DEVELOPMENT OF DESIGN TABLES FOR COTTAGE CONSTRUCTION

1. ROOF TIE DOWN

Assumptions:

- Applicable only to regions A, B and C as specified by AS/NZS1170.2:2002
- Average wind recurrence interval: 500 years
- Terrain category 3, h≤10m
- Outside local topographic zone of hills, ridges and escarpments
- Roof slope $\alpha \geq 15°$
- Roof area ≥ $100m^2$
- External walls fully enclosed and without significant openings
- Applicable only to regular rectangular shaped houses with roof trusses supported by two end walls

1.1 Calculation of uplift wind pressure on roof for region A

Site wind speed

$$V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t)$$

$V_R = 45$ m/s	(Table 3.1, AS/NZS1170.2:2002: 500 years return, region A)
$M_d = 1.0$	(Table 3.2, AS/NZS1170.2:2002: Any direction)
$M_{z,cat} = 0.83$	(Table 4.1(a), AS/NZS1170.2:2002: Terrain category 3, h≤10m)
$M_s = 1.0$	(Clause 4.3, AS/NZS1170.2:2002)
$M_t = 1.0$	(Clause 4.4, AS/NZS1170.2:2002: outside local topographic zone)

$$\therefore V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t) = 45 \times 1 \times 0.83 \times 1 \times 1 = 37.4 \text{ (m/s)}$$

Design wind speed

$V_{des,\theta} = V_{sit,\beta}$ =37.4 m/s (Clause2.3, AS/NZS1170.2:2002: any wind direction)

Design wind pressure

$p = (0.5\rho_{air})(V_{des,\theta})^2 C_{fig} C_{dyn}$ (Clause2.4.1, AS/NZS1170.2:2002)

$\rho_{air} = 1.2$ kg/m^3 (Clause2.4.1, AS/NZS1170.2:2002)

$C_{fig} = C_{p,e} K_a K_c K_l K_p$ (Clause5.2, AS/NZS1170.2:2002: external pressure)

$C_{fig} = C_{p,i} K_c$ (Clause5.2, AS/NZS1170.2:2002: internal pressure)

$C_{p,i} = 0.2$ (Table 5.1(A), AS/NZS1170.2:2002)

$C_{p,e} = -1.0$ (average) (Table5.3(A)-(C), AS/NZS1170.2:2002: α≥15°, h/d≥1)

$K_a = 0.8$ (Table 5.4, AS/NZS1170.2:2002: tributary area≥100m^2)

$K_c = 0.95$ (Table 5.5, AS/NZS1170.2:2002: case (d))

$K_l = 1.0$ (Clause 5.4.4, AS/NZS1170.2:2002)

$K_p = 1.0$ (Clause 5.4.5, AS/NZS1170.2:2002)

$\therefore C_{fig} = C_{p,e} K_a K_c K_l K_p - C_{p,i} K_c = -1 \times 0.8 \times 0.95 \times 1 \times 1 - 0.2 \times 0.95 = -0.95$

$C_{dyn} = 1.0$ (Clause 6.1(a), AS/NZS1170.2:2002)

$\therefore p = (0.5\rho_{air})(V_{des,\theta})^2 C_{fig} C_{dyn} = 0.5 \times 1.2 \times 37.4^2 \times (-0.95) \times 1 = -797.3$ (Pa) uplift pressure

1.2 Calculation of uplift wind pressure on roof for region B

Site wind speed

$V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t)$

$V_R = 57$ m/s (Table 3.1, AS/NZS1170.2:2002: 500 years return, region B)

$M_d = 0.95$ (Clause 3.3.2(a), AS/NZS1170.2:2002)

$M_{z,cat} = 0.83$ (Table 4.1(A), AS/NZS1170.2:2002: Terrain category 3, h≤10m)

$M_s = 1.0$ (Clause 4.3, AS/NZS1170.2:2002)

$M_t = 1.0$ (Clause 4.4, AS/NZS1170.2:2002: outside local topographic zone)

$$\therefore V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t) = 57 \times 0.95 \times 0.83 \times 1 \times 1 = 45 \text{ (m/s)}$$

Design wind speed

$V_{des,\theta} = V_{sit,\beta}$ =45 m/s (Clause2.3, AS/NZS1170.2:2002: any wind direction)

Design wind pressure

$p = (0.5\rho_{air})(V_{des,\theta})^2 C_{fig} C_{dyn}$ (Clause2.4.1, AS/NZS1170.2:2002)

$\rho_{air} = 1.2$ kg/m^3 (Clause2.4.1, AS/NZS1170.2:2002)

$C_{fig} = C_{p,e} K_a K_c K_l K_p$ (Clause5.2, AS/NZS1170.2:2002: external pressure)

