TECHNICAL NOTE - CAPACITY DESIGN OF LIGHT TIMBER-FRAMED PLYWOOD SHEAR WALLS

Hayato Auman, Lucas Seña Caicedo,
Silvester Clark Consulting Engineers

ABSTRACT

This worked example is intended to demonstrate the fundamental steps in detailing capacity designed multi-storey timber-framed walls. These calculations closely follow guidance presented in the BRANZ publication, Multi-storey Light Timber-Framed Buildings in New Zealand – Engineering Design (October 2019).

The calculations are limited to strength design only, and displacements may govern the structure's lateral load design; therefore, understanding system displacements is essential, and an iterative approach may be required. In the future, a paper on displacement and ductility calculations will be published.

1 CASE STUDY STRUCTURE

An elevation of the wall to be designed is shown in Figure 1 while Table 1 shows dimensions and gravity loads applied. All framing elements are to be 140x45 SG8 timber while 140 mm thick cross-laminated timber is used as flooring. A rigid concrete foundation is provided at the base of the wall.

Table 1 Wall geometry and gravity loading

Wall geometry	
Wall length	L_{wall}= 3000 mm
Storey height	$H_{1,2,3}$ = 3000 mm
Wall gravity loading	
G+0.3Q (UDL)	$W_{1,2}$= 5 kN/m W_3= 2 kN/m

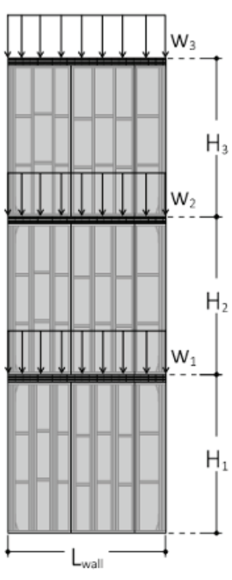

Figure 1 Wall geometry

2 DESIGN DEMANDS

Designers must assume and verify the target displacement ductility and force distribution on a case-by-case basis. In this example, assume that an equivalent static distribution of loads has been calculated for the building using a global displacement ductility factor of μ = 2.0 (including dynamic magnifications as appropriate) and apportioned to each shear wall via a rigid diaphragm analysis as shown in Table 2 and illustrated in Figure 2.

Table 2 Design demands

Level	Elevation	ESA Force	Storey shear	Overturning moment
		Fi	Vi	Mi
	(m)	(kN)	(kN)	(kNm)
L3	9	10	10	0
L2	6	8	18	31
L1	3	4	22	86
Base	0	-	-	153

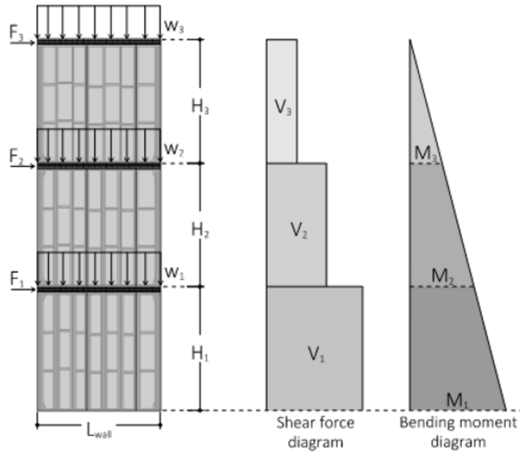

Figure 2 Lateral loading and action diagrams

3 DESIGN WALL COMPONENTS

3.1 Design of fasteners

All nonlinearity is designed to occur in the nailed connections between the sheathing and timber framing. The nail capacity is determined using NZS AS 1720.1:2022. For this example, the step-by-step procedure is not depicted, and the procedure outlined in NZS AS 1720.1:2022 should be followed.

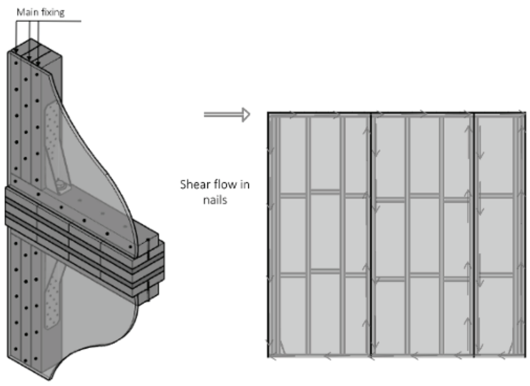

Figure 3 Left: Nailing of plywood to framing elements. Right: Shear flow demands in wall panel.

