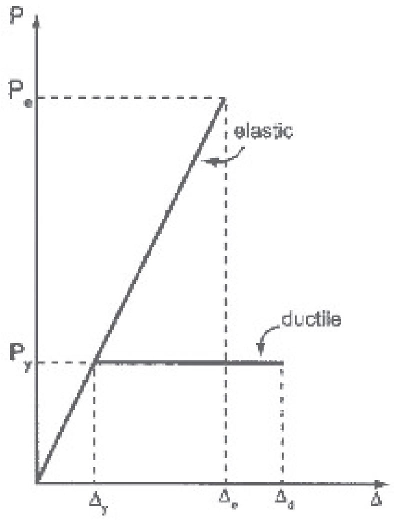
INFORMATION SHEET
SEISMIC DESIGN

DUCTILITY

The information provided below has been taken from the New Zealand Timber Design Guide 2007, published by the Timber Industry Federation and edited by Professor A H Buchanan. To purchase a copy of the Timber Design Guide, visit www.nztif.co.nz

Diagram 1 shows the idealized relationship between the total lateral force P at the top of a single storey structure and its lateral displacement, Δ. The straight sloping line illustrates perfectly elastic behaviour. The horizontal line shows the alternative, perfectly plastic, response once the structure has exceeded its yield displacement, known as elasto-plastic behaviour.

Diagram 1: Lateral force vs displacement for elastic and ductile structures



If the structure responds elastically to a given level of earthquake ground motion, it will be subjected to a maximum lateral displacement, Δ_e. This structure would be designed by applying an equivalent static lateral force of P_e to calculate design actions for each of the structural components. Alternatively, if the structure followed an idealised elasto-plastic response it would require less strength, P_y so that when subjected to the same earthquake, the structure would yield once it reached a displacement of Δ_y and respond plastically. Its maximum lateral displacement of Δ_d would normally be greater than Δ_e. The structural ductility μ is defined as the ratio of the maximum displacement to the yield displacement:

$$\mu = \Delta d / \Delta y$$

In most timber structures the maximum lateral displacement Δ_d will be slightly greater than elastic displacements Δ_e. In taller structures (those with their natural period > 0.7 seconds), the lateral force P_y is directly related to the elastic response force P_e according to:

$$P_y = P_e / \mu$$

Designers using NZS 1170.5 are required to calculate the ductility of a structure. That standard suggests that well detailed symmetrical timber structures can achieve the following levels of ductility:

μ = 1.25 for nominally ductile structures

μ = 3 for moment-resisting timber frames

μ = 4 for timber walls, or for braced frames with yielding occurring in both tension and compression.

Nominally ductile structures are those with steel connections capable of ductile performance, where specific calculations and detailing for ductility are not required. Structures which could fail in a sudden brittle manner must be designed for elastic response using μ = 1.0.

DUCTILITY OF THE CONNECTIONS

Because wood is inherently brittle, ductility is provided in timber structures by designing ductile steel connections, and avoiding any possibility of wood failures. The amount of ductility available in any connection depends on its type and geometry, and the level of displacement capacity relative to the yield displacement.

Effect of displacement

Diagram 2 shows the simplified behaviour of a typical nailed connection, and 3 shows the same for a bolted connection made with small diameter bolts. The only difference between the graphs is the initial low stiffness in the bolted connection due to some movement of the bolt in its hole. These graphs show behaviour under monotonic loading to failure, with no load reductions or load reversals. The initial linear elastic section of the graph (usually to about 40% of ultimate strength) is followed by a curve into a yield plateau, and eventual failure is shown by the dotted line. Depending on definitions, the yield force P_y is associated with the yield displacement Δ_y, the ultimate displacement is Δ_u and the available ductility μ_c of the connection is given by

$$\mu_c = \Delta_u / \Delta_y$$

The available ductility will depend on many factors such as the end and edge distances of the fastener, and the withdrawal resistance of some fasteners such as tooth-plate connectors.