$C_{fig} = C_{p,i} K_c$ (Clause5.2, AS/NZS1170.2:2002: internal pressure)

$C_{p,i} = 0.2$ (Table 5.1(A), AS/NZS1170.2:2002)

$C_{p,e} = -1.0$ (average) (Table5.3(A)-(C), AS/NZS1170.2:2002: α≥15°, h/d≥1)

$K_a = 0.8$ (Table 5.4, AS/NZS1170.2:2002: tributary area≥100m^2)

$K_c = 0.95$ (Table 5.5, AS/NZS1170.2:2002: case (d))

$K_l = 1.0$ (Clause 5.4.4, AS/NZS1170.2:2002)

$K_p = 1.0$ (Clause 5.4.5, AS/NZS1170.2:2002)

$$\therefore C_{fig} = C_{p,e} K_a K_c K_l K_p - C_{p,i} K_c = -1 \times 0.8 \times 0.95 \times 1 \times 1 - 0.2 \times 0.95 = -0.95$$

$C_{dyn} = 1.0$ (Clause 6.1(a), AS/NZS1170.2:2002)

$\therefore p = (0.5\rho_{air})(V_{des,\theta})^2 C_{fig} C_{dyn} = 0.5 \times 1.2 \times 45^2 \times (-0.95) \times 1 = -1154.3$ (Pa) uplift pressure

1.3 Calculation of uplift wind pressure on roof for region C

Site wind speed

$V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t)$

V_R=Fc 66=1.05×66=69.3 m/s (Table 3.1&Clause 3.4, AS/NZS1170.2:2002: 500 years return, region C)

$M_d = 0.95$ (Clause 3.3.2(a), AS/NZS1170.2:2002)

$M_{z,cat} = 0.89$ (Table 4.1(B), AS/NZS1170.2:2002: Terrain category 3, h≤10m)

$M_s = 1.0$ (Clause 4.3, AS/NZS1170.2:2002)

$M_t = 1.0$ (Clause 4.4, AS/NZS1170.2:2002: outside local topographic zone)

$\therefore V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t) = 69.3 \times 0.95 \times 0.89 \times 1 \times 1 = 58.6$ (m/s)

Design wind speed

$V_{des,\theta} = V_{sit,\beta}$ =58.6 m/s (Clause2.3, AS/NZS1170.2:2002: any wind direction)

Design wind pressure

$p = (0.5\rho_{air})(V_{des,\theta})^2 C_{fig} C_{dyn}$ (Clause2.4.1, AS/NZS1170.2:2002)

$\rho_{air} = 1.2$ kg/m^3 (Clause2.4.1, AS/NZS1170.2:2002)

$C_{fig} = C_{p,e} K_a K_c K_l K_p$ (Clause5.2, AS/NZS1170.2:2002: external pressure)

$C_{fig} = C_{p,i} K_c$ (Clause5.2, AS/NZS1170.2:2002: internal pressure)

$C_{p,e} = -1.0$ (average) (Table5.3(A)-(C), AS/NZS1170.2:2002: α≥15°, h/d≥1)

$C_{p,i} = C_{pe} = 1.0$ (Table 5.1(B), AS/NZS1170.2:2002)

$K_a = 0.8$ (Table 5.4, AS/NZS1170.2:2002: tributary area≥100m^2)

$K_c = 0.95$ (Table 5.5, AS/NZS1170.2:2002: case (d))

$K_c \geq 0.8 / K_a = 1$ (Clause 5.4.3, AS/NZS1170.2:2002)

$K_l = 1.0$ (Clause 5.4.4, AS/NZS1170.2:2002)

$K_p = 1.0$ (Clause 5.4.5, AS/NZS1170.2:2002)

$\therefore C_{fig} = C_{p,e} K_a K_c K_l K_p - C_{p,i} K_c = -1 \times 0.8 \times 1 \times 1 \times 1 - 1 \times 1 = -1.8$

$C_{dyn} = 1.0$ (Clause 6.1(a), AS/NZS1170.2:2002)

$\therefore p = (0.5\rho_{air})(V_{des,\theta})^2 C_{fig} C_{dyn} = 0.5 \times 1.2 \times 58.6^2 \times (-1.8) \times 1 = -3708.7$ (Pa)uplift pressure

1.4 Uplifting calculations

Uplifting stability check: $W_u < 0.9G$, where W_u is the ultimate wind action and G is the dead load action (Clause 4.2.1, AS/NZS1170.0:2002)

Self-weight of normal roofs

Table 1.1 Dead load of roofs

Roof type	Roof mass allowed (kPa)
Sheet roof	0.25
Sheet roof and ceiling	0.40
Tile roof	0.75
Tile roof and ceiling	0.90

Tie down anchor capacity

The tie down load capacity is calculated in Table 1.2 that is based on the roof tie down anchor tests conducted in Brahma Lodge Plant SA on 6 Dec 2002 by Peter Zwaans and Ben Ross.