The shear flow at a storey represents the shear demand per unit length of wall and is used to determine the nailing required:

$$v_i^* = \frac{V_i}{L_{wall}} \quad (1)$$

At the third storey, the shear flow demand is therefore

$$v_i^* = \frac{10\,kN}{3\,m} = 3.3\frac{kN}{m}$$

The design capacity of a single 2.80 mm nail under seismic loading is determined following the detailed procedures in NZ AS1720.1:2022 and found to be 0.575 kN using the parameters listed in Table 3.

Table 3 Nail Properties

Nail 2.8x60mm	
Characteristic tensile strength f_{uf}	800 N/mm^2
Characteristic density ply $\rho_{k,ply}$	480 kg/m^3
Characteristic density ply $\rho_{k,SG8}$	375 kg/m^3
Ply thickness	9 mm
Factor for load duration k_1	1.14

The design capacity of a nailed connection is further multiplied by a factor k_{17} accounting for multiple nails, with a value of 1.0 for 4 nails to 1.3 for 50 nails along a single edge of wall. The resulting design connection capacity can be iterated to achieve a value close to the shear flow demands.

The perimeter shear flow, nail spacings and design capacities at each storey are listed in Table 4. Since plywood comes in standard sizes, sheets may not extend the full height and length of the walls as illustrated in Figure 2. In these cases, nailing along the internal edges should be determined using shear flow demands calculated specific to the sheet dimensions used. In practice, different nailing patterns can be used to transfer loads properly to all studs.

Table 4 Design nail capacities.

Level	Vi *	v_i^* **	S_i ***	n ****	k_{17}	$\phi N_{\alpha,y,u}/S_i$ *****
	(kN)	(kN/m)	(mm)			(kN/m)
L3	10	3.3	160	19	1.09	3.91
L2	18	6.0	100	30	1.17	6.72
L1	22	7.3	80	38	1.22	8.77

* Storey shear; ** Shear flow demand; *** Spacing provided; **** Number of nails along one edge; ***** Design capacity

The minimum spacing as prescribed in NZS AS 1720.1:2022 is 28 mm (10D for holes not prebored in radiata pine) so the spacings calculated are acceptable.

Note that in this example, it is assumed that serviceability limit state fastener spacing requirements do not govern. This assumption should be confirmed by designers in practice.

3.2 Determination of overstrength actions in fasteners

The overstrength capacity of the nailed connections is determined to allow sizing of capacity-protected elements such as plywood sheathing, end chords, hold-down connectors and plates to ensure all nonlinear behaviour occurs in the nailed connections. The overstrength factor for nails is 1.6 as prescribed in NZS AS 1720.1:2022 As the overstrength capacity is calculated for a single nail, the k_{17} factor should be discounted. The overstrength shear flow at Level 3 is as follows:

$$v^*_{1,CPE} = \frac{\phi_o}{\phi}\frac{N_{\alpha,y,u}}{k_{17}s} \quad (2)$$

$$= \frac{1.6}{0.8}\frac{3.91kN/m}{1.09\ \cdot 1.0}$$

$$= 7.2\ kN/m$$

Table 5 lists the overstrength shear flows at all levels which are used to dimension the remaining components in the shear wall.

Table 5 Overstrength shear flow demands

Level	Design Capacity	Overstrength Shear Flow
	$\phi N_{\alpha,y,u}/s_i$	$v^*_{1,CPE}$
	kN/m	kN/m
L3	3.91	7.2
L2	6.72	11.5
L1	8.77	14.38

3.3 Design of sheathing panels

The plywood sheathing is dimensioned based on the overstrength shear flow:

$$v_{d,i} = \phi k_1 k_{19} k_{12} g_{19} f_s' \frac{2}{3} t \geq v^*_{1,CPE} \qquad (3)$$

