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The development of generic span tables for cold formed steel studs in residential and low-rise construction

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Abstract

The process of roll forming of steel provides the designer with virtually unlimited possibilities in the shape of the section which has led to the development a wide range of very efficient proprietary sections. However this has made it difficult for building designers and engineers to design and specify a house or other low-rise structure given the number of different sections available and complexity of the cold-formed steel standard AS/NZS 4600.

The sections commonly used in Australia and America were reviewed and a wide range of sections were chosen to cover the range of sections commonly used. They covered lipped and plain C channels made from G300 and G550 steel. These sections were investigated to determine their load carrying capacity. Factors evaluated included effective lengths for both clad and unclad studs, width of studs, height of walls, connection types, and influence of noggings. From this analysis a standard stud was chosen which was specified in terms of load capacity rather than section shape and thickness. This allows manufacturers to develop innovative new sections for which the span tables permit valid selection. It also allows last minute changes and substitutions to be made on site simply and quickly.

The load combinations are determined from the NASH Standard - Residential and Low-rise Steel Framing Part 1 Design Criteria. The critical load cases are identified and discussed. Examples of the resulting span tables are given together with typical installation details.

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1. Introduction

Steel product manufacturers have developed a wide range of innovative stud shapes to efficiently carry vertical loads and horizontal wind loads in low-rise building structures. Most of these shapes are owned by the relevant manufacturers, many of whom do not publish comprehensive load tables for their products. This has made it difficult for architects, building designers and engineers to quickly design and specify light steel framing for smaller structures. In addition, building surveyors and inspectors have had trouble in simply inspecting and approving the steel framing.

This paper reviews the design principles of studs and proposes some generic span tables that can be used in the design and specification of the wall framing. The steel frame fabricator may choose to adopt the generic design or may decide to optimise the design to suit their own particular system. The designer can confidently select structural components from a range of products based on published specifications and/or physical properties. In addition the connection details provided for the generic design will assist with inspections on site and in setting acceptable standards.

The concept is illustrated with the single storey external load bearing common stud in non-cyclonic areas.

2. History of the development of wall studs

The development of the modern steel wall stud was undertaken by John Lysaght (Aust) Ltd in the 1960's. (Watson 2007) Lysaght developed a welded wall frame system based around a 75 x 35 x 1.2 mm G300 Z200 steel channel. The studs were typically spaced at 450 mm or 600 mm to suit the internal lining eg plasterboard and external claddings eg brick, fibre cement, weatherboards.

In the 1980's a major wave of new developments was stimulated by the introduction of:

- Relatively low cost programmable roll formers
- Personal computers
- Aluminium / zinc coated steels
- G550 steels

This lead to the development of more efficient sections with less steel required to carry the given load. As illustrated in Figure 1, this was achieved by stiffening the flanges and webs of the sections but at the cost of more expensive roll formers. The thinner steel allowed the introduction of new forms of fastening in the factory including screws, rivets, nails and clinches.

The introduction of low cost programmable roll formers meant that the wall studs could now be economically rolled by the frame fabricator, rather than relying on large central roll forming manufacturing facilities.

Whilst initially the developments were based around the stud depth of 75 mm, later developments by some fabricators including matching the standard timber depth of 70 mm and 90 mm. Local historical practice still influences available stud depth to some extent.

Figure 1 Different types of stud profiles

3. Critical loading combinations

The following load combinations for the design of studs are given in the NASH Standard (NASH 2005):

$1.2\ G + 1.5\ Q_1$ (1)
$1.2\ G + 1.5\ Q_2$ (2)
$1.2\ G + (W_{uw} + W_{ur\ (down)})$ (3)
$0.9\ G + (W_{uw} + W_{ur\ (up)})$ (4)

Where
G = dead load of roof structure, including roof structure, roof cladding, roof battens, ceiling battens, ceiling, services and roof insulation if appropriate
Q_1 = roof distributed live load = 0.25 kPa
Q_2 = roof concentrated live load = 1.1 kN
W_{uw} = wind load normal to wall
W_{ur} = wind load perpendicular to the roof

The design of the stud is normally governed by the third combination (Equation 3) of dead load, wind perpendicular to wind ward wall and downward wind pressure on the roof. As the tensile capacity of the stud is typically 3-5 times the compressive capacity of the member, the uplift combination (Equation 4) does not generally govern the capacity. However the capacity of the connection of the stud to the top or bottom plate under uplift may be critical, particularly with light weight roofs.

The extra compressive force from bracing is usually not considered in the design of the wall studs as:

- The maximum wind load on a side wall is 15% less than the windward wall, with a positive internal pressure.
- The positive internal pressure reduces the downward pressure on the roof by approximately 70%.

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- The bracing is typically anchored at the end of a panel where there are at least two studs,

The NASH Standard and Handbook (NASH 2008) notes that an *action combination factor* may be applied to wind action effects when designing wall studs. This factor (K_c) is explained in AS/NZS 1170.2 Clause 5.4.3, and represents a concession in the magnitude of combined wind action effects in a major member resulting from simultaneous wind pressures on two or more surfaces. It reflects research observations that peak wind pressures on different surfaces of a building do not closely coincide but are slightly "out of phase" in their effects on an individual member. Therefore when multiple peak pressures are combined for a specific critical load case, the probability of coincidence is very small and the combined action effect may be factored accordingly, as described in AS/NZS 1170.2 Table 5.5.

In developing the load cases for standard wall stud sections, two critical wind action cases have been considered:

1. Stud in windward impermeable wall with positive (downward) pressure on the roof (for pitch between 20^o and 35^o) combined with external pressure and internal suction.
2. Stud in side wall with windward permeable wall and positive (downward) pressure on the roof (for pitch between 20^o and 35^o) combined with external suction and internal pressure.

