HYSTERETIC MODELLING OF
MOMENT-RESISTING NAILED TIMBER JOINTS

B.T. Kivell\textsuperscript{1}, P.J. Moss\textsuperscript{2} and A.J. Carr\textsuperscript{2}

Summary:

While nailed timber joints have been used for many years, attention is now being focused on producing nailed joints that have a significant moment resistance. Such joints can be made using steel sideplates and the increased moment resistance over more traditional nailing could be utilised to resist seismic loads. Like steel and concrete, the nailed joint is nonlinear, however at low enough slip values, the experimental load-slip curve for a laterally loaded nailed joint can be approximated by a linear relationship. While the load-slip behaviour in the linear range has been well studied, the torsion formula introduced, at most, no more error variation than is expected of wood itself. In looking at the serviceability requirements of a joint, a slip value of 0.4 mm has been accepted as a basis for determining the allowable load, and this also allows an elastic stiffness of the joint to be determined. Research on load-slip behaviour in the linear range has been carried out by Kuenzi\textsuperscript{(2)}, Wilkinson\textsuperscript{(1)} and Noren\textsuperscript{(3)}. Mack\textsuperscript{(4)} has presented a method for predicting the load-slip behaviour for a nailed joint during first loading up to a slip of 2.5 mm. Foschi\textsuperscript{(5)} looked at the estimation of ultimate loads for nailed joints. In New Zealand, research has included that by Lake\textsuperscript{(6)} who also reviewed the literature on the testing of nailed joints up to 1972.

Although there is a lot of information on timber joint behaviour where the nails are subjected to a shear loading, there is relatively little information on the ability of a group of nails to withstand an applied moment loading. The torsion formula, which has been used to determine the moment capacity of a group of bolts or rivets, was investigated by Perkins et al\textsuperscript{(7)} with respect to nailed joints and their results "indicated that the torsion formula introduces, at most, no more error variation than is expected of wood itself". It was found that, though the experimental load on the extreme nail was generally less than that calculated by the torsion formula.

Recent nailed joint research in New Zealand has been concerned with developing a suitable rigid moment-resisting joint incorporating nailed steel side plates. The nailing pattern is rectangular with the steel side-plates being predrilled. When the side plate is subjected to rotation the nails are loaded in shear which results in a torsional moment being developed on the nail group as a whole. Tests carried out to date on particular nail arrangements have all shown satisfactory performance of the nailed joint in resisting moments.

INTRODUCTION:

Timber is a very old structural material and has been widely used down through the ages in the construction of houses. Today, it is still used extensively to build low rise buildings, mainly one and two-storey houses, and it is especially suitable on account of its resilience under load for structures situated in earthquake prone areas of the world. The largest timber building in New Zealand is the 4 storey old Government Building, Wellington, built in 1875 and still standing and in use today having survived the high seismic activity of the Wellington region for over 100 years. With recent changes in the New Zealand fire code, timber building in New Zealand is now being focussed on producing nailed joints that have a significant moment resistance. Such joints can be made using steel sideplates and the increased moment resistance over more traditional nailing could be utilised to resist seismic loads. Like steel and concrete, the nailed joint is nonlinear, however at low enough slip values, the experimental load-slip curve for a laterally loaded nailed joint can be approximated by a linear relationship. While the load-slip behaviour in the linear range has been well studied, the torsion formula introduced, at most, no more error variation than is expected of wood itself. In looking at the serviceability requirements of a joint, a slip value of 0.4 mm has been accepted as a basis for determining the allowable load, and this also allows an elastic stiffness of the joint to be determined. Research on load-slip behaviour in the linear range has been carried out by Kuenzi\textsuperscript{(2)}, Wilkinson\textsuperscript{(1)} and Noren\textsuperscript{(3)}. Mack\textsuperscript{(4)} has presented a method for predicting the load-slip behaviour for a nailed joint during first loading up to a slip of 2.5 mm. Foschi\textsuperscript{(5)} looked at the estimation of ultimate loads for nailed joints. In New Zealand, research has included that by Lake\textsuperscript{(6)} who also reviewed the literature on the testing of nailed joints up to 1972.