Therefore, the design tensile capacity of each tie down in the rib of the wall that is anchored in accordance with the test setup is 2.8kN.

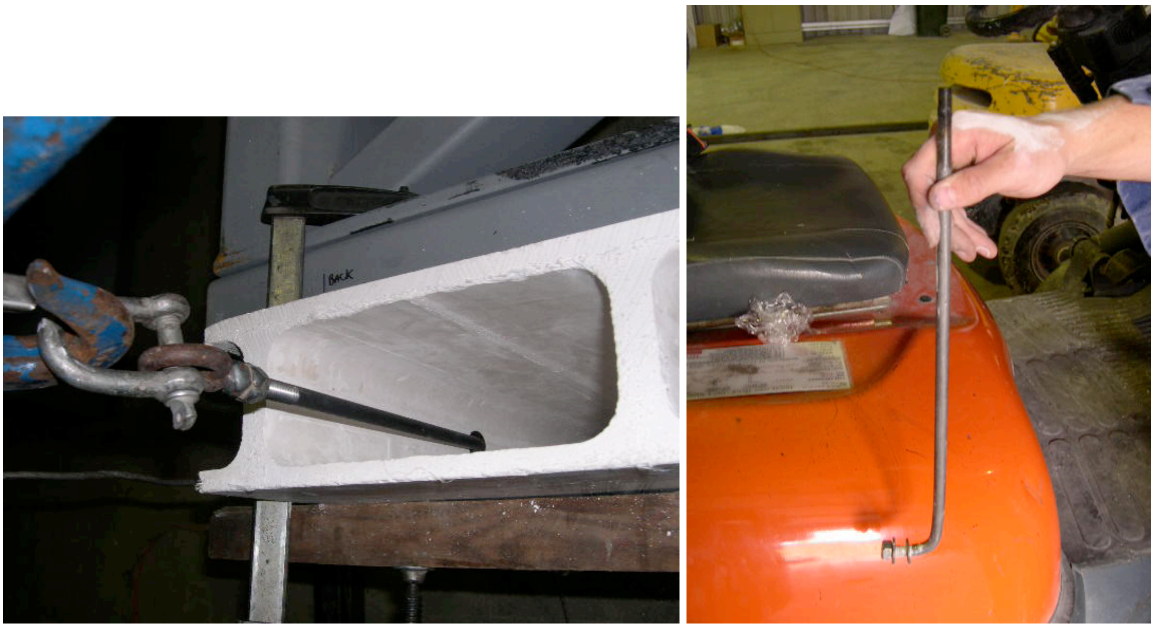

Fig.1.1 Tie down tests

Table 1.2 Tie down test results and design capacity

Specimen	Load capacity per tie (kN)
1	4.4
2	3.0
3	3.1
4	3.2
5	3.5
6	4.2
7	4.1
11	2.9
12	3.3
13	3.5
14	3.6
15	3.9
16	4.7
17	3.8
18	2.8
19	3.4
20	4.7
21	3.2
22	4.3
23	4.2
24	4.0
Average m_L	3.7
Coefficient of variation	0.08233
Variation reduction factor k	1.065
Maximum capacity $R=m_L/k$	3.5
Design capacity $R_d=R*\phi$	2.8

Capping C channel moment capacity

When 92mm×45mm×2.1mm C channel is used to support the roof trusses. The maximum bending capacity of the effective section is calculated to be 0.4kNm (see design calculations provided by Rondo SA).

Capping timber moment capacity

For simplicity, the same tie down details provided for C channel will be used for roof trusses supported on timber top plate. Therefore, the minimum bending moment capacity of the top timber plate shall be 0.4kNm.

1.5 Uplifting calculations for region A

The net uplifting pressure on roof is calculated in Section 1.1 to be 0.8kPa. The net uplifting force at the wall support is calculated by

Net uplifting load at support= (uplifting wind force/0.9-self-weight of roof)*truss span/2

The results are listed in Table 1.3. Based on the net uplifting force given in Table1.3 and the top C channel moment capacity of M_u=0.4kNm, the maximum spacing between ties can be calculated with

Tie down spacing = Sqrt(11* M_u /net uplifting load at support)

The results of the spacing are given in Table 1.4. Considering the void spacing inside the wall, only 0.75m, 1m, 1.25m and 1.5m spacings are used and the provided tie down spacings are listed in Table 1.5.

