



STUDY REPORT

No. 125 (2004)

Bottom plate anchors under NZS 3604:1999

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Technology from the Research for Industry Fund.

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Preface

This report was prepared following research into bottom plate anchors used with timber wall framing on concrete floor slabs, as provided for by the New Zealand Timber Framed Buildings Standard, NZS 3604:1999.

Acknowledgments

This work was funded by the Foundation for Research, Science and Technology from the Research for Industry Fund.

The agreement of proprietary fastener manufacturers to include their test results in this report is gratefully acknowledged.

Note

This report is intended for Structural Engineers, Standards committee members, Manufacturers, Building Officials.

BOTTOM PLATE ANCHORS UNDER NZS 3604

REFERENCE

Shelton, R.H., 2004. Bottom plate anchors under NZS 3604:1999. BRANZ Study Report No. SR125(2004). Building Research Association of New Zealand, Judgeford, Porirua.

ABSTRACT

An assessment was made of the load demands on bottom plate anchors under both extreme events (windstorm or earthquake), and in-service loads (general robustness). This showed that the provisions currently in NZS 3604 are generally satisfactory, but somewhat too high for internal walls.

An experimental study found that the common NZ practice of forming slab edges with concrete masonry header blocks is likely to result in an inferior performance by both generic and proprietary bottom plate anchors. This issue needs to be addressed by Standards New Zealand, through the NZS 3604 committee.

DISCLAIMER

Note that the results obtained for the proprietary fasteners tested pertain only to those fasteners, and no claim is made that these results are generally representative of fasteners of their type. The fact that some proprietary fasteners have not been tested implies no comment on their performance.

BRANZ Ltd therefore takes no responsibility for reliance on the results for the generic fasteners.

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1. INTRODUCTION

The revision of NZS 3604, “Timber framed buildings” in 1999 (SNZ, 1999), included for the first time, criteria to be met for proprietary, post-fixed anchors for fixing the bottom plates of timber wall framing to concrete floor slabs.

Prior to this revision there were no provisions for proprietary anchors, although cast in place bolts and dowels were specified, along with shot fired fasteners. The additions to NZS 3604 were made in response to numerous requests from the building industry for guidance in what was a very grey area. This was all the more critical because changes in New Zealand’s building practices since NZS 3604 was first published in 1978 have now reached the stage where over 80% of new houses are being constructed on concrete floor slabs.

2. REQUIREMENTS OF NZS 3604:1999

2.1 Proprietary fixings

The Standard drafting committee’s response to the requests for change was to prescribe a method to determine the strength of a fixing (in clause 2.4.7), and include pass/fail criteria (in clause 7.5.12).

The relevant clauses are reproduced verbatim below (with the permission of Standards New Zealand):

2.4.7 Connector capacity and durability

The capacity of a connector or fixing shall be calculated in accordance with the following equation:

$$R = \phi \times Q_k \times n \times k$$

where: R = connector capacity in kN

ϕ = capacity reduction factor from NZS 3603

Q_k = characteristic value of the seismic strength obtained by test in accordance with BRANZ Evaluation Method EMI, or AS/NZS 2699:Part 2 as appropriate

n = number of tested specimens making up the complete joint

k = modification factors from NZS 3603(section 4) as appropriate to the specific application.

In addition to verifying the load carrying capacity, the manufacturer shall also demonstrate, to the satisfaction of the Territorial Authority, that the fixings shall conform to the durability requirements of Clause B2 of the NZBC.

(It is outside the scope of this paper to comment on durability, and it would be premature to do so until the revisions to NZBC Clause B2 are finalised.)

7.5.12 Fixing of timber

7.5.12.1

Framing timbers shall be fixed to slab-on-ground floors as required by 6.11.9 or proprietary fasteners may be used in accordance with 7.5.12.2.

7.5.12.2

Proprietary fasteners complying with 7.5.12.3 or 7.5.12.4 may be used to fix bottom plates of walls to slab-on-ground floors, other than wall bracing elements, provided they are within 150 mm of each end of the plate and at not more than 900 mm centres elsewhere.

7.5.12.3

For internal walls, proprietary fasteners securing bottom plates to concrete floors shall have a minimum horizontal capacity when tested in accordance with 2.4.7 as follows:

- (a) *In the plane of the wall 4 kN;*
- (b) *Out of the plane of the wall 3 kN.*

7.5.12.4

For external walls, proprietary fasteners securing bottom plates to concrete floors shall have a minimum capacity when tested in accordance with 2.4.7 as follows:

- (a) *Horizontal loads in the plane of the wall 5 kN;*
- (b) *Horizontal loads out of plane of the wall 4 kN;*
- (c) *Vertical loads in axial tension of the fastener 8 kN.*

2.2 Cast in place and generic fixings

The 1990 version of NZS 3604 (SANZ, 1990) provided for cast in place anchors by referring to the provisions for fixing wall plates to foundation beams, although strictly speaking this is not the same application, especially with regard to durability. There were two alternatives given:

- a. A bent M12 bolt (with washer and nut) spaced at 1.4 metres along the plate.
- b. An R10 dowel (bent over the plate and stapled) spaced at 900 mm.

These are illustrated in Figure 1.

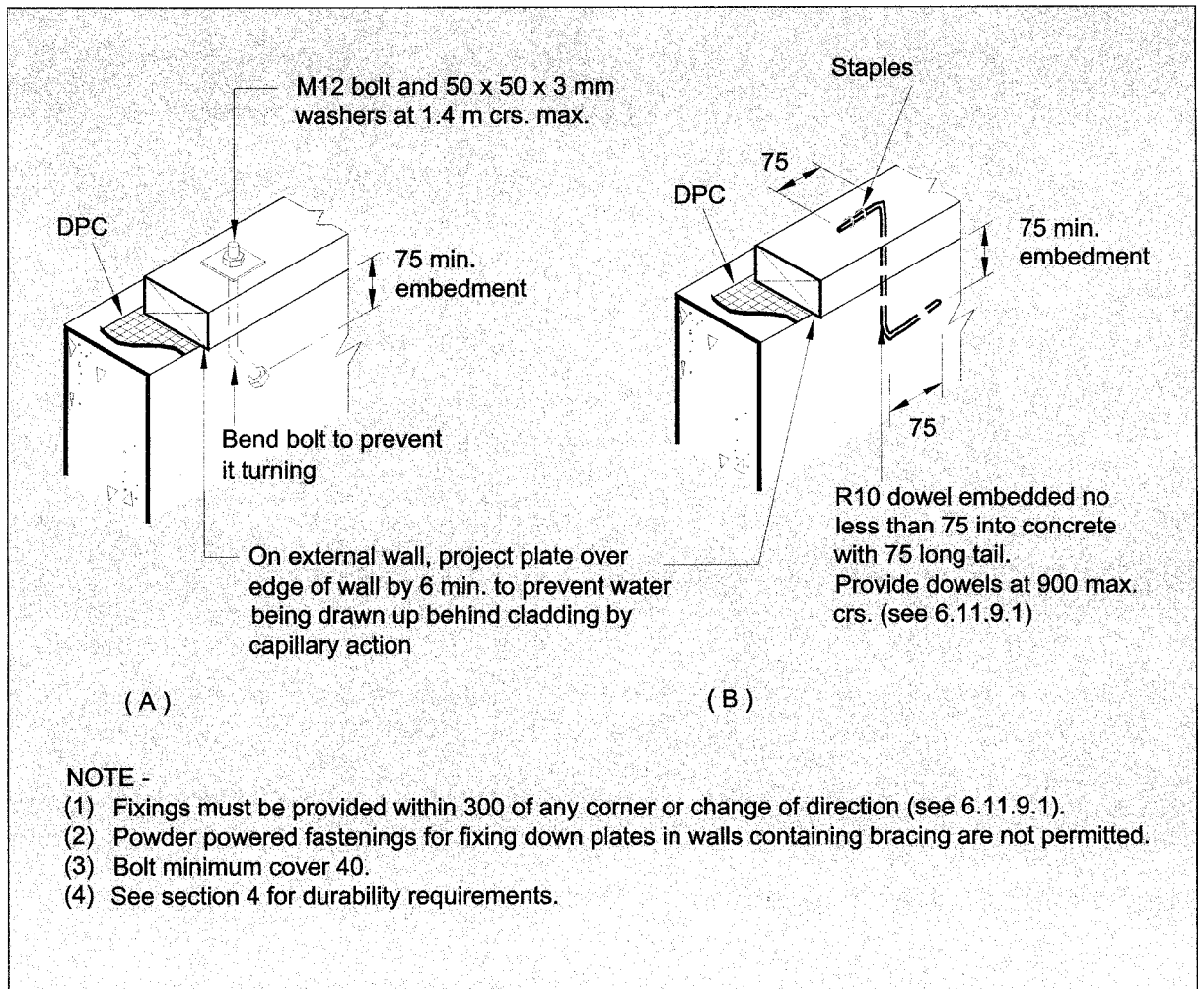


Figure 6.16 – Fixing of wall plates to foundation walls (see 6.11.9)

Figure 1: Figure 6.16 from NZS 3604:1999. (reproduced with permission)

With the revision of the standard in 1999, these provisions essentially remained unchanged.

Shot fired fasteners were allowed by the 1990 version of NZS 3604 for internal walls (non-bracing walls) when spaced at 900 mm. For the 1999 version, these were deemed to be proprietary fasteners, thus they were not specifically mentioned. However they may be included within the provisions of clause 7.5.12.2 (see above).