Although there is a lot of information on timber joint behaviour where the nails are subjected to a shear loading, there is relatively little information on the ability of a group of nails to withstand an applied moment loading. The torsion formula, which has been used to determine the moment capacity of a group of bolts or rivets, was investigated by Perkins et al\textsuperscript{(7)} with respect to nailed joints and their results "indicated that the torsion formula introduces, at most, no more error variation than is expected of wood itself". It was found that, though the experimental load on the extreme nail was generally less than that calculated by the torsion formula.

Recent nailed joint research in New Zealand has been concerned with developing a suitable rigid moment-resisting joint incorporating nailed steel side plates. The nailing pattern is rectangular with the steel side-plates being predrilled. When the side plate is subjected to rotation the nails are loaded in shear which results in a torsional moment being developed on the nail group as a whole. Tests carried out to date on particular nail arrangements have all shown satisfactory performance of the nailed joint in resisting moments.

2. Department of Civil Engineering, University of Canterbury, Christchurch.

BULLETIN OF THE NEW ZEALAND NATIONAL SOCIETY FOR EARTHQUAKE ENGINEERING, VOL. 14, NO. 4, DECEMBER 1981
More recently, Thurston and Flack have carried out some cyclic testing of full scale joints incorporating nailed steel side plates to determine the joint performance and possible load-slip or moment-rotation curves beyond the initial elastic region, i.e. beyond the initial 0.4 mm nail deformation. Pinched hysteresis loops were obtained but, even so, large energy absorption was still evident. They also repaired a joint with epoxy resin and on retesting, found it to be stiffer than the original - an encouraging result for primary load resisting timber structures that could possibly be subjected to large earthquake excitation and require subsequent repair. Further tests were carried out with a load limiting device in the way of necked metal side plates which limited the amount of nail deformation as the plate yielded plastically at a predetermined load.

**DEVELOPMENT OF HYSTERETIC MODELS**

Structures subjected to strong ground motion will sustain inelastic deformations at certain critical regions. These regions, in most dynamic inelastic analyses, are restricted to the ends of members and are usually considered to be very small in length (unless a finite length plastic hinge is assumed). Various hysteresis models have been developed to describe the behaviour of these critical regions for different materials and for different aspects of the joint construction. As researchers looked closer into the governing factors of joint behaviour, then so did the hysteresis models become more complicated. However, no one hysteresis model can apply to all materials and hence one must determine exactly what the governing factors are for a given material and joint construction and choose a hysteresis model accordingly, remembering that the model is only an idealisation of the actual behaviour.

When hysteretic models were first being developed they were required to be simple and efficient in their computational effort and so the elastic-perfectly plastic model was initially used by many investigators to give the overall behaviour of the structure to an earthquake loading but the results had to be treated with caution as it was an oversimplification of the problem.

The strain-hardening characteristics of steel were recognised by modifying the elastic-perfectly plastic model to give a post-yield stiffness greater than zero to give a bilinear model. This model has been used widely for both steel and reinforced concrete structures. The area contained within the hysteresis loops is a good measure of the energy dissipation of the member or structure, and to this extent, the bilinear model with its rather 'soft' hysteresis loops which may be an acceptable model for steel, tends to overestimate the energy dissipation capacity of reinforced concrete which shows a degradation in stiffness as a result of cyclic loading. Therefore, further investigation was necessary to look at the possibility of incorporating stiffness degradation characteristics into a hysteresis model.

Clough developed a stiffness model to represent the cracking of concrete where the degrading stiffness was associated with the closing up of the cracks. A further degrading stiffness model was developed by Takeda, Sozen and Nielsen which included stiffness changes at flexural cracking and yielding, and strain-hardening characteristics. This trilinear degrading model of Takeda which has a non-zero yielding stiffness appears to be the best basis on which any modification should be made to allow for various aspects of joint deformation. Emori and Schnobrich looked at the pinching effect caused by bond deterioration in reinforced concrete and produced a modified Takeda model which had a reduced stiffness range between the unloading and reloading stiffnesses.

