
<td>697</td>
</tr>
<tr>
<td>0</td>
<td>(Vb =) 120</td>
<td>(OTM=) 1048</td>
<td>311</td>
<td>737</td>
<td></td>
</tr>
</tbody>
</table>

**Wall 2c**

The resultant base shear and distributed lateral forces from the displacement-based design for Wall 2c are given Table G.2.5. The wall has effective period of 4.10s. Again, a linear projection of the displacement spectrum beyond the corner period is assumed, as discussed in Chapter 9.

<table>
<thead>
<tr>
<th>Storey $i$</th>
<th>Floor Force $F_i$ (kN)</th>
<th>Shear $V_{si}$ (kN)</th>
<th>Moment $M_{si}$ (kN.m)</th>
<th>Moment $M_{ufp}$ (kN.m)</th>
<th>Moment $M_w$ (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>33</td>
<td>33</td>
<td>0</td>
<td>69</td>
<td>69</td>
</tr>
<tr>
<td>5</td>
<td>30</td>
<td>63</td>
<td>125</td>
<td>137</td>
<td>57</td>
</tr>
<tr>
<td>4</td>
<td>24</td>
<td>87</td>
<td>364</td>
<td>206</td>
<td>227</td>
</tr>
<tr>
<td>3</td>
<td>18</td>
<td>105</td>
<td>695</td>
<td>275</td>
<td>489</td>
</tr>
<tr>
<td>2</td>
<td>12</td>
<td>117</td>
<td>1095</td>
<td>343</td>
<td>821</td>
</tr>
<tr>
<td>1</td>
<td>6</td>
<td>123</td>
<td>1540</td>
<td>412</td>
<td>1197</td>
</tr>
<tr>
<td>0</td>
<td>(Vb =) 123</td>
<td>(OTM=) 2008</td>
<td>412</td>
<td>1596</td>
<td></td>
</tr>
</tbody>
</table>
Wall 3c

The resultant base shear and distributed lateral forces from the displacement-based design for Wall 3c are given Table G.2.6. The wall has effective period of 6.65s, which is well above the corner period. Again, a linear projection of the displacement spectrum is assumed. Note; it is likely that serviceability limit state wind would govern the design (see Chapter 9) of this wall system.

Table G.2.6 – Design lateral forces for Wall 3c

<table>
<thead>
<tr>
<th>Storey</th>
<th>Floor Force Fi (kN)</th>
<th>Shear Vsi (kN)</th>
<th>Moment Msi (kN.m)</th>
<th>Moment Mufp (kN.m)</th>
<th>Moment Mw (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>21</td>
<td>21</td>
<td>0</td>
<td>56</td>
<td>56</td>
</tr>
<tr>
<td>9</td>
<td>21</td>
<td>41</td>
<td>79</td>
<td>112</td>
<td>33</td>
</tr>
<tr>
<td>8</td>
<td>18</td>
<td>60</td>
<td>237</td>
<td>167</td>
<td>126</td>
</tr>
<tr>
<td>7</td>
<td>16</td>
<td>76</td>
<td>465</td>
<td>223</td>
<td>298</td>
</tr>
<tr>
<td>6</td>
<td>14</td>
<td>89</td>
<td>753</td>
<td>279</td>
<td>530</td>
</tr>
<tr>
<td>5</td>
<td>11</td>
<td>101</td>
<td>1094</td>
<td>335</td>
<td>815</td>
</tr>
<tr>
<td>4</td>
<td>9</td>
<td>110</td>
<td>1478</td>
<td>391</td>
<td>1143</td>
</tr>
<tr>
<td>3</td>
<td>7</td>
<td>117</td>
<td>1898</td>
<td>446</td>
<td>1508</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>122</td>
<td>2343</td>
<td>502</td>
<td>1897</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>124</td>
<td>2806</td>
<td>558</td>
<td>2304</td>
</tr>
<tr>
<td>0</td>
<td>(Vb =) 124</td>
<td>(OTM=) 3278</td>
<td>558</td>
<td>2720</td>
<td></td>
</tr>
</tbody>
</table>
G.2.2. Wall and post-tensioning design

The design of the Wall 1 and 1c is described in full. Key results are provided for Wall 2, 2c, 3 and 3c.