Where ϕ = 1.0 for elements designed for overstrength actions; k_1 = 1.0 for peak actions lasting less than 5 seconds; k_{19} = 1.0 for plywood of stress grade > F7 and web thickness ratio < 19; k_{19} = 1.0 for plywood with a moisture content of 15% or less

Consider 9 mm thick Grade F8 plywood at all storeys. Its capacity is determined as follows:

$$v_{d,i} = 1.0 \cdot 1.0 \cdot 1.0 \cdot 1.0 \cdot 1.0 \cdot 4.2 \cdot \frac{2}{3} \cdot 9$$

$$= 25.2\ kN/m$$

This exceeds the maximum overstrength shear flow demand, therefore it can be used at all storeys.

3.4 Design of compression chord

The capacities of the tension and compression chords are verified next. An assumption of the sizes of chords is required at this stage since the distances between chord centroids are used to determine their axial demand, as illustrated in Figure 4. We will assume the that triple studs are used at ground and first floor levels, while double studs are used at second floor level. We will also assume that two lines of blocking are provided evenly along the studs' and chords' heights.

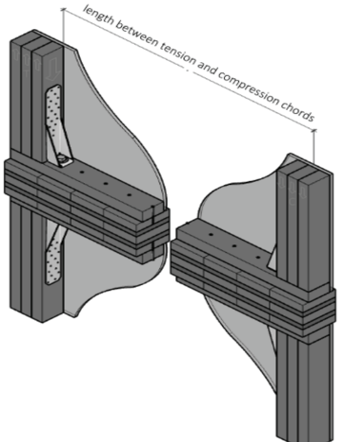

Figure 4 Lever arm between tension and compression chords

The incremental seismic chord force due to nail overstrength at the i^th storey can be determined as follows:

$$\Delta T_{i,CPE} = \Delta C_{i,CPE} = \frac{M_i}{l_{c,i}} = v^*_{1,CPE} \cdot h_i \cdot \frac{l_i}{l_{c,i}} \qquad (4)$$

Where $l_{c,i}$ = length between tension and compression chord centroids.

At the top storey, the incremental seismic chord force is

$$\Delta T_{2/F,CPE} = 7.2\frac{kN}{m} \cdot 3m \cdot \frac{3\ m}{(3\ m - 0.045 \cdot 3)}$$

$$= 21.36\ kN$$

The cumulative chord force at the ith storey is found by adding the incremental chord forces of all storeys above to that at the storey of interest Table 6 lists the incremental and cumulative seismic chord forces at all storeys:

Table 6 Seismic chord axial demands

Level	Overstrength Shear Flow	Chord width	Incremental seismic chord force	Cumulative seismic chord force
	$v^*_{1,CPE}$	B_{chord}	$\Delta T/C_{i,CPE}$	$T/C_{i,CPE}$
	(kN/m)	(mm)	(kN)	(kNm)
L2	7.2	90	22.61	21.61
L1	11.5	135	36.99	58.60
Base	14.38	135	46.26	104.86

The design axial demand is then determined by adding gravity axial loads to seismic demands as shown in Table 7. The gravity axial load on the chords is determined based on its tributary area:

$$T_i^* = T_{i,CPE} - (G + 0.3Q) \cdot l_i/2$$

$$C_i^* = C_{i,CPE} + (G + 0.3Q) \cdot l_i/2$$

Table 7 Design chord axial demands

Level	$\frac{T}{C_{i,CPE}}$ *	$\frac{G+0.3Q}{2}$	$\Delta N_{i,G+0.3Q}$ **	$N_{i,G+0.3Q}$ ***	T_i^* ****	C_i^* *****
	(kN)	(kN/m)	(kN)	(kN)	(kN)	(kN)
L2	21.61	1	0.4	0.4	21.2	22
L1	58.60	2.5	1	1.40	57.2	60
Base	104.86	2.5	1	2.40	102.46	107.26

* Cumulative seismic chord force; ** Incremental gravity axial load; *** Cumulative gravity axial load; **** Design chord tension axial load; ***** Design chord compression axial load.