All of these hysteresis models were developed to represent the various aspects of joint behaviour for either steel or concrete structures subjected to earthquake motions. However, the modified Takeda model with its representation of the pinching effect between unloading and reloading to represent bond deterioration and bar slip in nailed timber joints showed the governing factor of joint behaviour for a moment-resisting timber joint incorporating nailed steel side-plates.

**MODELLING A NAILED TIMBER JOINT**

While basing a hysteretic model for timber on the modified Takeda model, the trilinear backbone curve was replaced by bi-linear skeleton curve since this is a sufficient approximation to describe the basic load-deformation curve of a nailed timber joint.

In the model, if the maximum deformation (rotation, curvature or displacement) of the member never exceeds the predefined yielding points, then the member remains elastic. When the member deformation exceeds the yield point, then the stiffness is reduced to a given percentage of the initial elastic stiffness as would be the case in modelling the strain-hardening region of a steel member). When the member stops yielding and begins unloading, it unloads with a stiffness which is controlled by the ratio of the yield deformation, $D_y$, to the maximum deformation, $D_m$, in the direction of loading and the initial elastic stiffness, $K_0$, according to the equation

$$K = K_0 \left( \frac{D_y}{D_m} \right)^\alpha$$

where $\alpha$ is a variable determined from experimental data and which controls the deformation after unloading back to zero load.

The stiffness of the member in the region where slippage of the nails occurs is determined by assuming a cubic function
The modified Takeda model requires that the deformations at which the stiffness changes should be symmetric about the vertical (moment) axis, i.e., for point C (Figure 1) the deformation is already pre-determined and the moment can be derived by use of the cubic equation, equation (3) such that

$$B = (r_o^3, 0) \rightarrow C = (-r_o^3, -2\beta r_o^3) \tag{4}$$

Therefore when a change of direction occurs, the following points can be determined (for a given $\alpha$ value):

$$A = (r_1^3, m_1) \quad C = (-r_o^3, -2\beta r_o^3)$$
$$B = (r_o^3, 0) \quad D = (-r_1^3, -m_1) \tag{5}$$

To simplify the required change in incremental stiffness, straight lines $AB$, $BC$ and $CD$ were assumed to give an adequate representation of behaviour and the three lines hereafter referred to as the unloading, the slipping and the reloading stiffnesses respectively. A similar curve and set of straight lines was assumed for points $A'$, $B'$, $C'$ and $D'$ where the loading increment would be of opposite sign.

Several rules had to be formulated to enable the model to cope with changes in the direction of loading, i.e., a load reversal. These rules can be summarised as shown in Figure 2 and as follows:

(a) If the new loading direction (either a positive or negative deformation increment) was of the same sign as the current moment and deformation, then the model would aim for the maximum moment and deformation point that was of the same sign as the current moment and deformation values, as shown in Figure 2(a).

(b) If the new deformation increment was of a different sign to the current moment sign, then the member would have to unload and the resulting deformation sign at zero load would be looked at. If this 'zero load' deformation is of the same sign as the new deformation increment, then the model will head (with a stiffness of $k_0$) for the previous maximum moment and deformation point that was of the same sign as the new deformation increment, as shown in Figure 2(b).

(c) Otherwise, the model will head for the same 'maximum' point via a slipping stiffness and the reloading stiffness as in Figure 2(c).

(d) When the member is unloading but the

---

**Table:**

<table>
<thead>
<tr>
<th>Point</th>
<th>Coordinates</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>($r_1^3, m_1$)</td>
</tr>
<tr>
<td>B</td>
<td>($r_o^3, 0$)</td>
</tr>
<tr>
<td>C</td>
<td>($-r_o^3, -2\beta r_o^3$)</td>
</tr>
<tr>
<td>D</td>
<td>($-r_1^3, -m_1$)</td>
</tr>
</tbody>
</table>
sign of the current deformation is of the opposite sign to the new deformation increment (Figure 2(d)), the member is given a modified slipping stiffness and heads for the previous point C before following the reloading path.