The net uplift force given in Table 1.3 may be resisted by the weight of the two end supporting walls, which is assumed to be 2.85m high that has a self-weight of q=1.0kN/m. The additional tie down force except the self-weight of the two end walls shall be provided by anchoring the ties or walls into the foundation. The additional force in each tie is calculated by

Tie force into foundation = (Net uplifting load at support-wall weight q)*tie down spacing

The calculated results of the additional tie down force into the foundation are given in Table 1.6.

The tie down design provision is summarised in Table 1.7. When ties into the foundation are needed, the tie made up of one R6 bar with yield strength of 7kN is sufficient for all the cases.

Table 1.3 Net uplift force at walls supporting roof trusses for region A

		Net uplifting force at the two supporting walls (kN/m)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-Weight (kg/m^2)	15	1.112833	1.854722	2.596611	3.3385	4.080389	4.822278	5.564167
	25	0.965833	1.609722	2.253611	2.8975	3.541389	4.185278	4.829167
	35	0.818833	1.364722	1.910611	2.4565	3.002389	3.548278	4.094167
	45	0.671833	1.119722	1.567611	2.0155	2.463389	2.911278	3.359167
	55	0.524833	0.874722	1.224611	1.5745	1.924389	2.274278	2.624167

	65	0.377833	0.629722	0.881611	1.1335	1.385389	1.637278	1.889167
	75	0.230833	0.384722	0.538611	0.6925	0.846389	1.000278	1.154167
	90	0.010333	0.017222	0.024111	0.031	0.037889	0.044778	0.051667

Table 1.4 Tie down spacing required for region A

		Tie down spacing required at bending strength of 1.2mm C channel (m)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-weight (kg/m^2)	15	1.988434	1.540235	1.301736	1.148023	1.038426	0.955213	0.889255
	25	2.134397	1.653297	1.397291	1.232295	1.114653	1.025331	0.954531
	35	2.318081	1.795578	1.51754	1.338345	1.210578	1.11357	1.036677
	45	2.559149	1.982308	1.675356	1.477525	1.336472	1.229375	1.144486
	55	2.895447	2.242804	1.895515	1.671687	1.512098	1.390928	1.294883
	65	3.412528	2.643333	2.234024	1.970224	1.782135	1.639325	1.526129
	75	4.365933	3.381837	2.858174	2.520673	2.280034	2.097326	1.952505
	90	NA	NA	NA	NA	NA	NA	NA

Table 1.5 Tie down spacing provided for region A

		Tie down spacing provided (m)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-weight (kg/m^2)	15	1.5	1.5	1.25	1	1	0.75	0.75
	25	1.5	1.5	1.25	1	1	1	0.75
	35	1.5	1.5	1.5	1.25	1	1	1
	45	1.5	1.5	1.5	1.25	1.25	1	1
	55	1.5	1.5	1.5	1.5	1.5	1.25	1.25
	65	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	75	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	90	NA	NA	NA	NA	NA	NA	NA

Table 1.6 Tie down force at each tie in addition to wall self-weight for region A

		Tie down force at each tie (kN)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-weight (kg/m^2)	15	0.16925	1.282083	1.995764	2.3385	3.080389	2.866708	3.423125
	25		0.914583	1.567014	1.8975	2.541389	3.185278	2.871875
	35		0.547083	1.365917	1.820625	2.002389	2.548278	3.094167

	45		0.179583	0.851417	1.269375	1.829236	1.911278	2.359167
	55			0.336917	0.86175	1.386583	1.592847	2.030208
	65				0.20025	0.578083	0.955917	1.33375
	75						0.000417	0.23125
	90							

Table 1.7 Tie down design provision for region A

		Tie down provision						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-weight (kg/m^2)	15	FT@1.5	FT@1.5	FT@1.25	FT@1	FT@1	FT@0.75	FT@0.75
	25	WT@1.5	FT@1.5	FT@1.25	FT@1	FT@1	FT@1	FT@0.75
	35	WT@1.5	FT@1.5	FT@1.5	FT@1.25	FT@1	FT@1	FT@1
	45	WT@1.5	FT@1.5	FT@1.5	FT@1.25	FT@1.25	FT@1	FT@1
	55	WT@1.5	WT@1.5	FT@1.5	FT@1.5	FT@1.5	FT@1.25	FT@1.25
	65	WT@1.5	WT@1.5	WT@1.5	FT@1.5	FT@1.5	FT@1.5	FT@1.5
	75	WT@1.5	WT@1.5	WT@1.5	WT@1.5	WT@1.5	FT@1.5	FT@1.5
	90	NA	NA	NA	NA	NA	NA	NA

Note of Table 1.7:

- NA denotes no tie is provided
- FT denotes Foundation Tie that shall be anchored into foundation
- WT denotes Wall Tie that is tied into the ribs of the wall
- The number follows @ denotes the spacing of ties
- Details of tie are provided in typical details

1.6 Uplifting calculations for region B

In region B, the design net uplift wind pressure is calculated in Section 1.2 to be 1.15kPa. The tie down calculations are similar to that in Section 1.5 and summarised in Tables 1.8 to 1.12.