2.3 Bracing walls

Until the 1990 version of the document, NZS 3604 provided details for generic wall bracing elements, which were to be fixed to a concrete slab by a cast in M12 bolt at each end. This provision was removed with the 1999 version, as virtually all wall bracing elements constructed in NZ were proprietary systems. Anchorage of these to the slab was considered to be an essential part of the bracing system, and thus was covered by testing to establish a bracing rating for the system as a whole.

3. THE APPROACH USED IN OTHER COUNTRIES

3.1 Australia

The equivalent Australian Standard is AS 1684.2 “Residential timber framed construction. Non-cyclonic areas” (SA, 1999).

For buildings in Wind Classifications N1 and N2 (roughly equivalent to Low and Medium zones of NZS 3604) only “Nominal” fixings are required, which are described as “*One 75 mm masonry nail (hand driven at slab edge), screw, or bolt, at 1200 mm centres*”. For buildings in Classifications N3 and N4 (roughly equivalent to High and Very High), the building designer is required to calculate the uplift demand (a simplified method and a more detailed alternative are given), and match that to uplift capacities provided for various fixings. For seasoned radiata pine, these are:

Fixing	Uplift capacity (kN)
Hardened, hammered or fired masonry nails	1.0
Cast in M12 bolt	20
Chemical, expansion or fired proprietary fastener	Refer to manufacturer’s specifications

There is no guidance on how appropriate data is to be derived for publication in the manufacturer’s specifications.

3.2 USA

The International Building Code 2000 (ICC, 2000) requires that timber framed walls (foundation plates or sills) are fixed to concrete foundations with ½ inch (12.7 mm) bolts spaced at 6 feet (1.83 m). Bolts are to be embedded 7 inches (178 mm) and must be fitted with nuts and washers. For bracing walls, the spacing is to be reduced to 4 feet (1.22 m) for structures over 2 stories in height. Alternative approved anchors may also be used.

The ICBO Evaluation Service gives very detailed guidance on acceptance criteria for masonry anchors, adhesive anchors, expansion anchors, and pre-drilled fasteners (screw anchors). (ICBO, 1995; ICBO, 2001; ICBO, 2002; ICBO, 2002a respectively). These are intended to be used for approvals under the “Alternative materials” provisions of IBC 2000. The acceptance criteria evaluate the performance of anchors under a variety of service conditions under static and cyclic loading based on the test methods of ASTM E448 (ASTM, 1996).

ASTM E448 provides for static, seismic, fatigue, and shock loading in the tension and shear directions for post fixed and cast-in-place anchors. However the loading mechanism comprises a steel loading plate 12 mm thick for an M12 fixing, and does not incorporate a timber plate, so the method is not particularly relevant for timber bottom plate fixings, where the timber member could be the weakest part of the assemblage.

4. ASSESSMENT OF LOAD DEMAND

The limits on overall building size in the scope of NZS 3604:1999 allow an assessment to be made of the load demand on bottom plate fixings. The procedure used in this project was based on the Loading Standard, NZS 4203 (SNZ, 1992), and the parameters used are shown in Table 1.

Table 1: Parameters used for wind load demand estimate.

Wind zone	Speed v_z (m/s)	Pressure q_z (kPa)	Weights of construction (kPa)		
			Roof (in roof plane)	Light Heavy	0.15 0.5
L	32	0.61			
M	37	0.82			
H	44	1.16	Ceiling		0.15
VH	50	1.50	Wall	Medium	0.3
			Eave width	750	mm
			Wall height	2.4	m
Pressure coefficients	Wind direction	Parallel to ridge	Perpendicular to ridge	Pressure factors	
				Local (k_l)	Area (k_a)
C_{pe} (upwind slope)		-0.9	-0.7	Local (k_l)	1.0
C_{pe} (downwind slope)		-0.9	-0.3	Area (k_a)	1.0
C_{peave}		-0.65	+0.7	Porosity (k_p)	1.0
C_{pi}		+0.2	0.0		

Note: (-) indicates pressure acting away from the surface and (+) towards the surface.

4.1 Uplift load (tension)

Wind uplift forces acting on the roof and upper levels of the building (less the relevant dead loads) will be transferred to the resisting mass of the floor slab and foundation through the bottom plate anchor bolts acting in tension. An estimate of these loads (using the parameters described above) is presented in Table 2.

The estimate was done for a single storey building only, because the absence of the suspended floor load makes this configuration the most critical for uplift. Thus the wall height was taken as 2.4 m (for the dead load estimate), and height to eaves for this building would typically be 3.0 m. This introduces conservatism for sites in the more exposed situations because NZS 3604:1999 assumes a reference height, z , of 8 m when assigning site wind speeds. Variation of wind speed with height above ground, as quantified by Table 5.4.3. of NZS 4203, indicates that this conservatism results in an overestimation of wind speed by around 15%, with a corresponding increase in pressure of 32%.

Pressure coefficients were chosen having regard to intrinsic geometric limits embodied in realistic buildings within the scope of NZS 3604. For wind parallel to the ridge, the reference height, h , was taken as 4 m (3 m eave, plus 10 degree pitch and 12m truss span). Thus the h/d limit of ≤ 0.5 implies a building greater than 8 m long, giving a C_{pe} of -0.9 for the first 4 m length. For this case, C_{pi} was taken as +0.2 as it is reasonable to expect that the whole interior space adjacent to the windward wall would be under positive pressure. The under-eave pressure coefficient of -0.65 was used (as for side walls). For wind perpendicular to the ridge, the reference height is 3 m, giving h/d of 0.25 with a 12 m truss and roof pitch of 10 degrees, leading to the coefficients tabulated. However for uplift on anchor bolts, this wind direction does not produce the greatest loads.

Table 2: Calculated net uplift load (kN/metre along wall)

(Note: Uplift load is shown with a –ve prefix)

Light roof		Roof slope (degrees)				
Wind zone	Loaded dimension	10	15	25	45	60
Low	3	-0.47	-0.46	-0.43	-0.27	0.02
	4	-0.88	-0.87	-0.82	-0.62	-0.25
	5	-1.28	-1.27	-1.21	-0.97	-0.52
	6	-1.69	-1.67	-1.61	-1.32	-0.79
Medium	3	-1.20	-1.19	-1.15	-0.99	-0.70
	4	-1.83	-1.82	-1.77	-1.57	-1.20
	5	-2.46	-2.44	-2.39	-2.15	-1.69
	6	-3.09	-3.07	-3.01	-2.73	-2.19
High	3	-2.38	-2.37	-2.34	-2.18	-1.88
	4	-3.39	-3.38	-3.33	-3.13	-2.76
	5	-4.39	-4.38	-4.33	-4.08	-3.63
	6	-5.40	-5.38	-5.32	-5.04	-4.50
Very high	3	-3.56	-3.55	-3.52	-3.36	-3.06
	3.5	-4.25	-4.24	-4.20	-4.02	-3.69
	4	-4.94	-4.93	-4.88	-4.69	-4.31
	5	-6.32	-6.30	-6.25	-6.01	-5.55
	6	-7.70	-7.68	-7.62	-7.33	-6.80

The maximum tabulated uplift load of 7.7 kN/m gives a tension load of 10.78 kN for anchor bolts spaced at 1.4 metres, and 6.93 kN for a 0.9 m spacing. The 6.93 kN compares with the criterion of 8 kN for proprietary anchors at 0.9 m prescribed in clause 7.5.12.4 of NZS 3604. Considering the overestimation of demand caused by the assumed wall height, as discussed above, it could be argued that the value of 8 kN in NZS 3604 is unnecessarily conservative, and perhaps a value of 7 kN would be more appropriate.

It may be argued that where an opening occurs in the wall under consideration, the uplift loads are accumulated along the lintel and concentrated at the base of the trimmer stud. Thus the tension load could be as high as 16.6 kN at the end of the maximum lintel span of 3.4 m permitted for a 6 m loaded dimension [Uplift = $7.7 \times \left(\frac{0.9}{2} + \frac{3.4}{2} \right) = 16.6$]. This indicates that the hold-down requirement of 7.5 kN in clause 8.6.1.8 of NZS 3604 could be severely non-conservative.

However, it should be noted that the demand calculated as above applies only to the end 4 metres of the building. Considering the likelihood of all the required circumstances occurring together in one building, the 16.6 kN is somewhat excessive. Perhaps, after considering the overestimation of height referred to above, a value of 12 kN for clause 8.6.1.8 would be appropriate for buildings in High and Very High zones.

4.2 Shear perpendicular to plate

Shear loading perpendicular to the bottom plate is generated by face loading on the wall from wind or earthquake actions. The same process described above was used to estimate these demands using the parameters of Table 1. The results are shown in Table 3.