Diagram 2: Idealised plot of force vs displacement for a nailed connection

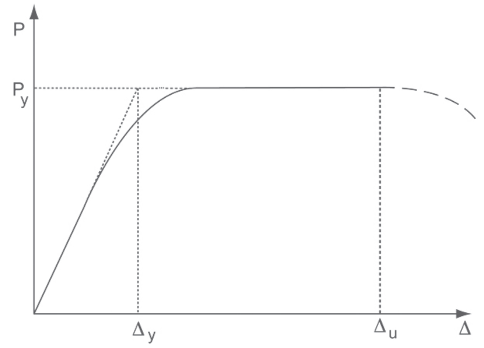


Diagram 3: Idealised plot of force vs displacement for a bolted connection

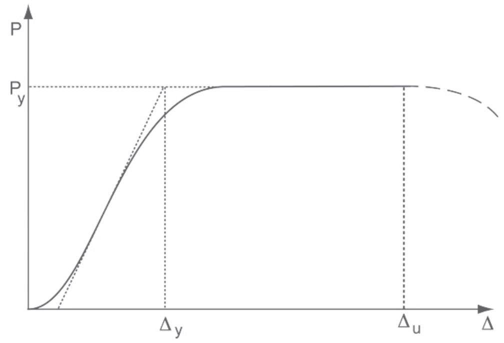
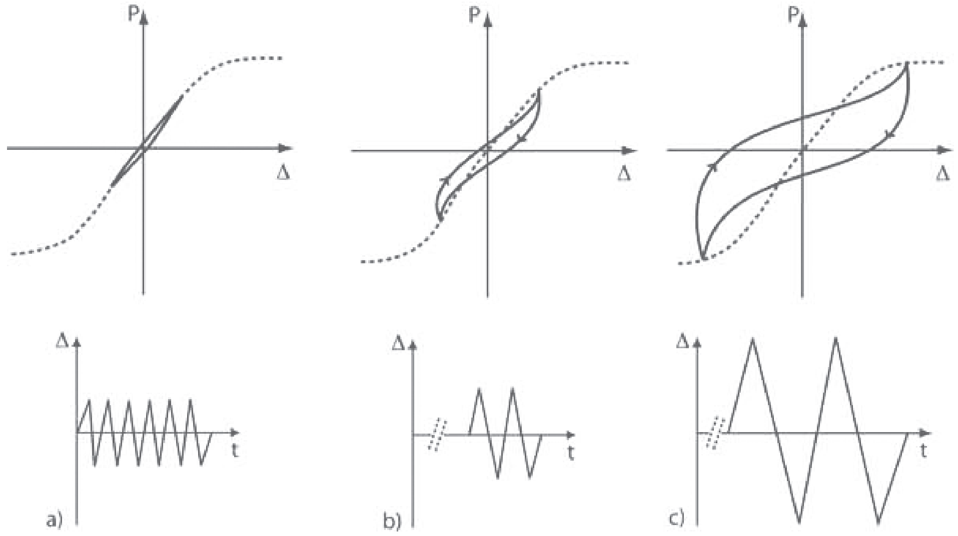


Diagram 4 shows how the these fasteners (with no initial bolt slip) would respond to reversed cyclic loading over a range of displacement levels. The three graphs show how the shape of the hysteresis loop changes as the displacement is increased.

The dotted line represents the upper bound of force for each of the displacement cycles, and this is very similar to the load-displacement plot for monotonic loading. The area inside the solid loop is a representation of the amount of energy absorbed in the connection during the earthquake. Large displacements (diagram 4 (c) absorb more energy but they also lengthen the period of vibration during earthquake shaking, hence reducing the strength demand. A problem with large loops of this shape is the large residual displacements remaining when the force drops to zero at the end of the test (or at the end of the earthquake).

