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Timbercrete Pty Ltd

Design of Partially Reinforced Timbercrete for In-Plane Racking Loads

This report has been prepared on behalf of **ELECTRONIC BLUEPRINT** by:

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Design of Partially Reinforced Timbercrete for In-Plane Racking Loads

Scope

This report considers the design of partially reinforced Timbercrete masonry for in-plane racking loads due to wind or earthquake, with particular reference to cyclical load tests reported in Ensis Test Report TE06-001.

The report is in three parts:

Part 1- Behaviour Similar to Conventional Partially Reinforced Masonry
Part 2- Design Based on Tests Reported in Ensis Test Report TE06-001
Part 3- Design Based on AS 3700-2001

Background

All building design must comply with the relevant State Building Regulations, which are set out in the BCA (Building Code of Australia) Volumes 1 and 2. The BCA defines the performance requirements, generally in very broad terms, and the means of compliance through:

- Deemed-to-Satisfy Provisions, which may include:
  - Acceptable Construction Manuals (e.g. nominated Australian Standards, AS 3700)
  - Acceptable Construction Practice (e.g. forms of construction reproduced in the BCA itself

- Alternative Solutions (e.g. Designs based on test results and engineering principles).

Each of these paths to compliance has equal status under the BCA. This report deals with the "Alternative Solutions" path.

In all but the rarest of cases, structural engineers will design using the Deemed-to-Satisfy Solution by:

- Determining loads using AS/NZS 1170 (Parts 0, 1, 2, 3) and AS 1170.4; and,
- Determining resistance using AS 4100, AS 3600, AS 3700, AS 1720, AS 1684, AS 2870 etc.

However, structural engineers are often asked to incorporate new products into their designs. The performance of such products is justified by test because they are outside the scope of the various Australian Standards specified by the BCA as Deemed-to-Satisfy. The requirements are set out in BCA Vol 1 Clauses A0.8, A0.9 and A0.10, and BCA Vol 2 Clauses 1.0.8, 1.0.9 and 1.0.10. When dealing with structural components, the following procedure is appropriate.

1. Determine the performance requirements using BCA Vol 1 Part B1 or BCA Vol 2 Part B.

2. Determine the loads using AS/NZS 1170 (Parts 0, 1, 2 3) and AS 1170.4.

3. Determine the relevant properties by test e.g. stability, strength, deflection etc.

4. Use AS/NZS 1170.0 :2002 Appendix B to assess the test data and obtain design values. Although this Appendix is informative, and therefore not formally part of the BCA Deemed-to-Satisfy path, it provides a reliable component of "Expert Judgement".

5. Use the design values so derived in the context of the normal structural design standards. It should be noted that such design values include allowance for reductions normally associated with the capacity reduction factors, φ.

Part 1- Behaviour Similar to Conventional Partially Reinforced Masonry

	Test 1	Test 2
Length of walls		
L	= 1,800 m	880 m
Height of simply supported wall		
H	= 2.400 m	
H/L	= 2.400 / 1.800	2.400 / 0.880
	= 1.33	2.73
	< 2.3	> 2.3
	Treat as a shear wall	Treat as a beam

The reported behaviour was similar to partially reinforced masonry walls reported previously, upon which AS 3700 Clause 8.6 is based.

Part 2(a)- Design Based on Tests Reported in Ensis Test Report TE06-001
1,800 mm long x 2,400 mm high wall

Number of samples tested
N = 1

Minimum failure load
P_{min} = 25.0 kN

Reduction factor
k_1 = 1.79
Based on AS 1170.0:2002 Table B1, for one sample tested and assumed coefficient of variation of 15%

Design capacity
φP = P_{min} / k_1
= 25.0 / 1.79
= 14.0 kN

Note:
If the number of identical tests were increased to (say) five, assuming that the minimum value was 95% of the single test, it could be reasonably expected that this value would increase to:
φP_{pred} = 25.0 x 0.95 / 1.46
= 16.3 kN.m

Part 2(b)- Design Based on Tests Reported in Ensis Test Report TE06-001
880 mm long x 2,400 mm high wall