**DYNAMIC ANALYSES**

These were carried out on models of two nail jointed timber portal frames that had been tested under cyclic loading in the laboratory(12). The centreline dimensions of the portal frames were a height of 2.000 m with columns 3.420 m apart, and pin-jointed at their bases (see Figure 3). The difference between the two frames lay in

<table>
<thead>
<tr>
<th>Member size</th>
<th>SPECIMEN 1</th>
<th>SPECIMEN 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>El</td>
<td>8.60 GPa</td>
<td>8.60 GPa</td>
</tr>
<tr>
<td>Nodal mass</td>
<td>922 kg</td>
<td>651 kg</td>
</tr>
<tr>
<td>Mod. of Elasticity</td>
<td>3.00, 3.25 kNm</td>
<td>1.80, 1.97 kNm</td>
</tr>
</tbody>
</table>

![FIG. 3 - The portal frames analysed in the computer models](image)

the strength of the nailed joint between the beam and columns, the assumed yield moments being 3.0 kNm and 1.8 kNm for frames 1 and 2 respectively. The nodal masses used for the dynamic analyses of the frames were 922 kg and 651 kg for frames 1 and 2 respectively. These values were arrived at by assuming that the beam end-moments for a uniformly distributed load were one half of those for a fully fixed ended beam when relating them to the respective yield moments in order to obtain the equivalent uniformly distributed load. This load was then used to determine the modal masses as well as the static loads in the portal frames.

The code working value of the joint moment is based on the initial 'linear' load-deflection curve but experimentally it had been found(12) that there is a softening of the structure with increase in deformation even though the load-carrying capacity of the joint continued to increase. This lowered stiffness value was obtained by modifying the value used for the modulus of elasticity, such that for specimen 1 the modified value was 0.42 of the code value (which was 8.60 GPa) while for specimen 2 the modified value was 0.32 of the code value. For each portal frame, two dynamic inelastic analyses were performed, one with the code working value for the modulus of elasticity and the other using the lower modified value.

The 'plastic' slope, P, of the moment-curvature diagram was kept constant and similar as far as possible for both choices for the modulus of elasticity for each specimen. The unloading stiffness was kept the same for both initial stiffness choices to give the possibility of obtaining similar moment-curvature diagrams. The parameter, a, which governs the unloading stiffness, is defined in equation 1. The various numerical values used are listed in Table 1 and were derived from the experimental results described in reference 12.

The earthquake used for all the analyses was the first 10 seconds of the El Centro, May 18, 1940 (N-S) record. This was used since it is usually taken as a "code calibration" earthquake record and has a greater effect on structures with periods in the 0.5 to 1.0 seconds range. While the elastic periods for the portal frames analysed using the code modulus of elasticity were less than 0.5 seconds, the periods were raised to this level when using the reduced moduli.

Since the second, third and fourth mode periods of the four analyses dropped sharply to about 0.01 - 0.025 seconds, a series of analyses were carried out using frame 1 with the code modulus of elasticity and a range of time steps in order to determine just how small the integration time step needs to be in order to obtain accurate results.

<table>
<thead>
<tr>
<th>Specimen 1</th>
<th>Specimen 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>E(GPa)</td>
<td>My (kNm)</td>
</tr>
<tr>
<td>8.60</td>
<td>3.00</td>
</tr>
<tr>
<td>3.61</td>
<td>3.25</td>
</tr>
<tr>
<td>8.60</td>
<td>1.80</td>
</tr>
<tr>
<td>2.75</td>
<td>1.97</td>
</tr>
</tbody>
</table>

**DUCTILITY**

Nailed joints in timber members do not display any true elastic and inelastic states but have a continuously changing stiffness giving rise to a curved load-deflection path. For the analyses reported herein, this continuous curve has been approximated by a bilinear curve. The flat portion of
the curve near failure (11) has been ignored as the deformation required has been assumed to be greater than that which would occur in a moderate to severe earthquake.