**Wall 1**

**Step 1: Estimate wall geometry**

As mentioned in Chapter 11, it is assumed that the wall geometry is constant up the entire height of the building. A solid section of 2400mm wide by 180mm thick is assumed.

**Step 2: Evaluate wall deformation**

The deformation of the wall is a combination of the flexural deformation and shear deformation and is approximated as:

\[
\theta_w = \theta_f + \theta_s = \frac{V_b H_e^2}{3EI_w} + \frac{0.8V_b}{GA_{sw}} = 0.0027 + 0.0011 = 0.0038
\]

**Step 3: Determine the allowable connection rotation**

The difference between the allowable drift and the wall deformation, gives the allowable connection rotation, which is used for connection design.

\[
\theta_{imp} \leq \theta_p - \theta_w = 0.020 - 0.0038 = 0.0162
\]

**Step 4: Evaluate connection moment capacity**

The connection moment is evaluated, using modeling procedures from Chapter 7, repeated here as Figure G.2.1. The post-tensioning is assumed to be positioned at the centerline of the wall section. The connections are assumed to remain elastic. The connection moment is evaluated for the combined axial load from the post-tensioning and gravity load. The gravity induced axial load is 480kN.
Seismic design of post-tensioned timber frame and wall buildings. M. P. Newcombe

The moment-rotation curve for the wall-base connection is shown in Figure G.2.2. The curve satisfies the design connection moment of 2115kN.m within the imposed rotation of 0.0162. The post-tensioning area and force from each analysis is 1089mm$^2$ and 1164kN respectively.

**Step 5: Detailed design**

The flexural and shear capacity of the wall is checked for ultimate limit state loading. Because this is not a critical step for defining the wall models for time-history analysis, the detailed design is not shown here. In general, the deflection criteria are more critical than the ultimate limit state strength demands on the wall elements.
Wall 2
For wall 2, a similar process, shown above for Wall 1, is repeated. A solid section of 3000mm wide by 180mm thick is required. The moment-rotation curve for the wall-base connection is given in Figure G.2.2. The imposed connection rotation at the base of the wall is 0.0131rad. The post-tensioning area and force from each analysis is 2079mm\(^2\) and 2222kN respectively. The gravity induced axial load on the wall is 975kN.

Wall 3
For wall 3, a solid section of 4000mm wide by 180mm thick is required. The moment-rotation curve for the wall-base connection is given in Figure G.2.2. The imposed connection rotation at the base of the wall is 0.0127rad. The post-tensioning area and force from each analysis is 2079mm\(^2\) and 2342kN respectively. The gravity induced axial load on the wall is 1635kN.

Wall 1c
Wall 1c is the first of the coupled walls. The deflection is determined based on the average moment and shear demand per wall. The design moment for the wall-base connections is 314kN.m. This moment demand is satisfied, ignoring axial forces induced by the coupling elements, as discussed in Chapter 7. A solid section of 1200mm wide by 180mm thick is required. The moment-rotation curve for the wall-base connection is given in Figure G.2.2. The imposed connection rotation at the base of the wall is 0.0162rad. The post-tensioning area and force from each analysis is 297mm\(^2\) and 301kN respectively. The gravity induced axial load is 230kN.

Wall 2c
For wall 2c, the design moment for the wall-base connections is 602kN.m. A solid section of 1800mm wide by 180mm thick is required. The moment-rotation curve for the wall-base connection is given in Figure G.2.2. The imposed connection rotation at the base of the wall is 0.0162rad. The post-tensioning area and force from each analysis is 198mm\(^2\) and 207kN respectively. The gravity induced axial load is 488kN.
**Wall 3c**

For wall 3c, the design moment for the wall-base connections is 983kN.m. A solid section of 2200mm wide by 180mm thick is required. The moment-rotation curve for the wall-base connection is given in Figure G.2.2. The imposed connection rotation at the base of the wall is 0.0147 rad. The post-tensioning area and force from each analysis is 198mm$^2$ and 124kN respectively. The gravity induced axial load is 818kN. Hence, the axial load is providing most of the base-moment.