The capacity of a single 140x45 SG8 stud is determined to check whether the assumed chord build-up possesses adequate strength. The height considered for buckling about the strong axis is calculated by subtracting the floor and plate thicknesses from the interstorey height. The height considered for buckling about the stud's weak axis is taken as the height between dwangs/plates. The slenderness ratios about the strong (S_3) and weak (S_4) axes are calculated thus:

$$S_3 = \frac{g_{13} h_{stud}}{b_{chord}} \tag{6}$$
$$= \frac{0.9 \cdot (3000 - 2 \cdot 45 - 140)}{140}$$
$$= 17.8$$

$$S_4 = \frac{L_{ay}}{b_{chord}} \ (7) \tag{7}$$
$$= \frac{(3000 - 2 \cdot 45 - 140)/3}{45}$$
$$= 20.5$$

The stability factor requires calculation of the material constant based on Appendix E of AS 1720.1 (calculated for r=1):

$$\rho_c = 11.39 \left(\frac{E}{f'_c}\right)^{-0.408} r^{-0.074} \tag{8}$$
$$= 11.39 \left(\frac{8000}{18}\right)^{-0.408} 1^{-0.074}$$
$$= 0.95$$

The stability factor can then be calculated by considering the greater slenderness ratio following the procedure in Cl. 3.3.3 of AS 1720.1:

$$\rho_c S = 0.95 \cdot 20.5$$
$$= 19.5 \tag{9}$$

For $10 \leq \rho_c S \leq 20$,

$$k_{12} = 1.5 - 0.05 \rho_c S \tag{10}$$
$$= 1.5 - 0.05 \cdot 19.5$$
$$= 0.53$$

The compression capacity can be determined as follows:

$$N_{d,c} = \phi k_1 k_4 k_6 k_{12} f'_c A_c \tag{11}$$

Where k_4= 1.0 seasoned timber with an in-service moisture content less than 15%; k_6 = 1.0 for covered timber structures under ambient conditions in New Zealand; f'_c = 18 MPa for SG8 timber; A_c= cross sectional area of a single stud.
The design capacity of a 140x45 SG8 stud is therefore:

$$N_{d,c} = 1.0 \cdot 1.0 \cdot 1.0 \cdot 1.0 \cdot 0.53 \cdot 18MPa \cdot 140mm \cdot 45mm$$
$$= 60.1 \text{ kN}$$

This capacity can be multiplied by the number of studs used as chords at every storey as shown in Table 8:

Table 8 Verification of compression chord capacities

Level	Chord build-up	Capacity-protected compression capacity	Design chord compression axial load
L3	2-140x45	120.2	22
L2	3-140x45	180.3	60
L1	3-140x45	180.3	107.26

3.5 Verification of bottom plate compression strength perpendicular-to-grain

Now that the compression chords are dimensioned, the bottom plate supporting the chords are checked to ensure that the bearing area loaded in compression perpendicular-to-grain, shown in Figure 5, is adequate.

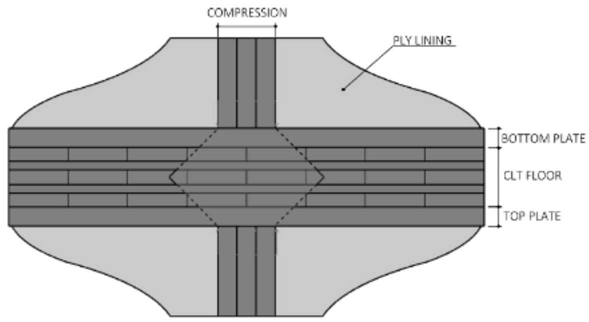

Figure 5 Perpendicular-to-grain compression of bottom plate under chords.

The compression strength of the bottom plate under the compression chord is calculated as follows:

$$N_{d,p} = \emptyset\, k_1\, k_4\, k_6\, k_7\, f'_p\, A_p \tag{12}$$

Where ϕ = 1.0 for elements designed for overstrength actions; k_1 = 1.0 for peak actions lasting less than 5 seconds; k_4 = 1.0 seasoned timber with an in-service moisture content less than 15%; = 1.0 for covered timber structures under ambient conditions in New Zealand; k_7 = 1.0 for bearing perpendicular to grain; f'_p = 6.9 MPa for SG8 timber; A_p = Bearing area for loading perpendicular to grain.