Related to damage and damage control, ductility factors are often used, especially in the comparison of the response of different structures. The displacement ductility is defined as

$$\mu = \frac{\Delta_{\text{max}}}{\Delta_y}$$

where $\Delta_{\text{max}}$ = maximum displacement

$\Delta_y$ = yield displacement

In the case of timber structures, the calculation of a displacement ductility factor is not as simple as for other materials having a sharply defined yield point and a definition of the yield displacement must be provided. In it the displacement at a factored code working load level with a modulus of elasticity equal to the code value, or is it the displacement at a factored code working level but with a modified modulus of elasticity based on a more accurate representation of the load-deflection diagram? From the plots of the cyclic maximum moment-curvature points given in Figures 8 and 14 of reference 12, a factor of approximately twice the code working load (or moment) seemed appropriate to define the 'yield' moment of the joint.

These alternative yield moments will lead to two alternative values for the modulus of elasticity, i.e. the code value and a reduced value associated with the higher value of the yield moment. Analyses were carried out using both choices for the modulus of elasticity and their associated yield moments.

The computer program used for the analyses automatically computes the curvature ductility at the ends of all members where inelastic rotation has taken place. The curvature ductility is defined as

$$\mu = \frac{\psi_{\text{max}}}{\psi_y}$$

where $\psi_{\text{max}}$ = maximum curvature of the member at the joint

$\psi_y$ = curvature of the member at the joint when the yield moment of the joint is first reached.

For an elastic member, the moment and curvature are related by the expression

$$M = EI\psi$$

and consequently the yield curvature, $\psi_y$, can be derived from the known yield moment.

RESULTS

Effect of Integration Time Step

Four analyses were carried out for frame one with values of 1/100, 1/200, 1/300, 1/400 seconds being used for the time step in integrating the equations of motion for the computer model. Displace-

ment and moment time history plots are given in Figures 4 and 5 for these four analyses. In both figures it appears that the analysis using a time step of 1/100 second differs considerably from those carried out using smaller time steps suggesting that this time step is too coarse to allow the higher modes to respond correctly (the first mode being a pure sway mode). From the figures it seems that a time step of 1/400 seconds is required for accurate results although a time step of 1/300 seconds would give almost the same results. For the remaining analyses reported herein, a time step of 1/400 second was used throughout.

By way of further comparison of the effect of the time step, the ductility requirements are shown in Table 2. The variation in maximum displacements and the ductility factors are often used, especially in the comparison of the response of different structures.
### TABLE 2: DUCTILITY REQUIREMENTS FOR DIFFERENT INTEGRATION PERIODS

<table>
<thead>
<tr>
<th>Integration time step (seconds)</th>
<th>Columns</th>
<th>Beams</th>
<th>( \lambda_{\text{max}} ) ductility (mm)</th>
<th>Curvature Ductilities</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Axial force</td>
<td>Maximum moment</td>
<td>Shear force</td>
<td>Axial force</td>
</tr>
<tr>
<td>1/100</td>
<td>6361</td>
<td>3700</td>
<td>2472</td>
<td>1345</td>
</tr>
<tr>
<td></td>
<td>6290</td>
<td>3883</td>
<td>-4734</td>
<td>1354</td>
</tr>
<tr>
<td></td>
<td>6173</td>
<td>4252</td>
<td>4081</td>
<td>1346</td>
</tr>
<tr>
<td></td>
<td>6245</td>
<td>4180</td>
<td>4471</td>
<td>1338</td>
</tr>
</tbody>
</table>

\( \delta y = 1.91 \text{ mm} \)

**FIG. 4** - Displacement-time history plots for frame 1 (code value of elastic modulus) for analyses using a time step of (a) 1/100th sec (b) 1/200th sec (c) 1/300th sec (d) 1/400th sec