Table 3: Calculated out-of-plane horizontal loads (kN/metre along wall)

Wind load (inwards)		Stud height (m)					
Wind zone		2.4	2.7	3	3.6	4.2	4.8
Low		0.74	0.83	0.92	1.11	1.29	1.47
Medium		0.99	1.11	1.23	1.48	1.72	1.97
High		1.39	1.57	1.74	2.09	2.44	2.79
Very high		1.80	2.03	2.25	+	+	+
Wind load (outwards)		Stud height (m)					
Wind zone		2.4	2.7	3	3.6	4.2	4.8
Low		0.63	0.71	0.78	0.94	1.10	1.25
Medium		0.84	0.94	1.05	1.26	1.47	1.68
High		1.18	1.33	1.48	1.78	2.07	2.37
Very high		1.53	1.72	1.91	+	+	+
Earthquake load		Stud height (m)					
Eq. zone	Wall weight	2.4	2.7	3	3.6	4.2	4.8
A	Light	0.12	0.13	0.14	0.17	0.20	0.23
	Medium	0.31	0.35	0.39	0.46	0.54	0.62
	Heavy	0.85	0.96	1.06	1.27	1.49	*
B	Light	0.10	0.11	0.12	0.14	0.17	0.19
	Medium	0.26	0.29	0.32	0.39	0.45	0.51
	Heavy	0.71	0.80	0.88	1.06	1.24	*
C	Light	0.06	0.07	0.07	0.09	0.10	0.12
	Medium	0.15	0.17	0.19	0.23	0.27	0.31
	Heavy	0.42	0.48	0.53	0.64	0.74	*
		+ Not included in Table 8.2 of NZS 3604					
		* Max. veneer height 4 metres					

The maximum tabulated shear load of 2.79 kN/m (2.37 kN/m for outwards loads) gives a shear load of 3.9 kN (3.32 kN outwards) for anchor bolts spaced at 1.4 metres, and 2.5 kN (2.13 kN outwards) for a 0.9 m spacing. The 2.5 kN compares with the criterion of 4 kN for proprietary anchors (external walls) spaced at 0.9 m, as prescribed in clause 7.5.12.4 of NZS 3604.

There will also be shear loading associated with intermittent events such as doors slamming and human impact, as well as long term loading caused by the reaction from fitments fixed to the wall, such as heavy shelving or appliances. These will be more likely to be associated with internal walls. Such impacts result in considerably lower loading on the bottom plate anchors than the uniform loading estimated above. Shelf or fitment loading is difficult to estimate, given the variety of situations possible. However an upper bound may be predicted based on the bending strength of a wall stud. The scenario is shown diagrammatically in Figure 2.

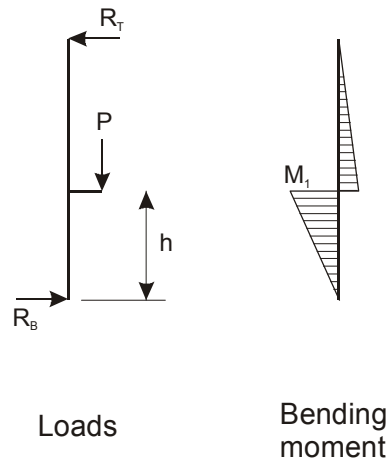


Figure 2: Shelf load on a wall.

From the figure: $R_B = \frac{M_1}{h}$.

Putting M_1 equal to the bending strength of a 100x50 stud calculated from NZS 3603 (SNZ, 1993), 0.91 kNm, and h at the smallest practical shelf height (500 mm), gives a maximum value of R_B of 1.82 kN per stud. For studs spaced at 600 mm and equally loaded, this equates to an out-of-plane shear load of 4.25 kN for anchor bolts spaced at 1.4 m, and 2.73 kN when spaced at 900 mm. These values are comparable to the NZS 3604 criteria of 3 kN for internal walls.

4.3 Shear parallel to plate

Apart from “designated” bracing walls, shear loads in the plane of the wall are very low. However many walls other than the designated brace walls do contribute to lateral load resistance of the buildings, and will therefore be subject to some in-plane loading. This is difficult to quantify, however a very crude estimate of such loads is included in the Appendix.

Additionally, friction between the plate and the floor plays a role in reducing the shear forces resisted by the anchor bolts. Considering all these factors, the criteria for anchor bolts quoted in NZS 3604 (at 5 kN for external walls, and 4 kN for internal walls) appear to be far too high, especially for the non-bracing walls being considered. It is suggested that a nominal value of 2 kN would be more than adequate to give a reasonable level of robustness for non bracing walls.

4.4 Bracing wall loads

Bottom plate anchor bolts at each end of bracing walls are subject to large pullout forces under the racking action of the bracing panel.

Simplistically, anchor bolt tension forces may be calculated using simple statics and published bracing ratings. (Figure 3)

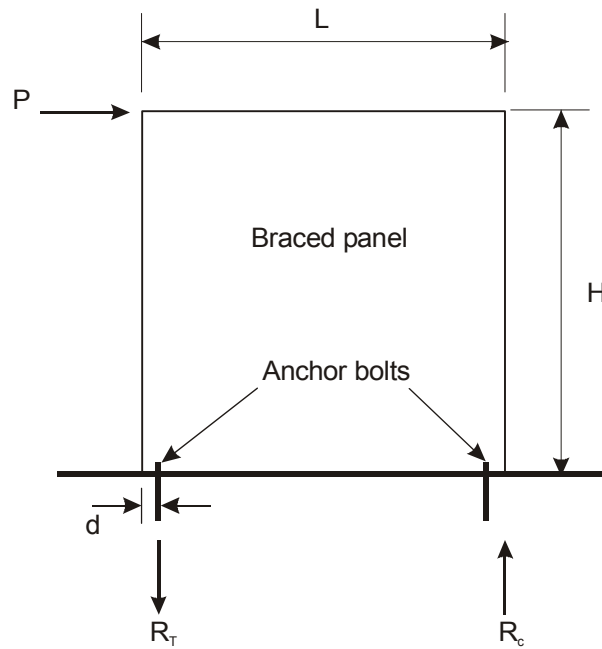


Figure 3: Statics of a braced panel.

For example, the BR6 system (Winstone Wallboards, 1999) shows a bracing rating of 150 bracing units per metre (BU/m) for a wall of minimum length of 1.2 m.

150 BU/m is equivalent to $150/20 = 7.5$ kN/m (SNZ, 1999),

$$\text{thus: } R_T = \frac{(P \times L) \times H}{(L - d)} = \frac{(7.5 \times 1.2) \times 2.4}{(1.2 - .09)} = 19.5 \text{ kN.}$$

However in real buildings the bracing walls are not built in isolation from the remainder of the walls. To simulate that continuity provided by adjoining walls, wall bracing tests are conducted with additional “P21” restraints at each end (Cooney and Collins 1978) This considerably reduces the anchor uplift forces calculated as above using simple statics. Anchor uplift forces are not normally recorded in standard racking tests, although BRANZ has made a few measurements in recent years. Thus the load demand on anchor bolts is not currently defined satisfactorily, and this was the main reason why anchors in braced walls were specifically excluded from the criteria of NZS 3604. Another reason is the wide variation in strengths of wall bracing elements, giving correspondingly wide variation in bolt load demands.

4.5 Summary of load demand

Based on the estimates as described in this section, the suggested revised strength criteria for bottom plate anchors may be summarised in the Table 4, with the current NZS 3604:1999 values provided for comparison: (Values have been rounded)

Table 4: Load demand summary

	Estimate as above (kN)	Current criteria (kN)	NZS 3604	Suggested criteria (kN)
Internal walls				
In-plane	2	4		2
Out-of-plane	2	3		2
External walls				
Shear out-of-plane (inwards)	2.5	5		3
Shear out-of-plane (outwards)	2.2	5		3
Shear in-plane	2	4		2
Tension	7	8		7
Trimmer studs				
Tension fixing (clause 8.6.1.8)	16.6	7.5		12 for H & VH zones 7.5 for L & M zones

5. EXPERIMENTAL PROGRAMME

The experimental programme to determine the strength of cast-in and post-drilled proprietary anchors can be considered in two phases:

- that work undertaken specifically for this project. The details of these tests are fully reported here, and all the information is in the public domain.
- that work undertaken for private clients on proprietary fasteners, and whose results are separately reported. However some of these results have been included here (where permitted by the client) to increase the relevance and application of this work.

5.1 Tests carried out at BRANZ under this project

The experimental programme was undertaken during September and October 2002 at BRANZ, Judgeford, Wellington.

Details of the fasteners tested are shown in Table 5.

Table 5: Description of products tested

Product Name	Anchor Type	Diameter	Length
Ramset Dynabolt DP16-110	Post drilled sleeve anchor	12 mm thread diameter (16 mm hole diameter)	106 mm
Ramset HD875 drive pin	Shot fired fastener	3.8 mm (head 8 mm) (washer 16 mm)	75 mm
M12 bolt (Hot dip galvanised)	Cast in anchor (NZS 3604)	12 mm	150 mm

Square washers where used, were 50x50x3 mm (hot dip galvanised), as specified by NZS 3604.

Photograph 1 shows the fasteners used for the tests.



Photograph 1. Fasteners tested (L-R: M12 bolt, sleeve anchor, drive pin).

5.1.1 Construction of slabs

To test the anchors, two series of concrete slabs were constructed, as detailed in Figure 4. One series had edges formed using concrete masonry “header blocks”, which is a common practice in New Zealand for domestic concrete floor slabs. The other series had edges formed by ordinary timber boxing. Both series of slabs were 190 mm deep. Concrete was delivered from a readymix plant with the requirement that it should have a 28 day compressive strength of 17.5 MPa, as stipulated by NZS 3604. Concrete test cylinders were crushed to confirm that strength. The slabs were left to cure for a minimum of 28 days before being moved for installation of anchors and testing.