![Figure G.2.2. Connection response for wall 1, 2, 3, 1c, 2c and 3c](image-url)
G.2.3. Calibration of the multi-axial-springs

To model the wall-base and splice connections in RUAUMOKO, the axial stiffness of the multi-axial-springs must be calculated (see Chapter 11) to match the analytical predictions (described in Chapter 7). The axial stiffness of the multi-axial-spring (MS) is determined by interpreting the neutral axis depth from the analytical prediction and calculating the effective length, $L_e$, as shown in Chapter 7. The calculated axial stiffness for the MS model, $K_A$, for each wall is shown in Table G.2.7.

<table>
<thead>
<tr>
<th>Wall</th>
<th>$c/L_w$</th>
<th>$L_e$ (mm)</th>
<th>$K_A$ (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.1072</td>
<td>541</td>
<td>8.41E6</td>
</tr>
<tr>
<td>2</td>
<td>0.1205</td>
<td>474</td>
<td>11.95E6</td>
</tr>
<tr>
<td>3</td>
<td>0.1074</td>
<td>540</td>
<td>13.99E6</td>
</tr>
<tr>
<td>1c</td>
<td>0.1139</td>
<td>506</td>
<td>4.48E6</td>
</tr>
<tr>
<td>2c</td>
<td>0.0944</td>
<td>624</td>
<td>5.45E6</td>
</tr>
<tr>
<td>3c</td>
<td>0.0933</td>
<td>632</td>
<td>6.58E6</td>
</tr>
</tbody>
</table>

G.2.4. UFP Coupler design

For Wall 1c, 2c and 3c the UFP couplers must be designed. There are several approaches that can be followed, as discussed in Chapter 9. Here the UFP couplers are designed for a set $\beta_{CE}$ – value (0.4), which was defined in Chapter 9 as the ratio of the over-turning moment provided by the coupling elements and the total overturning moment.
To determine the required strength of the UFP couplers the following equation is applied. It is assumed, based on analytical modeling (see Chapter 7), that the maximum variation of the center of compression, $c_c$, in each wall is 2% of the wall length, $l_w$.

$$V_{ufp} = \frac{\beta_{CE} OTM}{l_c}$$

Where:

$$l_c = l_{cl} - \left( c_{c,1} - c_{c,2} \right) \approx l_{cl} - 0.02l_w$$
is the distance between center of compression of each wall;

$l_{cl}$ is the distance between the centerline of each wall;

$c_{c,1}$ & $c_{c,2}$ is the distance from the extreme compression fiber to the center of compression of each wall, $c_{c,3}$ is the wall with maximum compression.

The shear provided from the couplers is then divided evenly to each floor (giving $V_{ufp,i}$). As discussed in Chapter 11, an elastic-perfectly-plastic hysteresis rule is used to model the UFP couplers. The axial stiffness of the coupling elements, $K_{ufp}$, can be determined from the design shear force, $V_{ufp,i}$, and the assumed slip. It is assumed that there is 1mm slip per UFP anchorage. Hence, the yield point is at 2mm of axial displacement. A summary of the above calculations is given in Table G.2.8.

<table>
<thead>
<tr>
<th>Wall</th>
<th>OTM (kN.m)</th>
<th>lcl (m)</th>
<th>Vufp (kN)</th>
<th>Vufp,i (kN)</th>
<th>Kufp (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1c</td>
<td>1048</td>
<td>1.35</td>
<td>316</td>
<td>105</td>
<td>52690</td>
</tr>
<tr>
<td>2c</td>
<td>2008</td>
<td>1.95</td>
<td>420</td>
<td>69.9</td>
<td>34970</td>
</tr>
<tr>
<td>3c</td>
<td>3278</td>
<td>2.35</td>
<td>569</td>
<td>56.9</td>
<td>28430</td>
</tr>
</tbody>
</table>
G.2.5. Verification of the modelled hysteretic damping

In Chapter 11, a cyclic adaptive pushover analysis is used to ensure that the assumed hysteretic damping from the displacement-based design (15%) is achieved by the UFP couplers. The hysteretic damping values achieved from the cyclic pushover analysis are shown in Table G.2.9.