**FIG. 5** - Moment-time history plots for frame 1 using different time steps as in FIG. 4.
FIG. 6 - Moment-curvature plots for the columns frame 1 using code modulus of elasticity

FIG. 7 - Moment-curvature plots for the columns of frame 1 using the reduced modulus of elasticity

FIG. 8 - Moment-time histories for frame 1 using (a) code modulus (b) reduced modulus

FIG. 9 - Displacement-time histories for frame 1 using (a) code modulus (b) reduced modulus

FIG. 10 - Moment-curvature plots for the columns of frame 2 using the code modulus of elasticity
FIG. 11 - Moment-curvature plots for the columns of frame 2 using the reduced modulus of elasticity

FIG. 12 - Moment-time histories for frame 2 using (a) code modulus (b) reduced modulus

FIG. 13 - Displacement-time histories for frame 2 using (a) code modulus (b) reduced modulus

FIG. 14 - Moment-curvature plots for the columns of frame 1 using code modulus for (a) $\alpha = 0.4$ (b) $\alpha = 0.6$
Ductility

Displacement ductility factors were calculated for both cases and are listed in Table 3 along with the curvature ductility factors for the two columns and the ends of both beams. The table shows that a greater ductility demand is incurred for analyses using the unmodified code value of the modulus of elasticity.

As the bilinear back-bone curve used to model the hysteretic behaviour of the joint region ignores effects such as initial slippage under small deformations, the initial stiffness will appear to be slightly greater than the probable true stiffness of the joints and hence will enhance the stiffness of the frame. For these reasons it would appear that the analyses using the modified modulus of elasticity give more realistic ductility requirements for the joints.

DISCUSSION

The moment-curvature diagrams obtained from the two sets of dynamic inelastic analyses appear to have the bilinear back-bone curve and pinched hysteresis loops typical of moment-resisting nailed joints[11]. There are discrepancies between the computer representation and the experimental results, especially with the sudden changes in the model - a result of the trilinear representation of the assumed cubic function in the moment-curvature relationship.

Analyses were carried out with two different initial stiffnesses for the members. The response of the structure for these two cases was virtually identical except that for the case where the stiffness had been softened off, the amount of permanent 'plastic' displacement was almost nil. In all cases, the moment-curvature hysteresis loops were displaced to one side. Although the value of the initial stiffness used in the analysis had no major effect on the structural response it does effect markedly the ductility factor required by the structure as this is dependent on the yield moment and initial stiffness of the members.

To compare the behaviour of timber structures to that of other structures the ductility factor is commonly used as the benchmark. However, for a timber structure the ductility factor must be mentioned in conjunction with the initial stiffness used in the analysis because, for the case of the code level of stiffness being used, a large ductility factor does not necessarily mean that the design should provide that level of ductility. This is because the true initial stiffness is somewhat less than the code level of stiffness and the required ductility factor is consequently reduced (as can be seen from Table 3). This then brings up the unanswered questions as to what is the initial stiffness of a timber structure which is to provide the primary lateral load resisting system for the structure. This question is even more important when considering the structure's period in determining the lateral load required for the equivalent elastic code design. As can be seen from the two initial stiffness cases used in the inelastic analyses, there was approximately a 50% increase in the elastic period of the structure. This softening of the initial stiffness could lead to a reduction in the required code lateral load resistance of the structure (as per NZS 4203:1080 Figure 3). However, the choice of the initial stiffness value had little effect on the apparent inelastic periods of the structure, as the moment and displacement time histories for both frames (Figures 8 and 9, 12 and 13) showed that the periodicity was similar whether the code value or the reduced value was used for the modulus of elasticity.