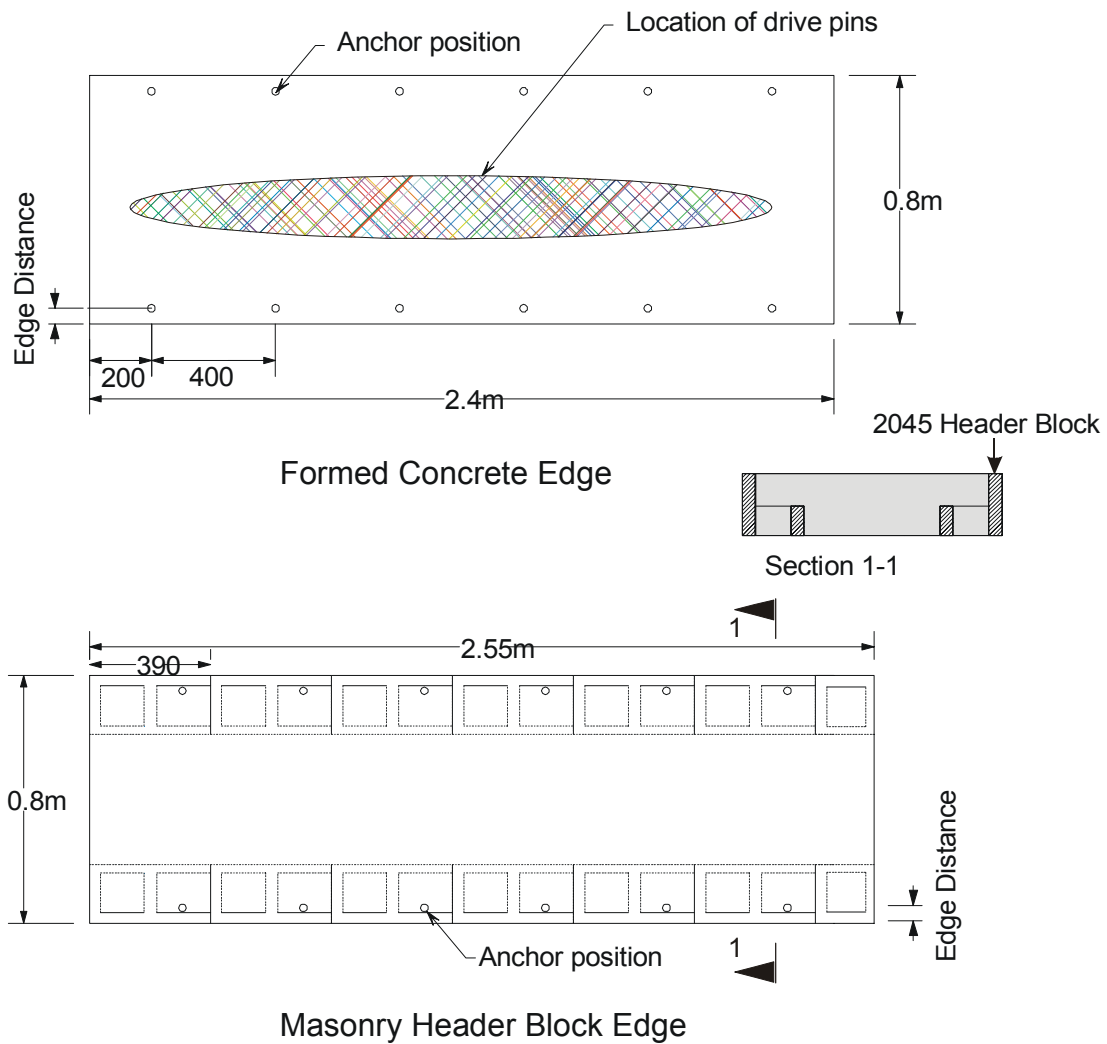


Figure 4: Test slab construction

5.1.2 Installation of anchors

The cast in M12 bolts were installed at the time of placing concrete. They were bent before installation to prevent turning when tightening the nut, and were set to an embedment depth of 75 mm, as stipulated by NZS 3604. The distance to the edge of the masonry header block slabs was approximately 35 mm to the centre of the anchor. This was governed by the thickness of the face shells of the header blocks, which were approximately 30 mm thick. Edge distance for the formed edge slabs was 50 mm.

The other fasteners were installed just prior to testing, in accordance with the manufacturer's instructions.

Sleeve anchors were installed at slab edge distances of 40 mm and 50 mm. An edge distance of 30 mm was also attempted, but this meant that the hole was right on the interface between masonry and concrete, and the softer concrete of the masonry caused the drill bit to wander towards the slab edge and break off the face shell. This edge distance was therefore abandoned. It was observed after testing that even with some of the bolts at 40 mm edge distance, the bit had wandered towards the edge, thus weakening the anchor system. This is noted in sub-section 5.1.5 "Observations". All holes were drilled with a rotary hammer drill. Nuts were tightened against the bottom plate to a torque of 50 Nm. In none of these instances was the indentation of the washer in the timber plate excessive.

Shot fired anchors were installed in the centre of the slab as shown on Figure 4. Pins were installed with and without washers, as noted in the test schedule.

For all anchors, the bottom plate was 90 x 45 kiln dried radiata pine No. 1 framing. A length of bituminous damp-proof course (DPC) was placed between the plate and the concrete surface on all of the shear test specimens. The location of the anchor in relation to the slab edge and plate is detailed in Figure 5. Note that because of the bottom plate overhang stipulated in NZS 3604, and required slab edge distance, most anchors are not on the centre of the plate. It should also be noted that to accommodate the 50 x 50 washer without intruding beyond the inside of the framing, the maximum edge distance is approximately 60 mm.

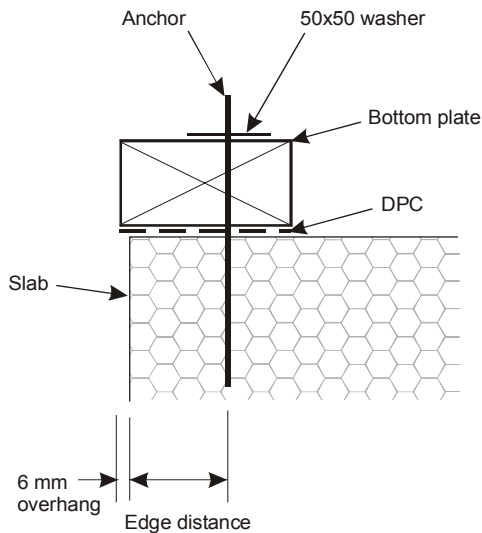
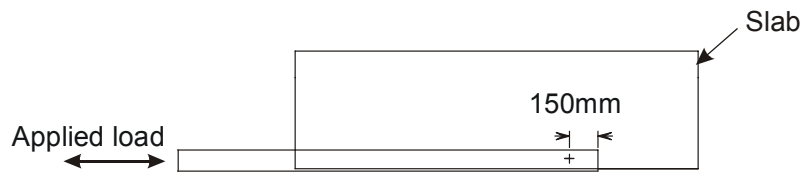


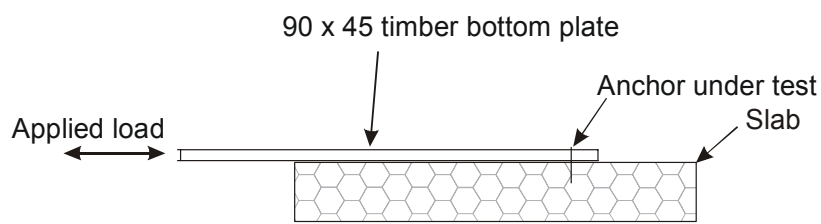
Figure 5: Installation of anchor.

5.1.3 Test setup and equipment

The test setups for the three loading directions (in-plane, out-of-plane, and tension) are shown in Figures 6, 7, and 8 respectively. For the out-of-plane shear tests, the load was applied to the timber plate by clamps centred 188 mm on either side of the anchor. The clamps were arranged to rotate in the horizontal direction to allow the plate to bend. For the tension tests, the load was applied via two steel load applicators centred 150 mm on either side of the anchor. The plate was cut locally to provide clearance for the applicators, as shown in Figure 8.

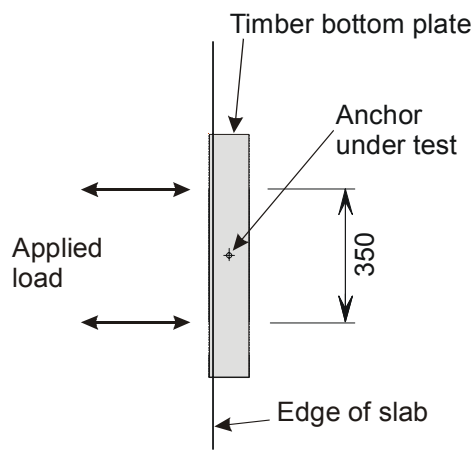


Plan View

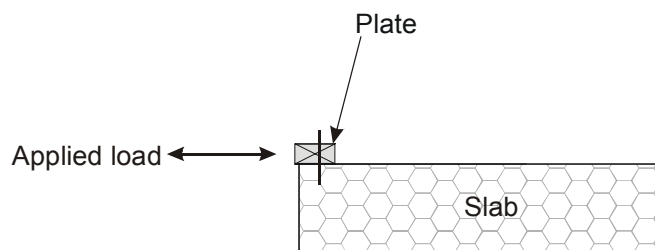


Side view

Figure 6: In-plane test set up.



Plan view



Side view

Figure 7: Out-of-plane test set up.