Number of samples tested
N = 1

Minimum failure load
P_{min} = 12.3 kN

Reduction factor
k_1 = 1.79
Based on AS 1170.0:2002 Table B1, for one sample tested and assumed coefficient of variation of 15%

Design capacity
φP = P_{min} / k_1
= 12.3 / 1.79
= 6.9 kN

Note:
If the number of identical tests were increased to (say) five, assuming that the minimum value was 95% of the single test, it could be reasonably expected that this value would increase to:
φP_{pred} = 12.3 x 0.95 / 1.46
= 8.00 kN.m

Part 3(a) - Design Based on AS 3700-2001 - Wall 1 – 1,800 mm long x 2,100 mm high

Number of courses in wall
$N_{courses}$ = 14

Height of each course
H_{course} = 171 mm

Height of wall
H = 2,400 mm

Length of wall
L = 1,800mm

Wall thickness
D = 150 mm

No of horizontal grouted beams or strips
$N_{horiz\ grouted}$ = 6

Length from end of wall to anchorage steel
l' = 100 mm

Masonry unit characteristic unconfined compressive strength
f'_{uc} = 3.0 MPa

Block type factor
k_m = 1.4
Units are solid

Equivalent brickwork strength
$f'_{mb} = k_m\ (f'_{uc})^{0.5}$
$= 1.4\ (3.0)^{0.5}$
= 2.42 MPa

Mortar joint height
h_j = 21.5 mm

Masonry unit height
h_b = 171.5 mm

Ratio of block to joint thickness
h_b/h_j = 171.5 / 21.5
= 8.9

Block height factor
k_h = 1.04

Characteristic masonry strength
$f'_m = k_h\ f'_{mb}$
= 1.04 x 2.42
= 2.53 MPa

Capacity reduction factor
φ = 0.75

Steel tensile yield strength

f_{sy} = 500 MPa

Number of vertical bars in each core
N_{vert} = 1

Number of vertical anchorage bars
$N_{vert\ anch}$ = 1

Vertical reinforcement (N12, M12, N16, M16, N20, M20)
Rv = M12 (Threaded 12 mm bar)

Horizontal reinforcement is strip steel

Strip width
b_s = 25 mm

Strip thickness
t_s = 0.9 mm

Area of one horizontal strip
$A_{sh\ 1}$ $= b_s\ t_s$
$= 25 \times 0.9$
$= 22.5\ \text{mm}^2$

Total area of horizontal bars or strip
A_{sh} $= N_{horiz\ grouted}\ A_{sh\ 1}$
$= 6 \times 22.5$
$= 135\ \text{mm}^2$

Number of horizontal bars or strips crossing the crack
$N_{eff\ v}$ = rounddown ($N_{horiz\ grouted}$ L / H)
= 6 x 1,800 / 2,100
= 5

Effective area horizontal bars or strip crossing crack (Amdt 1)
$A_{sh\ eff}$ $= A_{sh}\ N_{eff} / N_{horiz\ grouted}$
$= 135 \times 5 / 6$
$= 113\ \text{mm}^2$

Area of one vertical bar
$A_{sv\ 1}$ $= 84.3\ \text{mm}^2$ Threaded M12 bar

Number of vertical bars or strips crossing the crack
$N_{eff\ v}$ = 2

Total area vertical bars (effective area of vertical bars)
A_s $= N_{eff\ v}\ A_{sv\ 1}$
$= 2 \times 84.3$
$= 169\ \text{mm}^2$

Effective depth for overturning
d = L – 2 l'
= 1,800 –100
= 1,700 mm

Design area of main tensile reinforcement
A_{sd} $= \min [\ 0.29\ (1.3\ f'_m)\ b\ d / f_{sy}\ ,\ A_{st}\ ]$ AS 3700 Clause 8.5

= min [(0.29 x 1.3 x 2.53 x 150 x 1,700 / 500) , 84.3]
= min [486 , 84.3]
= 84.3 mm^2