In order to gauge the effect of the unloading stiffness, additional analyses were carried out on frame 1 using the code elastic modulus. The results of using 0.4 and 0.6 for the unloading parameter, a (see equation 4) are compared with those for a = 0.5 in Table 4. It can be seen that as the unloading stiffness decreases (i.e. a increases), the maximum displacement ductility and column curvature ductility requirements increase while the beam ductility requirement decreases. As a increases there is a greater degradation of the stiffness of the joints. The moment-curvature plots for a = 0.4 and 0.6 are shown in Figure 14 and should be compared with those for a = 0.5 in Figure 6.

Figure 15 shows the moment-curvature diagram for one column joint of frame 1 using the reduced modulus of elasticity and subject to the Pacoima Dam earthquake record. Here the maximum curvature reached is much greater than for the El Centro earthquake and in spite of the bias in one direction, the development of the pinching behaviour can be clearly seen.

FIG. 15 - Moment-curvature diagram for frame 1 using reduced modulus under Pacoima Dam earthquake.

As discussed above, the computer model as developed does give hysteresis loops that display pinching characteristics, however, the rules that govern the behaviour of the loops may be too restrictive in two ways. The first restriction is that the loops are dependent on the absolute maximum as in the modified Takeda model. This could possibly be relaxed by incorporating a scaling factor to relate the present hysteresis loops to the directional maximum thus.
obtaining some independence between the positive and negative directions. This will also remove the requirement for symmetry about the origin. The other restriction is that degradation characteristics have not been included in the present model. In setting up the present model, the overriding requirement was that the model should be kept reasonably simple consistent with adequately modelling the pinching characteristics of a nailed timber joint.

Also, more information is obviously required on the dynamic characteristics of timber joints such as modelled herein in order that the parameters required for a dynamic inelastic analysis could be determined. Parameters such as a 'yield' load, possibly in the form of a factored working load, an initial stiffness for the member in relation to the period of the structure, and the 'plastic' stress in all the further investigation and discussion in relation to particular joint configurations.

To go a step further, dynamic inelastic analyses should be carried out on three and four storey timber buildings designed according to NZS 4203:1980 to determine the response of such structures and to determine the maximum curvature ductility demand. A further item to be considered is whether the bilinear load-deflection curve needs to be replaced with a tri-linear curve with a flatter third region.

SUMMARY AND CONCLUSIONS

The model developed to represent the hysteretic behaviour of a moment-resisting timber joint does appear to fulfill the requirement of displaying the pinching characteristics associated with such timber joints. From the very small data base available, various parameters were determined though these were all dependent on the chosen value for the initial stiffness of the member.

Various questions have been raised by this research. The most important one is what exactly should the initial stiffness of a timber member be when considering a dynamic inelastic analysis. Clearly, a code value for the modulus of elasticity appears to lead to a structure much stiffer than it really is, whilst the experimental data suggests a values of 0.3 - 0.4 times the code value for the modulus of elasticity. Clearly, with this further investigation and discussion in relation to particular joint configurations.

From the data obtained to date, the following values for the parameters used in the hysteretic model are suggested:

Case (i) \( E_{\text{mod}} = 1.0 \ E_{\text{code}} \)

\[
\begin{align*}
\frac{P}{a} & = 0.045 \pm 0.005 \\
\alpha & = 0.45 \pm 0.05
\end{align*}
\]

Case (ii) \( E_{\text{mod}} = 0.3 - 0.4 \ E_{\text{code}} \)

\[
\begin{align*}
\frac{P}{a} & = 0.12 \pm 0.02 \\
\alpha & = 0.15 \pm 0.05
\end{align*}
\]

For an elastic lateral code analysis it is suggested that the period of the structure be determined using a modified modulus of elasticity of perhaps 0.4 \( E_{\text{code}} \). This would lengthen the predicted natural elastic period to give a value more representative of the structure after some hysteretic behaviour than would be obtained using the code elastic modulus. The result might possibly lower the design load requirements, though a value of 0.5 to 0.6 could be used if it is desired to err on the side of conservatism.