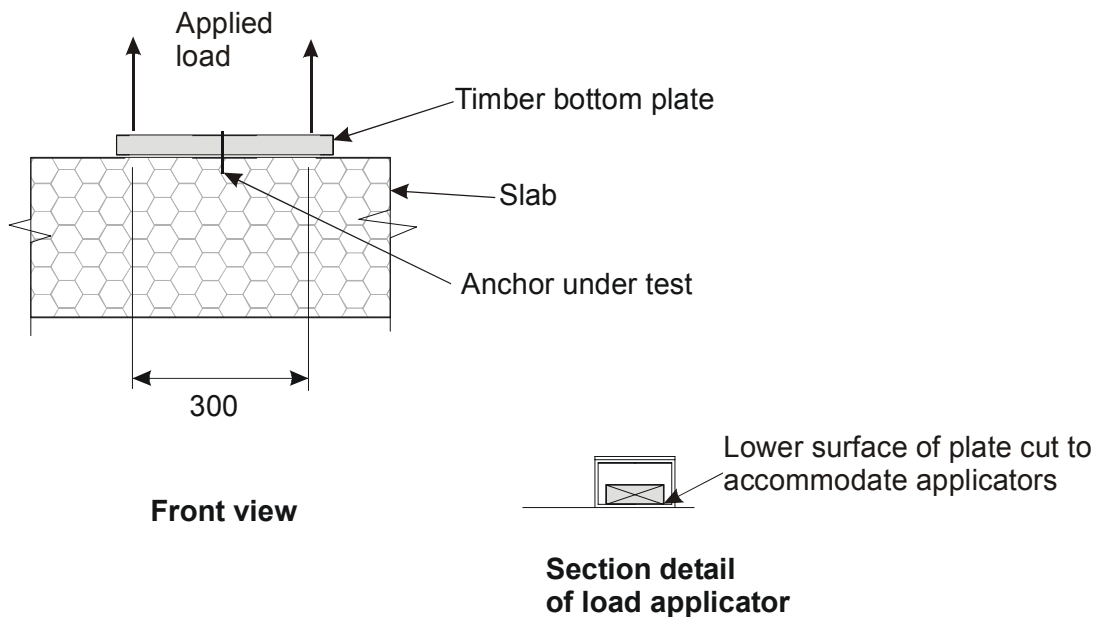


Figure 8: Tension test set up.

The slabs were laid on the laboratory strong floor, and arranged so they could be moved to allow each anchor in turn to be brought into position for testing, and then firmly restrained. Load was applied to the anchors with a 100 kN capacity closed loop hydraulic actuator and measured with a 50 kN load cell within Grade 1 accuracy (BS-EN-ISO 7500-1:1999). Displacement of the specimen was measured by the Linear Voltage Displacement Transducer within the actuator, reading to an accuracy of ± 0.05 mm. The test load and displacement measurements were recorded using a PC running a software program to record the data for subsequent analysis.

5.1.4 Test procedure

For all tests except the drive pins, the loading regime was earthquake cyclic, in accordance with BRANZ Evaluation Method EM1 (BRANZ, 1999). All tests were carried out under displacement control.

For the shear tests, the first specimen in the series for each configuration was cycled three times to each of the levels of load corresponding to ± 0.25 , ± 0.375 , ± 0.5 , ± 1.0 , ± 2.0 , ± 3.0 , etc. multiplied by the target load specified in NZS 3604. The remaining specimens in the series were then cycled to multiples of the displacement recorded at half the ultimate load achieved in the first specimen, as detailed in EM1.

For all tension tests, the loading regime involved cycling three times to each of $+0.25$, $+0.375$, $+0.5$, $+1.0$, $+2.0$, $+3.0$, etc. multiplied by the target load specified in NZS 3604.

For the drive pins the gravity loading regime of EM1 was used. This was because the drive pins are only intended for internal wall applications, where the load criteria are for general robustness only, and are not intended to be used for earthquake resistance.

5.1.5 Observations

Under the applied displacements, most anchors bent at the slab surface and embedded into the timber plate, accompanied by sliding of the plate and DPC on the concrete surface. The actual failure mechanisms can be classified into one or more of the following categories:

(the letters are the failure identifiers shown in the detailed test results)

- A. Masonry face shell fractured and broke off slab edge. This failure mode was frequently promoted by the drill bit wandering towards (or into) the masonry face shell during anchor installation. Photograph 2.
- B. Fracture and break out of formed concrete slab edge. Photograph 3.
- C. Bottom plate split and/or fractured. Photograph 4.
- D. Fastener pullout from the slab. Generally accompanied by only minor concrete disturbance. Photograph 5.
- E. Plate pulled over the fastener head. Photograph 6.



Photograph 2. Failure mode A



Photograph 3. Failure mode B



Photograph 4. Failure mode C



Photograph 5. Failure mode D



Photograph 6. Failure mode E (note, the lower 2 pins show failure mode D)

5.1.6 Results

5.1.6.1 Concrete Compressive Strength

The average measured compressive strength of the concrete cylinders, cast at the same time as the slabs, at the time of testing was 25 MPa.

5.1.6.2 Anchor Test Results

The peak third cycle load (or peak load in the case of the drive pins) and failure mode for each test specimen is listed in Tables 6 to 11.

As prescribed by EM1, the peak third cycle loads are defined as the maximum load resisted by the anchor, taken from an envelope curve covering the peak third cycle loads at each displacement increment. This is illustrated by the example in Figure 9.

Representative plots of load against displacement for each configuration are given in Figures 10 to 13 in the Appendix.

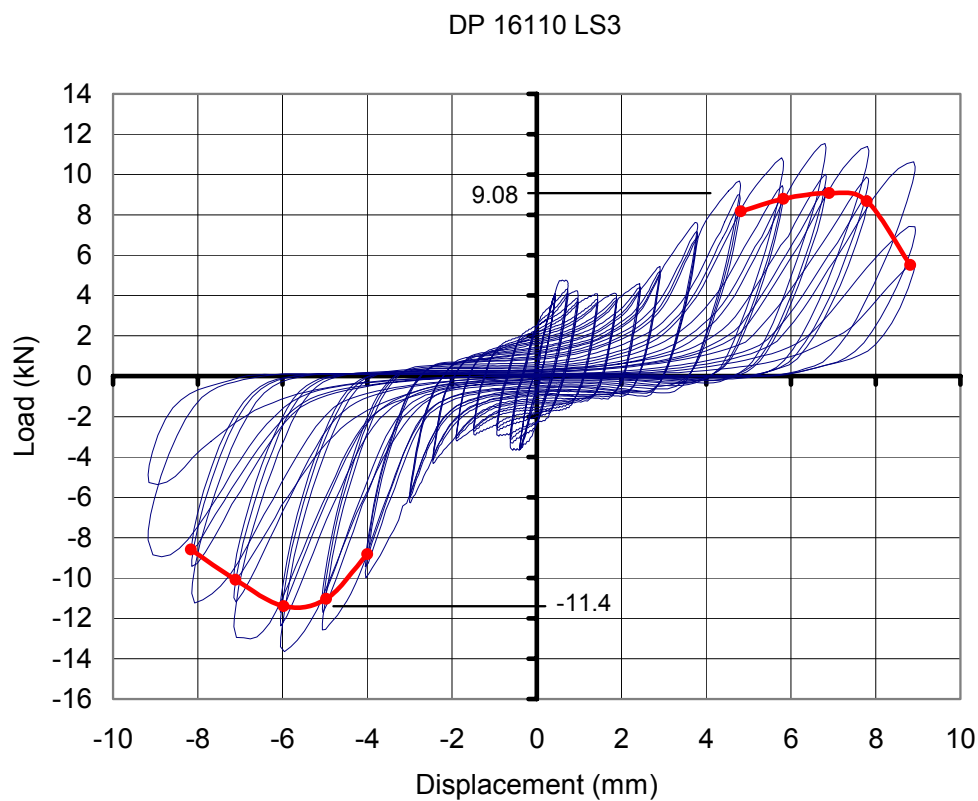


Figure 9: Definition of peak third cycle load.

$$\text{Peak third cycle load} = \frac{9.08 + 11.4}{2} = 10.2$$

Table 6: M12 Cast in bolt (masonry)

M12 Cast-in bolt Masonry slab edge	External Wall - 35 mm edge distance					
	In-plane		Out of plane		Tension	
	Peak load (kN)	Failure mode	Peak load (kN)	Failure mode	Peak load (kN)	Failure mode
Specimen 1	15.0	A,C	4.0	A	20.0	A
Specimen 2	13.0	A,C	5.5	A	26.0	A
Specimen 3	11.5	A,C	3.4	A	16.3	A
Specimen 4	10.3	A,C	6.3	A	23.8	A
Specimen 5	7.8	A,C	6.2	A	24.3	A
Specimen 6	9.0	A,C	-	A	24.1	A
Mean	11.1		5.1		22.4	
Std. deviation	2.7		1.3		3.6	
Coefficient of variation	0.2		0.3		0.2	
Characteristic strength (R_{ek})	5.4		2.2		12.8	

Note Peak load measured on 3rd cycle (see text)

Key to failure modes:
A = masonry face shell split off
B = failure of formed concrete edge
C = plate split
D = bond failure anchor to concrete
E = head pull through plate

Table 7: M12 Cast in bolt (concrete)