Proportion grouted (if not completely filled)
p = 1.00

Height ratio
H/L = 2,400 / 1,800
= 1.33
< 2.3

Masonry confined strength
f_{vr} = (1.50 – 0.5 H / L)
= (1.50 – [0.5 x 2,400 / 1,800])
= 0.833 MPa

Average density of wall
γ_{wall} = 1,000 kg/m3

Self weight
P_{self} = γ_{wall} L H D
= 1,000 x 9.81 x 1,800 x 2,100 x 150 / 1,000,000,000,000
= 5.6 kN

Externally applied vertical load
P_{ext} = 0.0 kN

Total vertical load
P_v = P_{self} + P_{ext} 12.7 kN
= 5.6 + 0.0
= 5.6

Shear area
A_d = L D
= 2,100 x 150 /1,000,000
= 315,000 mm^2

Toe crushing factor
k_{sw} = [1 – P_v / (A_d f'_m)]
= [1 – 5.6 x 1,000 / (315,000 x 2.53)]
= 0.993

Wall shear capacity
φV_{mas} = φ (f_{vr} A_d + 0.8 f_{sy} A_s)
= 0.75 [(0.833 x 315,000) + (0.8 x 500 x 113)] / 1,000
= 224 kN

Overturning shear capacity
$\varphi V_{overturn}$= φ [k_{sw} P_v L / 2 + f_{sy} A_{sv}(L – 2l')] / H
= 0.75 [(0.993 x 5.6 x 1,800 / 2) + (500 x 84.3 {1,800 – (2 x 100)}) / 1,000] / 2,400
= 0.75 x 30.2
= 22.6 kN

Comments
The calculated ultimate capacity is 30.2 kN. This is in excess of the of the test result of 25 kN, which is dictated by the capacity of the testing equipment.

Part 3(b) - Design Based on AS 3700-2001 - Wall 2 – 880 mm long x 2,100 mm high

Number of courses in wall
$N_{courses}$ = 14

Height of each course
H_{course} = 171 mm

Height of wall
H = 2,400 mm

Length of wall
L = 880mm

Wall thickness
D = 150 mm

No of horizontal grouted beams or strips
$N_{horiz\ grouted}$ = 6

Length from end of wall to anchorage steel
l' = 100 mm

Masonry unit characteristic unconfined compressive strength
f'_{uc} = 3.0 MPa

Block type factor
k_m = 1.4
Units are solid

Equivalent brickwork strength
$f'_{mb} = k_m\ (f'_{uc})^{0.5}$
$= 1.4\ (3.0)^{0.5}$
= 2.42 MPa

Mortar joint height
h_j = 21.5 mm

Masonry unit height
h_b = 171.5 mm

Ratio of block to joint thickness
h_b/h_j = 171.5 / 21.5
= 8.9

Block height factor
k_h = 1.04

Characteristic masonry strength
$f'_m = k_h\ f'_{mb}$
= 1.04 x 2.42
= 2.53 MPa

Capacity reduction factor
φ = 0.75

Steel tensile yield strength

f_{sy} = 500 MPa

Number of vertical bars in each core
N_{vert} = 1

Number of vertical anchorage bars
$N_{vert\ anch}$ = 1

Vertical reinforcement (N12, M12, N16, M16, N20, M20)
Rv = M12 (Threaded 12 mm bar)

Horizontal reinforcement is strip steel

Strip width
b_s = 25 mm

Strip thickness
t_s = 0.9 mm

Area of one horizontal strip
$A_{sh\ 1}$ = $b_s\ t_s$
= 25 x 0.9
= 22.5 mm^2

Total area of horizontal bars or strip
A_{sh} = $N_{horiz\ grouted}\ A_{sh\ 1}$
= 6 x 22.5
= 135 mm^2

Number of horizontal bars or strips crossing the crack
$N_{eff\ v}$ = rounddown ($N_{horiz\ grouted}$ L / H)
= 6 x 1,800 / 2,100
= 5

Effective area horizontal bars or strip crossing crack (Amdt 1)
$A_{sh\ eff}$ = $A_{sh}\ N_{eff}$ / $N_{horiz\ grouted}$
= 135 x 5 / 6
= 113 mm^2