A ductility factor can be applied to a timber structure but exactly what it represents needs to be examined closely. If a displacement ductility is to be used, then consideration needs to be given to a suitable definition of the yield displacement. It appears from the results presented in this paper that any definition of the ductility factor should be related to a modified initial stiffness.

Modelling of the hysteretic behaviour of a timber joint incorporating nailed metal sideplates has been shown to be possible and leads to realistic computer modelling of the joint behaviour. However, development of such moment-resisting timber joints is still at an early stage and more research needs to be undertaken to determine their characteristics before the computer model can be used to reliably predict the overall structural behaviour and aid in the future design of multi-storey timber structures.

REFERENCES


Division of Forest Products, Australia, 1966.


---

**TABLE 3**: DUCTILITY REQUIREMENTS UNDER EL CENTRO (1940) (N-S) \( \Delta T = 1/400 \) seconds

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>Time (sec)</th>
<th>My (kNm)</th>
<th>Axial force (kN)</th>
<th>Axial force (kN)</th>
<th>Axial force (kN)</th>
<th>Axial force (kN)</th>
<th>Displacement (mm)</th>
<th>Curvature ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>0.34</td>
<td>0.60</td>
<td>3.00</td>
<td>6245</td>
<td>4180</td>
<td>1338</td>
<td>2179</td>
</tr>
<tr>
<td></td>
<td>0.52</td>
<td>0.57</td>
<td>0.875</td>
<td>3.25</td>
<td>5764</td>
<td>4291</td>
<td>1320</td>
<td>2143</td>
</tr>
<tr>
<td></td>
<td>0.42E</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.28</td>
<td>0.035</td>
<td>1.00</td>
<td>4165</td>
<td>2805</td>
<td>409</td>
<td>1746</td>
<td>1.46</td>
</tr>
<tr>
<td></td>
<td>0.22E</td>
<td>0.50</td>
<td>0.628</td>
<td>1.97</td>
<td>4020</td>
<td>2806</td>
<td>907</td>
<td>1403</td>
</tr>
</tbody>
</table>

**TABLE 4**: EFFECT OF VARIATIONS IN THE UNLOADING STIFFNESS PARAMETER, \( \alpha \)

<table>
<thead>
<tr>
<th>Unloading stiffness parameter ( \alpha )</th>
<th>Axial force (kN)</th>
<th>Maximum moment (kNm)</th>
<th>Maximum shear (kN)</th>
<th>Axial force (kN)</th>
<th>Maximum moment (kNm)</th>
<th>Maximum shear (kN)</th>
<th>Displacement ductility ( \mu )</th>
<th>Curvature ductility</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.4</td>
<td>6191</td>
<td>4127</td>
<td>1993</td>
<td>1337</td>
<td>2277</td>
<td>6748</td>
<td>35.36</td>
<td>19.5</td>
</tr>
<tr>
<td></td>
<td>2886</td>
<td>-2473</td>
<td>-1824</td>
<td>61</td>
<td>-3165</td>
<td>3452</td>
<td>-55.50</td>
<td>29.1</td>
</tr>
<tr>
<td>0.5</td>
<td>6245</td>
<td>4180</td>
<td>2015</td>
<td>1338</td>
<td>2179</td>
<td>6654</td>
<td>44.17</td>
<td>23.1</td>
</tr>
<tr>
<td></td>
<td>2421</td>
<td>-3565</td>
<td>-1852</td>
<td>13</td>
<td>-3157</td>
<td>3600</td>
<td>-58.25</td>
<td>30.5</td>
</tr>
<tr>
<td>0.6</td>
<td>6335</td>
<td>4200</td>
<td>2099</td>
<td>1339</td>
<td>2582</td>
<td>6829</td>
<td>52.65</td>
<td>27.6</td>
</tr>
<tr>
<td></td>
<td>2331</td>
<td>-3749</td>
<td>-1902</td>
<td>-21</td>
<td>-3094</td>
<td>3307</td>
<td>-59.82</td>
<td>31.3</td>
</tr>
</tbody>
</table>

\( \delta y = 1.91 \) mm