Table 1.8 Net uplift force at walls supporting roof trusses for region B

		Net uplifting force at the two supporting walls (kN/m)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-weight (kg/m^2)	15	1.696167	2.826944	3.957722	5.0885	6.219278	7.350056	8.480833
	25	1.549167	2.581944	3.614722	4.6475	5.680278	6.713056	7.745833
	35	1.402167	2.336944	3.271722	4.2065	5.141278	6.076056	7.010833

	45	1.255167	2.091944	2.928722	3.7655	4.602278	5.439056	6.275833
	55	1.108167	1.846944	2.585722	3.3245	4.063278	4.802056	5.540833
	65	0.961167	1.601944	2.242722	2.8835	3.524278	4.165056	4.805833
	75	0.814167	1.356944	1.899722	2.4425	2.985278	3.528056	4.070833
	90	0.593667	0.989444	1.385222	1.781	2.176778	2.572556	2.968333

Table 1.9 Tie down spacing required for region B

		Tie down spacing required at bending strength of 1.2mm C channel (m)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-weight (kg/m^2)	15	1.610616	1.247578	1.054396	0.92989	0.841117	0.773715	0.720289
	25	1.6853	1.305428	1.103288	0.973009	0.880119	0.809592	0.753689
	35	1.77144	1.372152	1.15968	1.022742	0.925105	0.850972	0.792212
	45	1.872301	1.450278	1.225709	1.080973	0.977777	0.899424	0.837318
	55	1.992617	1.543474	1.304474	1.150438	1.04061	0.957222	0.891125
	65	2.139572	1.657306	1.400679	1.235283	1.117355	1.027817	0.956846
	75	2.324715	1.800716	1.521883	1.342175	1.214043	1.116757	1.039644
	90	2.722419	2.108777	1.782242	1.571789	1.421737	1.307808	1.217503

Table 1.10 Tie down spacing provided for region B

		Tie down spacing provided (m)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-weight (kg/m^2)	15	1.5	1	1	0.75	0.75	0.75	0.5
	25	1.5	1.25	1	0.75	0.75	0.75	0.75
	35	1.5	1.25	1	1	0.75	0.75	0.75
	45	1.5	1.25	1	1	0.75	0.75	0.75
	55	1.5	1.5	1.25	1	1	0.75	0.75
	65	1.5	1.5	1.25	1	1	1	0.75
	75	1.5	1.5	1.5	1.25	1	1	1
	90	1.5	1.5	1.5	1.5	1.25	1.25	1

Table 1.11 Tie down force at each tie in addition to wall self-weight for region B

		Tie down force at each tie (kN)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-	15	1.04425	1.826944	2.957722	3.066375	3.914458	4.762542	3.740417

weight	25	0.82375	1.977431	2.614722	2.735625	3.510208	4.284792	5.059375
(kg/m^2)	35	0.60325	1.671181	2.271722	3.2065	3.105958	3.807042	4.508125
	45	0.38275	1.364931	1.928722	2.7655	2.701708	3.329292	3.956875
	55	0.16225	1.270417	1.982153	2.3245	3.063278	2.851542	3.405625
	65		0.902917	1.553403	1.8835	2.524278	3.165056	2.854375
	75		0.535417	1.349583	1.803125	1.985278	2.528056	3.070833
	90			0.577833	1.1715	1.470972	1.965694	1.968333

Table 1.12 Tie down design provision for region B

		Tie down provision						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-	15	FT@1.5	FT@1	FT@1	FT@0.75	FT@0.75	FT@0.75	FT@0.5
weight	25	FT@1.5	FT@1.25	FT@1	FT@0.75	FT@0.75	FT@0.75	FT@0.75
(kg/m^2)	35	FT@1.5	FT@1.25	FT@1	FT@1	FT@0.75	FT@0.75	FT@0.75
	45	FT@1.5	FT@1.25	FT@1	FT@1	FT@0.75	FT@0.75	FT@0.75
	55	FT@1.5	FT@1.5	FT@1.25	FT@1	FT@1	FT@0.75	FT@0.75
	65	WT@1.5	FT@1.5	FT@1.25	FT@1	FT@1	FT@1	FT@0.75
	75	WT@1.5	FT@1.5	FT@1.5	FT@1.25	FT@1	FT@1	FT@1
	90	WT@1.5	WT@1.5	FT@1.5	FT@1.5	FT@1.25	FT@1.25	FT@1

1.7 Uplifting calculations for region C

In region c, the design net uplift wind pressure is calculated in Section 1.3 to be 3.7kPa. The tie down calculations are similar to that in Section 1.5 and summarised in Tables 1.13 to 1.17. An R6 tie may not be sufficient in some cases. Therefore, R10 tie is provided with yield strength of 19.6kN for trusses at 9m and 15m spans.