Area of one vertical bar
$A_{sv\ 1}$ = 84.3 mm^2 Threaded M12 bar

Number of vertical bars or strips crossing the crack
$N_{eff\ v}$ = 2

Total area vertical bars (effective area of vertical bars)
A_s = $N_{eff\ v}\ A_{sv\ 1}$
= 2 x 84.3
= 169 mm^2

Effective depth for overturning
d = L – 2 l'
= 880 –100
= 780 mm

Design area of main tensile reinforcement
A_{sd} = min [0.29 (1.3 f'_m) b d / f_{sy} , A_{st}] AS 3700 Clause 8.5

$= \min [(0.29 \times 1.3 \times 2.53 \times 150 \times 780 / 500) \quad , \quad 84.3]$
$= \min [223 \ , \ 84.3]$
$= 84.3 \text{ mm}^2$

Proportion grouted (if not completely filled)
$p = 1.00$

Height ratio
$H/L = 2{,}400 / 880$
$= 2.72$
> 2.3

Masonry confined strength
$f_{vm} = 0.35$ MPa

Average density of wall
$\gamma_{wall} = 1{,}000$ kg/m3

Self weight
$P_{self} = \gamma_{wall} \, L \, H \, D$
$= 1{,}000 \times 9.81 \times 1{,}800 \times 2{,}100 \times 150 / 1{,}000{,}000{,}000{,}000$
$= 5.6$ kN

Externally applied vertical load
$P_{ext} = 0.0$ kN

Total vertical load
$P_v = P_{self} + P_{ext}$ 12.7 kN
$= 5.6 + 0.0$
$= 5.6$

Shear area
$A_d = L \, D$
$= 2{,}100 \times 150 / 1{,}000{,}000$
$= 315{,}000 \text{ mm}^2$

Toe crushing factor
$k_{sw} = [1 - P_v / (A_d \, f'_m)]$
$= [1 - 5.6 \times 1{,}000 / (315{,}000 \times 2.53)]$
$= 0.993$

Beam shear capacity
$\varphi V_{beam} = \varphi \, (f_{vm} \, b_w \, d + f_{vs} \, A_{st} + f_{sy} \, A_{sv} \, d / s)$
$= 0.75 \, [(0.35 \times 150 \times 780) + (17.5 \times 84.3) + (500 \times 22.5 \times 780 / 343)] / 1{,}000$
$= 0.75 \, (41.0 + 1.5 + 25.6)$
$= 51.0$ kN

Overturning shear capacity
$\varphi V_{overturn} = \varphi \, [k_{sw} \, P_v \, L / 2 + f_{sy} \, A_{sv}(L - 2l')] / H$
$= 0.75 \, [(0.993 \times 5.6 \times 1{,}800 / 2) + (500 \times 84.3 \, \{1{,}800 - (2 \times 100)\}) / 1{,}000] / 2{,}400$
$= 0.75 \times 13.0$
$= 9.7$ kN

Comments
The calculated ultimate capacity is 13.0 kN. This is in excess of the of the test result of 12.3 kN, which was the highest load tested. Failure was apparently by bearing failure of the timber under the washers at the threaded rods.

Conclusions

1. The test is consistent with the expected behaviour of partially reinforced masonry whe subjected to in-plane racking loads.

2. When the rest result is analyzed in accordance with AS/NZS 1170.0 Appendix B Clause B3,design shear capacities are of 14.0 kN and 6.9 kN are derived.

3. Further identical tests would enable this value to be increased. Typically, if the number of identical tests were increased to (say) five, it could be reasonably expected that these values would increase to approximately to 16.3 and 8.0 kN.m.

4. Using AS 3700-2001, the design shear capacities are 22.6 kN and 9.7 kN.

5. Therefore, it is recommended that further tests are not necessary and that design should be based on AS 3700-2001. In Australia, it is not necessary to design houses for cyclic action due to earthquake load (except in areas of exceptionally high hazard). Therefore the above-mentioned shear capacities will be used principally for design to resist wind loads.

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