Diagram 4: Hysteresis loops for cyclic loading of a fastener at various levels of displacement 2



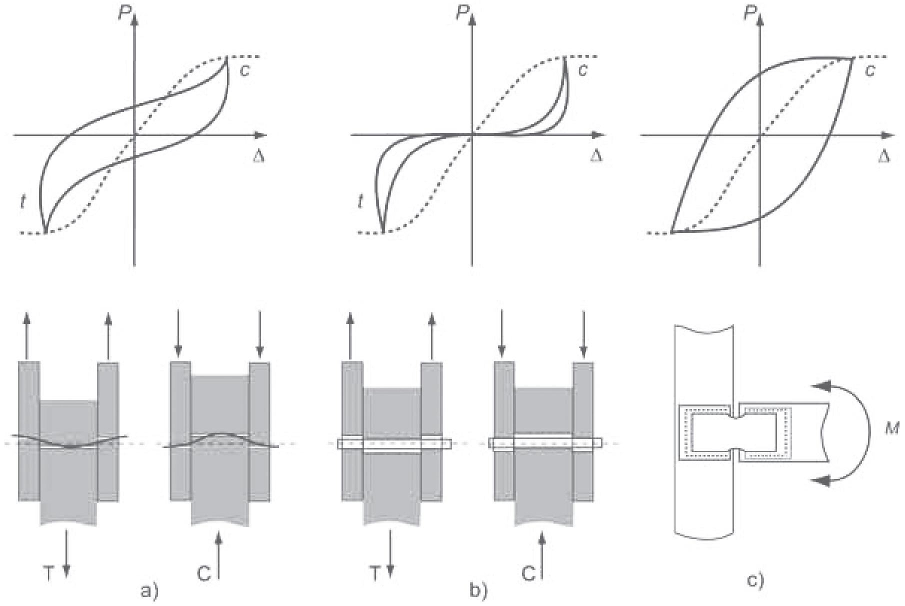
Effect of connection geometry

Diagram 5 shows the effect of connection geometry on the shape of the hysteresis loops. Diagram 5 (a) shows that small diameter steel fasteners develop ductility by flexural yielding of steel with only a small amount of wood crushing. Diagram 5 (b) shows the hysteresis loop for a system with crushing of wood fibres and no steel yielding. This results in a very “sloppy” connection (similar to a diagonally braced structure with tension-only yielding). Diagram 5 (c) shows a connection relying on flexural yielding of steel for ductility, producing desirable fat loops, but again with large residual displacements when the force drops to zero.

New types of structural timber systems are being developed to reduce residual displacements at the end of the earthquake. These use post-tensioned steel tendons, which remain elastic for the duration of the earthquake and pull the building back to the zero displacement position.

Pinched hysteresis loops (moderate in diagram 4 and diagram 5 (a), severe in diagram 5 (b) is the result of wood crushing in the nailed or bolted connections, leading to reduces stiffness, large displacements and high levels of damage.

Diagram 5: Hysteresis loops for cyclic loading of different types of fastener 2



Overstrength factor

The capacity design procedure for any structure requires that the non-ductile parts of the structural system be over-designed to avoid premature failure. This requires that all the non-yielding structural elements be designed for the actions obtained from the initial analysis, multiplied by an overstrength factor. The value of overstrength factor represents the amount by which the actual strength of the yielding components at large displacements may exceed the design strength. The overstrength factor depends on several factors:

- The strength reduction factor ϕ used in the initial design
- Strain-hardening in the steel of the yielding components due to large displacements
- The difference between the 5th percentile (characteristic) strength used for design of the yielding components and the 95th percentile strength value which could be present
- Any other unintentional over-design of the weakest components.

For nailed plywood connections in most timber structures, the overstrength factor has a value of ϕ_o = 2.0. This must be used for design of the non-yielding components, such as the chords and holding-down brackets in a plywood shearwall. This figure is obtained from tests which show that the actual strength (at large displacements) of nailed connections between plywood and timber is approximately two times the design strength of nails in NZS 3603 multiplied by the special factors for plywood connections and large numbers of nails.