Table 1.13 Net uplift force at walls supporting roof trusses for region C

		Net uplifting force at the two supporting walls (kN/m)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-	15	5.946167	9.910278	13.87439	17.8385	21.80261	25.76672	29.73083
weight	25	5.799167	9.665278	13.53139	17.3975	21.26361	25.12972	28.99583
(kg/m^2)	35	5.652167	9.420278	13.18839	16.9565	20.72461	24.49272	28.26083
	45	5.505167	9.175278	12.84539	16.5155	20.18561	23.85572	27.52583
	55	5.358167	8.930278	12.50239	16.0745	19.64661	23.21872	26.79083
	65	5.211167	8.685278	12.15939	15.6335	19.10761	22.58172	26.05583

	75	5.064167	8.440278	11.81639	15.1925	18.56861	21.94472	25.32083
	90	4.843667	8.072778	11.30189	14.531	17.76011	20.98922	24.21833

Table 1.14 Tie down spacing required for region C

		Tie down spacing required at bending strength of 1.2mm C channel (m)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-weight (kg/m^2)	15	0.860217	0.666321	0.563144	0.496646	0.449233	0.413235	0.384701
	25	0.871051	0.674713	0.570237	0.502901	0.454892	0.418439	0.389546
	35	0.882305	0.683431	0.577604	0.509399	0.460769	0.423846	0.394579
	45	0.894007	0.692495	0.585265	0.516155	0.46688	0.429467	0.399812
	55	0.906188	0.70193	0.593239	0.523188	0.473241	0.435319	0.40526
	65	0.91888	0.711761	0.601548	0.530516	0.479869	0.441416	0.410936
	75	0.932121	0.722018	0.610217	0.53816	0.486784	0.447776	0.416857
	90	0.953102	0.738269	0.623951	0.550273	0.497741	0.457855	0.42624

Table 1.15 Tie down spacing provided for region C

		Tie down spacing provided (m)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-weight (kg/m^2)	15	0.75	0.5	0.5	0.5	0.25	0.25	0.25
	25	0.75	0.5	0.5	0.5	0.25	0.25	0.25
	35	0.75	0.5	0.5	0.5	0.25	0.25	0.25
	45	0.75	0.5	0.5	0.5	0.25	0.25	0.25
	55	0.75	0.5	0.5	0.5	0.25	0.25	0.25
	65	0.75	0.5	0.5	0.5	0.25	0.25	0.25
	75	0.75	0.5	0.5	0.5	0.25	0.25	0.25
	90	0.75	0.5	0.5	0.5	0.25	0.25	0.25

Table 1.16 Tie down force at each tie for region C

		Tie down force at each tie (kN)						
Roof truss span (m)		3	5	7	9	11	13	15
Roof self-weight (kg/m^2)	15	4.459625	4.955139	6.937194	8.91925	5.450653	6.441681	7.432708
	25	4.349375	4.832639	6.765694	8.69875	5.315903	6.282431	7.248958
	35	4.239125	4.710139	6.594194	8.47825	5.181153	6.123181	7.065208
	45	4.128875	4.587639	6.422694	8.25775	5.046403	5.963931	6.881458

	55	4.018625	4.465139	6.251194	8.03725	4.911653	5.804681	6.697708
	65	3.908375	4.342639	6.079694	7.81675	4.776903	5.645431	6.513958
	75	3.798125	4.220139	5.908194	7.59625	4.642153	5.486181	6.330208
	90	3.63275	4.036389	5.650944	7.2655	4.440028	5.247306	6.054583

Table 1.17 Tie down design provision for region C

Roof truss span (m)		3	5~9	11~15
Roof self-weight (kg/m^2)	15~90	FT@0.75	FT@0.5	FT@0.25

2. VERTICAL POINT LOAD BEARING CAPACITY OF WALLS TO ROOF STRUCTURES

Structural tests were undertaken in University of South Australia on the vertical load bearing capacity of unfilled full height wall panels under point loads simulating load from roof trusses. The point loads were applied through 50mm wide steel bearers at 600mm centres via a 120mm wide×45mm thick timber plate and a 150PFC capping channel to the top of the wall. Local crushing of plaster on top of the wall under the steel bearers was observed in the tests. Therefore, the tests results provided an indication of local crushing strength under compression. The test results are summarised in Table 2.1.