M12 Cast in bolt Formed concrete slab edge	External Wall - 50 mm edge distance					
	In-plane		Out of plane		Tension	
	Peak load (kN)	Failure mode	Peak load (kN)	Failure mode	Peak load (kN)	Failure mode
Specimen 1	12.8	C	8.0	C	23.4	B
Specimen 2	11.9	C	10.0	B	21.7	B
Specimen 3	12.0	C	8.8	C	22.2	B
Specimen 4	12.5	C	12.1	D	20.1	B
Specimen 5	13.3	C	9.3	-	23.6	B
Specimen 6	12.9	C	11.1	B	24.5	B
Mean	12.6		9.9		22.6	
Std. deviation	0.5		1.5		1.6	
Coefficient of variation	0.0		0.2		0.1	
Characteristic strength (R_{ek})	11.2		6.4		18.1	

Note Peak load measured on 3rd cycle (see text)

Key to failure modes:
A = masonry face shell split off
B = failure of formed concrete edge
C = plate split
D = bond failure anchor to concrete
E = head pull through plate

Table 8: Dynabolt (40 mm edge distance)

D16-110 Dynabolt Masonry slab edge	External Wall - 40 mm edge distance					
	In-plane		Out of plane		Tension	
	Peak load (kN)	Failure mode	Peak load (kN)	Failure mode	Peak load (kN)	Failure mode
Specimen 1	5.93 *	A	3.00	A	6.10	A
Specimen 2	3.37 #	A	3.56	A	6.10	A
Specimen 3	10.05	A	3.67	A	7.10	A
Specimen 4	8.90	A	2.88	A	12.20	A
Specimen 5	6.38	A	4.38	A	8.10	A
Specimen 6	7.62	A	3.15	A	13.40	A
Specimen 7	9.85					
Mean	8.56		3.44		8.83	
Std. deviation	1.55		0.56		3.18	
Coefficient of variation	0.18		0.16		0.36	
Characteristic strength (R_{ek})	4.69		2.26		3.55	

Note Peak load measured on 3rd cycle (see text)

Note: * Specimen 1 (In-plane), 35 mm edge distance.
Specimen 2 (In-plane), Drill wandered during installation. Result ignored.

Key to failure modes:
A = masonry face shell split off
B = failure of formed concrete edge
C = plate split
D = bond failure anchor to concrete
E = head pull through plate

Table 9: Dynabolt (50 mm edge distance)

D16-110 Dynabolt Masonry slab edge	External Wall - 50 mm edge distance					
	In-plane		Out-of-plane		Tension	
	Peak load (kN)	Failure mode	load (kN)	Failure mode	load (kN)	Failure mode
Specimen 1	12.97	A	3.73 *	A	14.01	A
Specimen 2	11.99	A	7.22	A	13.9	A
Specimen 3	10.44	A	6.91	A	12.63	A
Specimen 4	11.62	A	6.25	C	9.9	A
Specimen 5	13.36	A	7.16	C	14.9	A,D
Specimen 6	11.51	A	5.72	A	17.86	A
Specimen 7			6.9	A		
Mean	11.98		6.69		13.87	
Std. deviation	1.06		0.59		2.62	
Coefficient of variation	0.09		0.09		0.19	
Characteristic strength (R_{ek})	9.14		5.01		7.45	

Note Peak load measured on 3rd cycle (see text)

Note: * Specimen 1 (out of plane), oversized hole drilled (18mm). Result ignored.

Key to failure modes:
A = masonry face shell split off
B = failure of formed concrete edge
C = plate split
D = bond failure anchor to concrete
E = head pull through plate

Table 10: HD875 Drive pins

HD875 drive pins (With washers) Note: Gravity test sequence used for these tests. 5 pins used for in-plane specimens 1-4, 4 pins specimens 5&6, 2 pins all out-of-plane All results quoted per pin	Internal wall																																																																	
	<table border="1"> <thead> <tr> <th></th> <th colspan="2">In-plane</th> <th colspan="2">Out of plane</th> </tr> <tr> <th></th> <th>Peak load (kN)</th> <th>Failure mode</th> <th>Peak load (kN)</th> <th>Failure mode</th> </tr> </thead> <tbody> <tr> <td>Specimen 1</td> <td>5.06</td> <td>D</td> <td>4.71</td> <td>D</td> </tr> <tr> <td>Specimen 2</td> <td>4.28</td> <td>D</td> <td>-</td> <td></td> </tr> <tr> <td>Specimen 3</td> <td>4.70</td> <td>D,F</td> <td>4.6</td> <td>D</td> </tr> <tr> <td>Specimen 4</td> <td>4.60</td> <td>C,D</td> <td>6.5</td> <td>D</td> </tr> <tr> <td>Specimen 5</td> <td>4.10</td> <td>D</td> <td>4.43</td> <td>D</td> </tr> <tr> <td>Specimen 6</td> <td>4.80</td> <td>D</td> <td>5.07</td> <td>D</td> </tr> <tr> <td>Specimen 7</td> <td>-</td> <td></td> <td>5.6</td> <td>D</td> </tr> <tr> <td>Mean (per pin)</td> <td>4.59</td> <td></td> <td>5.15</td> <td></td> </tr> <tr> <td>Std. deviation</td> <td>0.35</td> <td></td> <td>0.78</td> <td></td> </tr> <tr> <td>Coefficient of variation</td> <td>0.08</td> <td></td> <td>0.15</td> <td></td> </tr> <tr> <td>Characteristic strength (R_{ek})</td> <td>3.66</td> <td></td> <td>3.53</td> <td></td> </tr> </tbody> </table>		In-plane		Out of plane			Peak load (kN)	Failure mode	Peak load (kN)	Failure mode	Specimen 1	5.06	D	4.71	D	Specimen 2	4.28	D	-		Specimen 3	4.70	D,F	4.6	D	Specimen 4	4.60	C,D	6.5	D	Specimen 5	4.10	D	4.43	D	Specimen 6	4.80	D	5.07	D	Specimen 7	-		5.6	D	Mean (per pin)	4.59		5.15		Std. deviation	0.35		0.78		Coefficient of variation	0.08		0.15		Characteristic strength (R_{ek})	3.66		3.53	
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It should be noted that, for the anchors loaded out-of-plane (and contrary to BRANZ EM1) the direction of loading that provided the lowest envelope curve was the one used to determine the residual strength. This was invariably with load acting toward the slab edge. For the anchors loaded in tension, only the tension result was used.

It was not expected that the drive pins without washers would resist higher in-plane loads than the pins with washers, although the greater variability resulted in a similar characteristic value. The explanation is likely to lie in the difference in the load sharing capability of the two primary failure mechanisms. Fastener head pull-through the timber is relatively ductile, and allows the load to be spread more evenly between fasteners, while the concrete pull-out mechanism does not. Thus overall failure occurs as soon as the first fastener fails.

Tables 6 to 11 also include (for each set of anchor tests) mean, standard deviation, and coefficient of variation, calculated using standard statistical procedures, and characteristic strength, R_{ek} , calculated in accordance with equation (2a) of BRANZ EM1. For completeness equation (2a) is reproduced below:

$$R_{ek} = P_{(\min)} \left(\frac{n}{27} \right)^v$$

where: v = the coefficient of variation of the individual values,

n = the number of specimens in the sample

$P_{(\min)}$ = minimum value of P_e (peak load of the individual test).

A capacity reduction factor, ϕ , of 0.8 was used to derive the capacities of the various fasteners in accordance with clause 2.4.7 of NZS 3604. The value of 0.8 was chosen for ϕ rather than the value of 0.7 as suggested in NZS 3603, because the test set up and the EM1 method used far more closely simulates the actual in-service conditions than the much more general tests that formed the basis for characteristic values of fasteners set out in NZS 3603.

The results of this calculation for anchor capacity have been included in the summary Table 12. Refer to the Conclusion section of this report for comments about anchor performance in conjunction with masonry header blocks. Adjusted spacings are included (figures in brackets) so the anchors are able to meet equivalent performance per length of bottom plate.

5.2 Tests carried out at BRANZ on proprietary anchor bolts

The following tests (arranged in alphabetical order) were undertaken for various private clients, whose permission has been given to reproduce the results here. Those not included either may not have tested their bottom plate anchors at BRANZ, or may have withheld permission for their own reasons.

For further information and a copy of the relevant BRANZ test report giving the detailed test information, contact the manufacturer concerned.

5.2.1 Blacks Fasteners Ltd

PO Box 7229
Sydenham
Christchurch.

12mm x 130mm wedge anchor, hot dip galvanised.

5.2.2 Hilti (NZ) Ltd

PO Box 112 030
Penrose
Auckland.

HSA stud anchor, M12 x 150

HIT HY 150 injection system with M12 x 160 HAS threaded rod

X-DNI 72 MX Pin.

5.2.3 Powers Fasteners (NZ) Ltd, and Co

Private Box 302 076
North Harbour Postal Centre
Auckland.

SBA 12135, Wedge anchor

Acrylic 100, 8480 threaded rod, Chemical anchor

Excalibur HSB 12/150, Screwbolt.

5.2.4 Reid Engineering Systems Ltd

PO Box 101 157
North Shore Mail Centre
Auckland.

HSB12/150 Hexagon screw bolt

HSB12/100 Hexagon screw bolt

CAS12-160 Chemical stud anchor

WA12140 Wedge anchor

WA12120 Wedge anchor

SA12-99 Sleeve anchor

SA12-129 Sleeve anchor

The results from these tests are included in the summary Table 12.