For the area under the compression chords at Level 2:

$$N_{d,p} = 1.0 \cdot 1.0 \cdot 1.0 \cdot 1.0 \cdot 1.00 \cdot 6.9 \cdot (2 \cdot 45mm \cdot 140mm)$$
$$= 86.9\ kN$$

The compression strengths of bottom plates perpendicular-to-grain are listed in Table 9:

Table 9 Verification of bottom plates

Level	Chord build-up	Bearing area for loading perpendicular-to-grain	Compression strength perpendicular-to-grain	Design chord compression axial load
		A_P	$N_{d,p}$	C_i^*
		mm^2	kN	kN
L3	2-140x45	12600	86.9	22.00
L2	3-140x45	18900	130.41	60.00
L1	3-140x45	18900	130.41	107.26

While the bottom plates have sufficient perpendicular-to-grain compression strength, their capacities are lower than those of the compression chords – it is possible that chord sizes need to be increased to ensure the bearing area below the chords is adequate. Moreover, it is possible for chord sizing to be governed by stiffness – chords need to be dimensioned to ensure deflections and drifts are within acceptable limits. Additionally, if the bottom plate material is much stronger than the floor material, the bearing strength of the floor may govern.

3.6 Design of hold-down connectors

In this example, non-specific proprietary brackets will be used to resist chord tension forces as shown in Figure 6. While compression loads usually govern the sizing of chords, the designer must ensure that checks relating to the hold-down connector selected (for both ductile and brittle failure modes) are performed. Consider proprietary brackets with capacities as listed in Table 10:

Table 10 Verification of hold-down connectors

Level	Bracket	Bracket design capacity	Design chord tension axial load
			$N_{d,p}$
L2	XHD60	60	21.20
L1	XHD60	60	57.20
G	XHD110	110	102.46

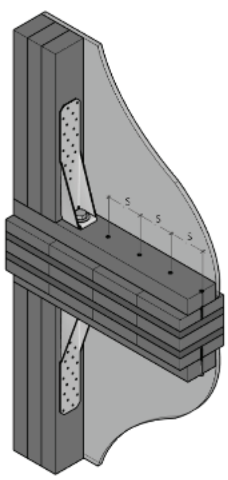

Figure 6
Hold-down connector and bottom plate fixings into floor

The capacities of timber connectors are sensitive to the materials to which they attach and achieve varying capacities for different types and grades of timber, as well as their moisture content.

Proprietary connectors typically have technical assessment reports that include checks that designers should consider. It is again emphasized that brittle failure modes of the connection should be checked according to NZS AS 1720.1:2022.

3.7 Design of shear connection to flooring

The bottom and top plate fixings to the floor (shown in Figure 6) are verified to ensure that the loads are able to be transferred to the wall. This connection should be checked against demands from a diaphragm analysis using a pESA envelope. Assume in this case that the governing shear flow in the wall-diaphragm interface is equal to $\upsilon^*_{1,CPE}$.

The design capacity of a single 2.80 mm nail under seismic loading is assumed to be 0.60kN. The procedure outlined in NZS AS 1720.1:2022 should be followed to calculate the capacity of the nails.

Table 11 Verification of hold-down connectors

Level	Overstrength Shear Flow	Spacing provided *
	$\upsilon^*_{1,CPE}$	
	kN/m	mm
L3	7.2	80
L2	11.5	50
L1	14.38	40

** Nails could be placed in two rows if required*

In the calculation of fastener spacing, it is important for designers to consider that the effective length over which the fasteners can be installed is less than the total wall length due to the presence of studs and chords. Wood screws or proprietary angle brackets may also be used if higher connection strengths are required.

4. LIMITATIONS

The calculation of deflections is not shown here and may be the subject of a subsequent technical note. This includes confirming the adequacy of chord stiffnesses, checking displacement demand in the fasteners to ensure these do not exceed maximum limits and checking the ductility demand in the fasteners is in line with the system ductility initially assumed to derive the wall seismic demands. Readers may refer to [1] for guidance on these.

5. ACKNOWLEDGEMENTS

The authors gratefully acknowledge the contributions of ENGCO Consulting Engineers and PTL Structural Consultants in reviewing this technical note.

6. REFERENCES

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