Table 2.1 Local bearing strength of unfilled wall panel

Specimen	1	2	3	4	5	6	Mean value m_L	Coeff. of varia-tion	Varia. reduc-tion factor k	Max. value $R=m_L/k$	Design value $R_d=R^*\phi$
Failure point load with timber top plate and PFC capping beam (kN)	51.5	43	40	71	72.5	95.5	62.3	0.343	1.86	33.5	20.1 (ϕ =0.6)
Failure point load with timber plate only (kN)	44						44		2 assumed	22	13.2 (ϕ =0.6)

For a normal tile roof supported by trusses of 15m span spaced at 1.2m c/c, the maximum ultimate load at wall support is given by

$$P_u = (1.25 \times 0.9 + 1.5 \times 0.25) \times 1.2 \times 15 / 2 = 13.5 \text{ (kN)}$$

which is just ok for walls with timber top plate.

3. OUT-OF-PLANE BENDING OF WALL

The Rapidwall® panels have significant flexural capacity to resist gust wind pressure on the wall. The full height (2.85m) wall supported on its top and bottom can resist a maximum ultimate pressure of 1.6kPa without cracking, which is sufficient for cottage construction in regions A and B. For region C, the unfilled panel is not sufficient to resist the wind pressure and the calculations are given below.

3.1 Calculation of wind pressure on wall for region C

Site wind speed

$V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t)$

V_R=Fc 66=1.05×66=69.3 m/s (Table 3.1&Clause 3.4, AS/NZS1170.2:2002: 500 years return, region C)

$M_d = 0.95$ (Clause 3.3.2(a), AS/NZS1170.2:2002)

$M_{z,cat} = 0.89$ (Table 4.1(B), AS/NZS1170.2:2002: Terrain category 3, h≤10m)

$M_s = 1.0$ (Clause 4.3, AS/NZS1170.2:2002)

$M_t = 1.0$ (Clause 4.4, AS/NZS1170.2:2002: outside local topographic zone)

$\therefore V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t) = 69.3 \times 0.95 \times 0.89 \times 1 \times 1 = 58.6$ (m/s)

Design wind speed

$V_{des,\theta} = V_{sit,\beta}$ =58.6 m/s (Clause2.3, AS/NZS1170.2:2002: any wind direction)

Design wind pressure

$p = (0.5\rho_{air})(V_{des,\theta})^2 C_{fig} C_{dyn}$ (Clause2.4.1, AS/NZS1170.2:2002)

$\rho_{air} = 1.2$ kg/m^3 (Clause2.4.1, AS/NZS1170.2:2002)

$C_{fig} = C_{p,e} K_a K_c K_l K_p$ (Clause5.2, AS/NZS1170.2:2002: external pressure)

$C_{fig} = C_{p,i} K_c$ (Clause5.2, AS/NZS1170.2:2002: internal pressure)

For windward walls:

$C_{p,e} = 0.8$ (Table5.2(A), AS/NZS1170.2:2002: h≤25m)

$C_{p,i} = -0.3$ (Table 5.1(A)&(B), AS/NZS1170.2:2002)

For leeward walls:

$C_{p,e} = -0.5$ (Table5.2(B), AS/NZS1170.2:2002)

$C_{p,i} = 0.6$ (Table 5.1(A), AS/NZS1170.2:2002)

For side walls:

$C_{p,e} = -0.65$ (Table5.2(C), AS/NZS1170.2:2002)

$C_{p,i} = C_{p.e} = 0.65$ (Table 5.1(B), AS/NZS1170.2:2002)

Worst case: side walls

$K_a = 0.8$ (Table 5.4, AS/NZS1170.2:2002: tributary area≥100m^2)

$K_c = 0.95$ (Table 5.5, AS/NZS1170.2:2002: case (d))

$K_c \geq 0.8 / K_a = 1$ (Clause 5.4.3, AS/NZS1170.2:2002)

$K_l = 1.0$ (Clause 5.4.4, AS/NZS1170.2:2002)

$K_p = 1.0$ (Clause 5.4.5, AS/NZS1170.2:2002)

$\therefore C_{fig} = C_{p,e}K_aK_cK_lK_p - C_{p,i}K_c = -0.65 \times 0.8 \times 1 \times 1 \times 1 - 0.65 \times 1 = -1.17$

$C_{dyn} = 1.0$ (Clause 6.1(a), AS/NZS1170.2:2002)

$\therefore p = (0.5\rho_{air})(V_{des,\theta})^2 C_{fig} C_{dyn} = 0.5 \times 1.2 \times 58.6^2 \times (-1.17) \times 1 = -2410.6$ (Pa)

3.2 Out-of-plane bending of walls

In theory the wall shall be designed as compression member with flexural moment. However, the vertical load is very small and it will disappear under uplift wind pressure. As the compression resistance of the wall cross-section is much bigger than the tensile resistance, the worst-case scenario is when the wall is under uplift tension and in the mean time subjected to bending. For simplicity, the uplift tension and bending are calculated separately and the reinforcement needed is then added together to provide the design.