Where a test result for a proprietary anchor did not reach the target criteria set by NZS 3604, an equivalent (adjusted) spacing was given to the manufacturer to allow the anchor to be used and achieve the same strength per metre as the target criteria (based on a spacing of 900 mm). The adjustment was calculated as follows:

$$\text{Adjusted spacing} = \frac{\text{achieved capacity}}{\text{target capacity}} \times 900 \text{ mm}$$

These adjusted spacings are given in brackets in the table. If the revised criteria are accepted by the NZS 3604 committee, then these spacings will need to be revisited.

Table 11: Overall result summary

Anchor (all for single fasteners)	Slab edge type	Edge distance	Load direction	Characteristic value	Capacity	NZS 3604 criteria	Pass/fail (adj. spacing)	Suggested criteria	Pass/fail
M12 bolt (NZS 3604)	Masonry	35mm	In plane	5.44	4.35	5.0	Fail	2.0	Fail
			Out of plane	2.20	1.76	4.0		3.0	
			Tension	11.36	9.09	8.0		7.0	
	Concrete	50mm	In plane	11.19	8.95	5.0	Pass	2.0	Pass
			Out of plane	6.35	5.08	4.0		3.0	
			Tension	18.09	14.47	8.0		7.0	
Ramset Dynabolt D16-110	Masonry	40mm	In plane	4.69	3.75	5.0	Fail	2.0	Fail
			Out of plane	2.26	1.81	4.0		3.0	
			Tension	3.55	2.84	8.0		7.0	
	Masonry	50mm	In plane	9.14	7.31	5.0	Fail	2.0	Fail
			Out of plane	5.01	4.01	4.0		3.0	
			Tension	7.45	5.96	8.0		7.0	
Ramset drive pin HD875 (no washer)	Internal	-	In plane	3.48	2.78	5.0	Fail (385mm)	2.0	Fail
			Out of plane	2.15	1.72	4.0		2.0	
Ramset drive pin HD875 (with washer)	Internal	-	In plane	3.66	2.92	5.0	Fail (635mm)	2.0	Pass
			Out of plane	3.53	2.82	4.0		2.0	

Table 11: (continued)

Anchor (External)	Slab edge type	Edge distance	Load direction	Characteristic value	Capacity	NZS 3604 criteria	Pass/fail (adj. spacing)	Suggested criteria	Pass/fail
Blacks Fasteners 12x130 wedge anchor	Concrete	50mm	In plane	7.67	6.14	5.0		2.0	
			Out of plane	4.63	3.70	4.0	Fail (830mm)	3.0	Pass
			Tension	14.20	11.36	8.0		7.0	
Hilti M12x150 HAS stud anchor	Concrete	50mm	In plane	12.27	9.82	5.0		2.0	
			Out of plane	5.07	4.06	4.0	Pass	3.0	Pass
			Tension	13.37	10.70	8.0		7.0	
Hilti HIT HY injection system	Concrete	50mm	In plane	14.55	11.64	5.0		2.0	
			Out of plane	5.97	4.78	4.0	Pass	3.0	Pass
			Tension	21.14	16.91	8.0		7.0	
Powers SBA 12135 Wedge anchor	Concrete	50mm	In plane	9.69	7.75	5.0		2.0	
			Out of plane	5.19	4.16	4.0	Pass	3.0	Pass
			Tension	11.82	9.45	8.0		7.0	
Powers Excalibur HSB12/150	Masonry	50mm	In plane	12.85	10.28	5.0		2.0	
			Out of plane	4.78	3.82	4.0	Fail (865mm)	3.0	Pass
			Tension	13.24	10.59	8.0		7.0	
Powers Acrylic 100, 8480	Masonry	50mm	In plane	10.24	8.19	5.0		2.0	
			Out of plane	3.87	3.10	4.0	Fail (700mm)	3.0	Pass
			Tension	10.77	8.61	8.0		7.0	
Reid HSB12/150	Masonry	50mm	In plane	8.14	6.51	5.0		2.0	
			Out of plane	3.61	2.89	4.0	Fail (650mm)	3.0	Fail
			Tension	17.76	14.21	8.0		7.0	
Reid CAS12-160	Concrete	50mm	In plane	12.78	10.22	5.0		2.0	
			Out of plane	5.64	4.51	4.0	Pass	3.0	Pass
			Tension	21.25	17.00	8.0		7.0	
Reid WA12140	Masonry	50mm	In plane	10.93	8.74	5.0		2.0	
			Out of plane	4.16	3.33	4.0	Fail (750mm)	3.0	Pass
			Tension	13.32	10.66	8.0		7.0	
Reid WA12140	Concrete	50mm	In plane	9.11	7.29	5.0		2.0	
			Out of plane	4.62	3.70	4.0	Fail (833mm)	3.0	Pass
			Tension	12.97	10.38	8.0		7.0	
Anchor (Internal)	Slab edge type	Edge distance	Load direction	Characteristic value	Capacity	NZS 3604 criteria	Pass/fail (adj. spacing)	Suggested criteria	Pass/fail
Hilti X-DNI 72 MX Pins	Internal	-	In plane	3.19	2.55	4.0	Fail (574mm)	2.0	Fail
			Out of plane	2.44	1.95	3.0		2.0	
Reid HSB12/100	Internal	-	In plane	13.77	11.01	4.0		2.0	
			Out of plane	13.31	10.65	3.0	Pass	2.0	Pass
Reid SA12-99	Internal	-	In plane	5.83	4.66	4.0		2.0	
			Out of plane	14.84	11.87	3.0	Pass	2.0	Pass
Reid SA12-129	Internal	-	In plane	12.00	9.60	4.0		2.0	
			Out of plane	12.93	10.34	3.0	Pass	2.0	Pass
Reid WA12120	Internal	-	In plane	5.12	4.10	4.0		2.0	
			Out of plane	9.02	7.22	3.0	Pass	2.0	Pass

6. CONCLUSIONS

Table 12 summarises the anchor capacities derived, plus those of the proprietary tests, along with the NZS 3604:1999 target values and suggested revised values for comparison.

It is evident that the common practice of using masonry header blocks to form slab edging results in a significant loss of strength from several bottom plate anchors, including those specifically prescribed by NZS 3604. To meet the criteria, cast-in M12 bolts could be lengthened (to perhaps 180 mm, as in USA) to provide greater embedment and thus more reliable fixing, but this was not investigated in this project. It is also evident the edge distance is a critical parameter, and considering the tolerances expected in NZ domestic construction (SNZ, 1997) it is concluded that the margin is too low (even when using the revised criteria) for reliable performance.

This edge detail was not envisaged by the writers of the original standard in 1978, and has merely found its way into popular usage by default. The acceptability of this method of forming slab edging should be carefully re-assessed if the New Zealand building industry is to provide a reliable fixing for wall frames to concrete floor slabs. If it is to become formally accepted, then some detail changes may be needed to maintain a constant standard of performance for bottom plate anchors.

It is suggested that the revised criteria set out in Table 4 are more realistic than those currently in NZS 3604:1999, being based on a rational assessment of the in-service load demand. The drafting committee should consider these criteria with a view to an appropriate revision to NZS 3604.

7. REFERENCES

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8. APPENDIX

8.1 In-plane wall loads

Estimation of in-plane wall loads resulting from the participation of non-designated bracing walls under lateral loading.

- 1 Assume the maximum bracing load for the lower storey of a 2 storey building on a concrete floor slab (Table 5.10 of NZS 3604:1999) is 24 BU/m². This equates to 1.2 kN/m².
- 2 Assume the “designated” bracing walls resist 75% of this bracing demand, thus “other” walls will resist 25%.
- 3 Assume the “other” walls are spaced at 4 metres, and the anchor bolts at an average spacing of 1.2 m along those walls. Thus tributary area for each bolt is 1.2 x 4 = 4.8 m².
- 4 Therefore the “accidental” bolt load is 1.2 x 0.25 x 4.8 = 1.44 kN.

Considering that the external walls are likely to attract a higher in-plane load than the internal walls and the crudeness of the estimate, it would be prudent to round this up to 2 kN per bolt.