<table>
<thead>
<tr>
<th>Wall</th>
<th>$\xi_{\text{hyst}}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1c</td>
<td>19.4</td>
</tr>
<tr>
<td>2c</td>
<td>21.2</td>
</tr>
<tr>
<td>3c</td>
<td>21.1</td>
</tr>
</tbody>
</table>

All hysteretic damping values are in excess of the values assumed by the displacement-based design by up to 40%. However, according to displacement based design process (discussed in Chapter 9), using a hysteretic damping of 21.2% rather than 15% will result in a base shear that is 13% less. Notably, the hysteretic damping correction factor in the ductility ranges of the coupled wall systems (5 to 7) from Priestley et al (2007) are approximately unity for the flag-shaped hysteresis.

An alternative approach to verify the hysteretic damping is to use the analytical modelling approaches (from Chapter 7). To do this the yield deformation of the wall system must be estimated. As discussed in Chapter 9, the yield deformation can be estimated using empirical expressions, which are repeated below. These empirical relationships are compared with the results from the pushover analysis to ensure they are sufficiently accurate.

$$\Delta_{y,e} = \Delta_{ufp,e} + \Delta_{w,e}$$

Where:

- $\Delta_{y,e}$ = the total yield deformation at the effective height;
- $\Delta_{ufp,e}$ = the yield deformation due to couplers at the effective height;
- $\Delta_{w,e}$ = the yield deformation due to the wall at the effective height;
The yield deformation due to the coupler is given below. It is assumed that the slip of the UFP couplers relative to the walls is 2mm.

$$\Delta_{ufp,e} = 2H \frac{\Delta_{ufp}}{e}$$

Where:  
- $H_e$ is the effective height of the wall;  
- $\Delta_{ufp}$ is the slip of the UFP coupler relative to the walls.

The wall deformation is approximated using the following expression:

$$\Delta_{w,e} = \frac{0.43V_b H_e}{\gamma_{LS}GA_{sw}} + 2H_e V_{ufp} \left( \frac{1}{G A_{sw,v}} + \frac{1}{l_w EA_w} \right)$$

Where:  
- $\gamma_{LS}$ is 1.0 and 1.25 for serviceability and ultimate limit state design respectively;  
- $V_b$ is the base shear of a double wall system;  
- $A_{sw}$ is the horizontal shear area of one wall element;  
- $V_{ufp} \approx \frac{\beta_{CE}^{OTM}}{l_{cl}}$ is the total shear from the UFP couplers;  
- $l_{cl}$ is the centerline distance between each wall;  
- $A_{sw,v}$ is the vertical shear area of one wall element;  
- $G$ is the shear modulus of the timber;  
- $H_n$ is total height of the wall;  
- $A_w$ is the cross sectional area of one wall element.

The yield displacement predicted by the above equations is compared with the results from the pushover analyses in Table G.2.10. However, because the finite element model uses Giberson frame elements, vertical shear distortion is not possible. Therefore, the vertical shear distortion, in the above expression, is ignored.

The estimated yield rotation is larger than the results from the pushover analysis by 40% on average. This is desirable from a design point of view, as it results in
conservative estimates for the hysteretic damping of the system. Furthermore, it is likely that the numerical model does not include all of the significant deformation contributions.

<table>
<thead>
<tr>
<th>Wall</th>
<th>$\Delta u_{f,p,e}$ (m)</th>
<th>$\Delta w_{e}$ (m)</th>
<th>$\Delta y_{e}$ (m)</th>
<th>$\theta y_{e}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1c</td>
<td>0.0146</td>
<td>0.0260</td>
<td>0.0406</td>
<td>0.46</td>
</tr>
<tr>
<td>2c</td>
<td>0.0182</td>
<td>0.0532</td>
<td>0.0714</td>
<td>0.44</td>
</tr>
<tr>
<td>3c</td>
<td>0.0241</td>
<td>0.1245</td>
<td>0.1486</td>
<td>0.56</td>
</tr>
</tbody>
</table>

Table G.2.10 – Yield deformation from analytical model and pushover analysis
APPENDIX REFERENCES

ACI. (2001). "Acceptance Criteria for Moment Frames Based on Structural Testing and Commentary." American Concrete Institute Innovation Task Group 1 and Collaborators, American Concrete Institute, Farmington Hills, MI.


Appendix references