Flexural design:

Design moment: $M^* = \frac{1}{8}wL^2 = \frac{1}{8} \times 2.4 \times 2.85^2$ =2.4 (kNm/m)

There are four RC cores in one meter width of wall, therefore, the design moment for each RC core is *M**=0.6kNm.

$$\frac{M^*}{bd^2} = \phi f'_c\, q(1 - q/1.7)$$

ϕ=0.8

b=230 (mm)

d=94/2=47 (mm)

f'_c=25 (N/mm^2)

Therefore,

q=0.061

$$q = \frac{A_{st} f_{sy}}{bdf'_c} = 0.061$$

f_{sy}=250 (N/mm^2)

$\therefore A_{st} = 66$ (mm^2), $k_u = \frac{A_{st} f_{sy}}{0.85\gamma bdf'_c} = 0.08<0.4$, OK.

From Table 1.16, the maximum tie down load is 8.9kN, which need a rebar area of $\frac{8.9\times10^3}{0.8\times250}$=44.5 (mm^2). Therefore, total A_{st} =66+44.5=110.5. Provide R12, $A_{st,prov} = 113\text{mm}^2$.

Conclusion: R12 in every cavity to resist both out-of-plane bending and uplifting resistance.

4. LINTELS

Concrete filled lintels shall be designed by a qualified professional engineer in accordance with the design principals provided in the first part of this manual. For lintels without reinforced concrete infill, it is generally not advisable for the lintels to take substantial long term loading due to its creep nature. However, deep lintels with aspect ratio (depth of lintel divided by span length) of not less than 1.0 can be used as structural member to support dead and live load in accordance with Table 3.1. The total ultimate load shall be calculated in accordance with Eq.3.1:

$$F = 1.25G + 1.5Q \qquad (3.1)$$

where G and Q are the total dead and live load on the span, respectively.

Table 3.1 Design table for deep unfilled lintels

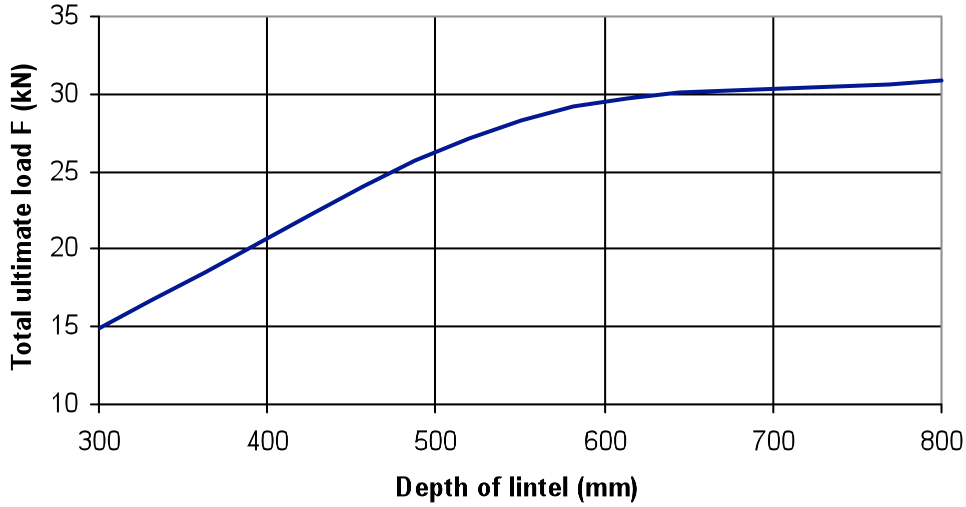

Table 3.1 was calculated from the maximum moment capacity given in the first part of design manual. For example, the design moment capacity for a 300mm deep lintel is found from Fig.9.2 of the design manual to be ϕM=1.12kNm. The total load on a beam is given by $P = 8 \times (\phi M) / L \quad (\because \phi M = \frac{1}{8} PL)$. As ϕM is a constant, the shorter the span, the bigger the total load *P*. However, the lintel tests from which the moment capacity was derived do not cover all the span length. Therefore, the shortest span length tested was used to calculate *P*. In the case of 300mm lintel, the shortest span is 0.6m which gives P=8×1.12/0.6=14.9 kNm. The spans shorter than 300mm for 300mm deep lintels shear strength may govern and any *P* values greater than 14.9kN that is calculated from moment equation will not be justifiable without further tests.

If the load on the lintel is point load, local crushing check shall be undertaken in accordance with Table 2.1.