8.2 Representative load displacement plots

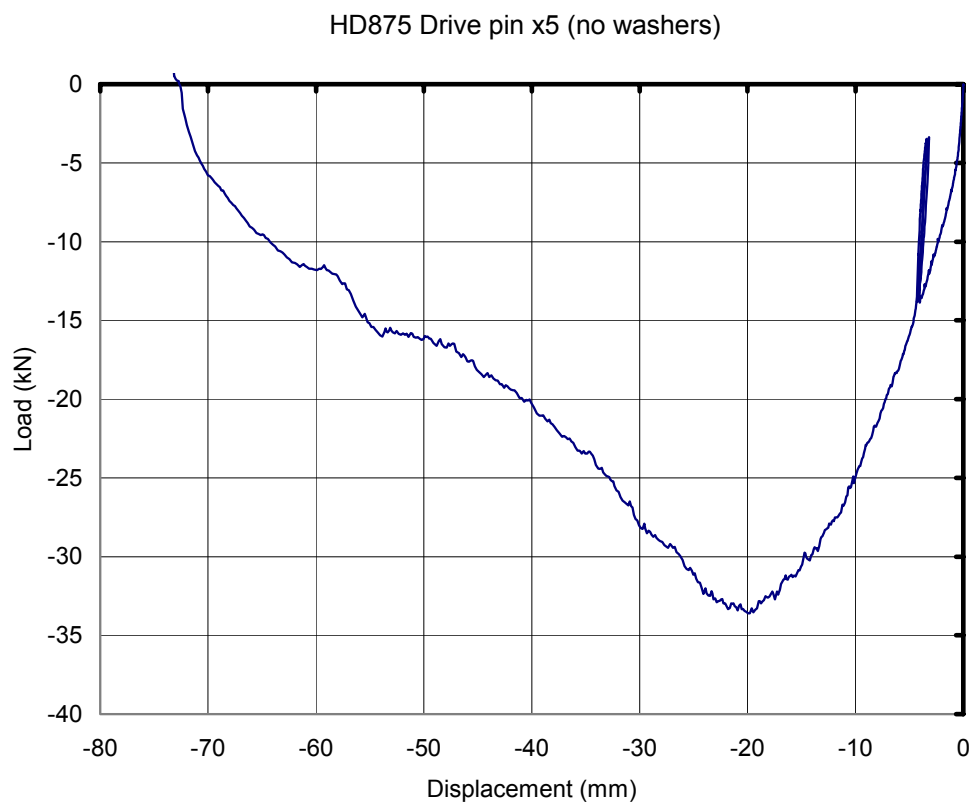


Figure 10: Typical plot of drive pins

Dynabolt, shear out-of-plane

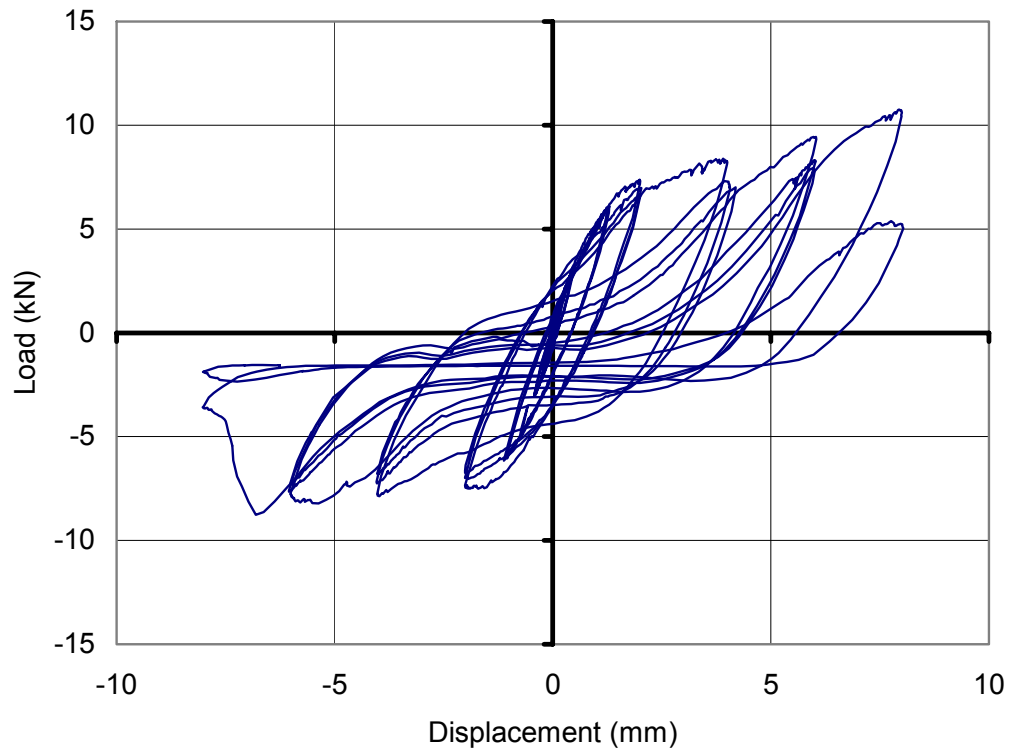


Figure 11: Typical plot of sleeve anchor in shear out-of-plane

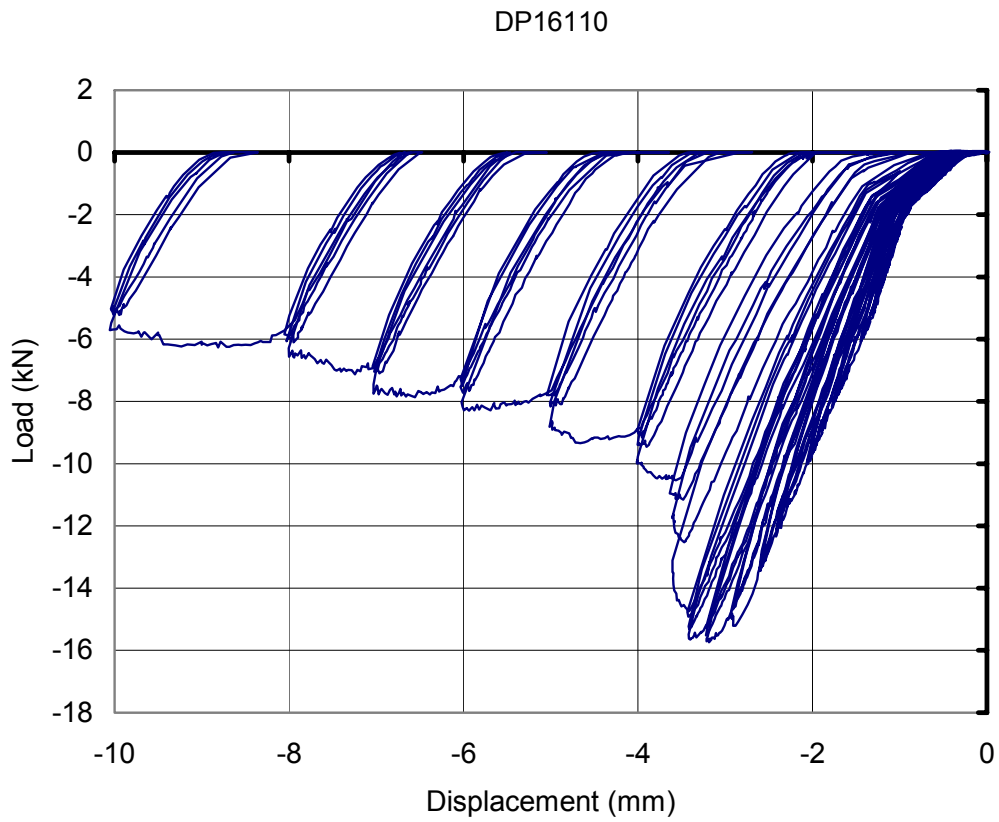


Figure 12: Typical plot of sleeve anchor in tension. (note bolt withdrawing from concrete)

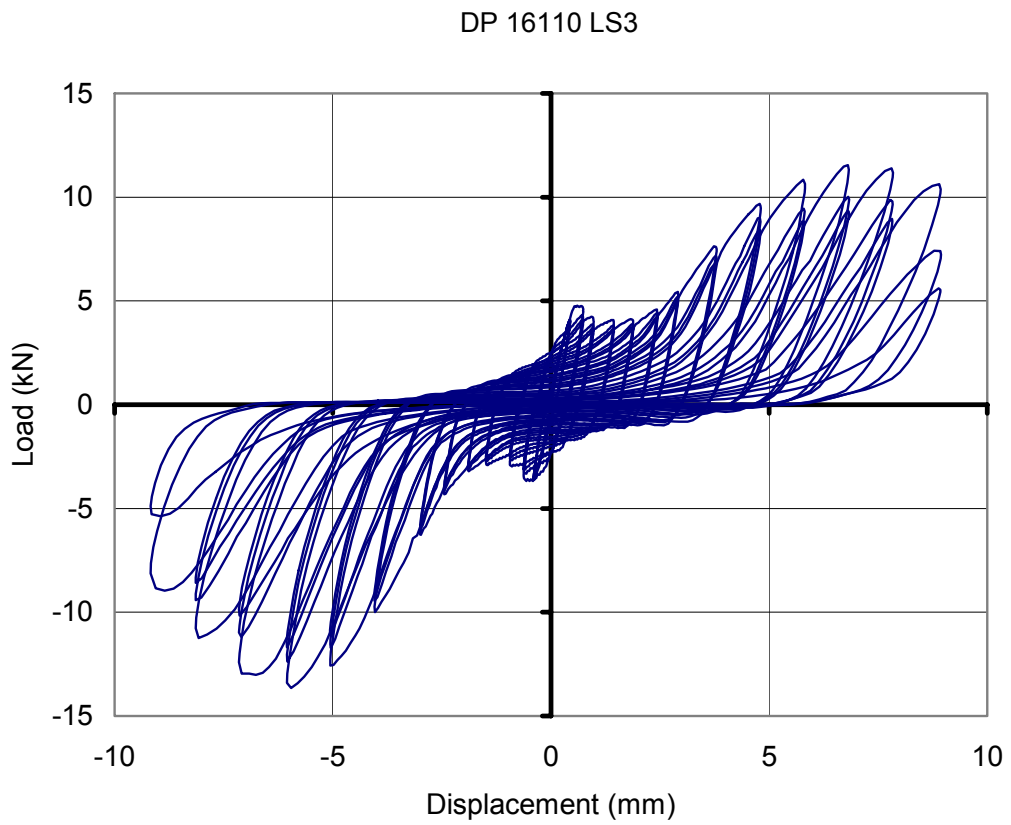


Figure 13: Typical plot of sleeve anchor under in-plane shear