In both these cases, a value for K_c of 0.8 may be used.

4. Design of wall studs

The NASH Handbook provides guidance on the critical design conditions as well as some key design assumptions such as effective lengths. Wall design can be carried out by calculation or by prototype testing or by a combination of both.

The contribution of the cladding to the load carrying capacity of the wall needs careful consideration. Experimental evidence indicates that plasterboard can carry considerable compression load. However, it is common practice to consider claddings such as plaster board as restraints to the stud members but ignore their contribution to compression or bending capacities because that contribution cannot always be relied upon. Where testing is used as the means to assess load capacities, then care should be taken in setting up the test to eliminate the cladding contribution to the stud axial load capacity but to maintain its contribution to stud restraint.

For brick veneer walls, it is common design practice to assume that the steel frame will carry all the loads (vertical and lateral) and the brick wall acts only as cladding and relies on the steel frame for lateral stability through the brick ties. This is a safe assumption to make for the ultimate limit state.

5. Section performance criteria

One of the principal functions of a stud is to support the external cladding and internal lining. AS/NZS 2589 (Standards Australia 2007) specifies a minimum bearing width of 32 mm for a stud at a plasterboard joint.

Most houses are constructed in non-cyclonic areas in regions N1 or N2 as classified by AS 4055 with a small number in region N3. The moments resulting from wind actions are given Table 1. Wall heights in residential construction are typically 2700 mm or 2400 mm with a smaller number

of 3000 mm walls. Higher walls are becoming more common and these usually require additional framing or larger sections than used for the traditional heights.
The height of the external wall studs used in calculations has been increased by 70 mm over the nominal ceiling height to allow for timber flooring, gap from floor to bottom of wall plaster, ceiling lining and battens.

Table 1 Stud bending moments (kNm) from wind action on windward face

Wind Class (Ultimate wind speed)		**N1 (34 m/s)**		**N2 (40m/s)**		**N3 (50m/s)**	
Stud spacing (mm)		**450**	**600**	**450**	**600**	**450**	**600**
Stud height (mm)	2400	0.238	0.317	0.329	0.439	0.515	0.686
	2700	0.299	0.399	0.414	0.552	0.647	0.863
	3000	0.368	0.490	0.509	0.679	0.795	1.060

6. Generic stud selection and span tables

The capacities of some typical lipped and unlipped channels were determined using Coldsteel, a program developed by Sydney University. The sections considered were 70, 75 and 90 mm deep as shown in Table 2. Base metal thicknesses of 0.55, 0.75, 1.0 and 1.2 mm were evaluated in G300 and/or G550 steel. Lipped channels were designated as L and plain channels as C. These sections are based on current industry practice. The main difference between these generic sections and the proprietary sections is that webs of proprietary sections often have stiffeners incorporated giving them generally a higher capacity.

The capacities of the studs are given in Table 3. From these capacities, span tables can be developed for the relevant studs. However, the stud chosen by the designer may not be available in the local region. Therefore it was decided to specify the stud with a set of minimum design capacities in both bending and compression. In addition, this approach allows manufacturers to certify that their studs meets the minimum requirements and therefore the tables produced are applicable. This would be particularly useful when making late changes on site. This approach also encourages manufacturers to develop innovative new studs.

For instance, a Type A stud could have the minimum properties as set out in Table 4. From these parameters, the span tables can be derived as illustrated in Table 5. The depth of stud assumed in the design would prefix the types descriptor ie a 70A stud would have a depth of 70 mm with the capacities as given in Table 4.

Prior to offering specific stud products in compliance with the design tables, the manufacturer of a proprietary stud would need to determine its capacity through calculation in accordance with AS/NZS 4600 and/or through testing. The appropriate effective length would need to be evaluated for the system.

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Table 2 Section parameters

Stud Designation[1]	**Steel grade**	**d mm**	**b mm**	**t mm**	**Stud Designation[1]**	**Steel grade**	**d mm**	**b mm**	**t mm**
L07035055550	G550	70	35	0.55	C07035055550	G550	70	35	0.55
L07035075550	G550	70	35	0.75	C07035075550	G550	70	35	0.75
L07035100550	G550	70	35	1.0	C07035100550	G550	70	35	1.0
L07535100300	G300	75	35	1.0	C07535100300	G300	75	35	1.0
L07535120300	G300	75	35	1.2	C07535120300	G300	75	35	1.2
L07535055550	G550	75	35	0.55	C07535055550	G550	75	35	0.55
L07535075550	G550	75	35	0.75	C07535075550	G550	75	35	0.75
L07535100550	G550	75	35	1.0	C07535100550	G550	75	35	1.0
L09035100300	G300	90	35	1.0	C09035100300	G300	90	35	1.0
L09035120300	G300	90	35	1.2	C09035120300	G300	90	35	1.2
L09035055550	G550	90	35	0.55	C09035055550	G550	90	35	0.55
L09035075550	G550	90	35	0.75	C09035075550	G550	90	35	0.75
L09035100550	G550	90	35	1.0	C09035100550	G550	90	35	1.0

Note

1. Designation X-DDD-BB-TTT-GGG where
   X is the shape code (L = lipped channel, C = unlipped channel)
   DDD is the depth of the section in mm (d)
   BB is the width of the flange in mm (b)
   TTT is the base metal thickness x 100 (t) eg 0.55 is represented by 055
   GGG is the yield stress in MPa
2. The minimum length of the lip is 5 mm for lipped channels

Table 3 Section Capacities

Stud Size	Steel grade	ΦN_t (kN)	ΦN_s (kN)	ΦN_c (kN)			ΦM_{sx} (kNm)	ΦM_{bx} (kNm)			N_{ox} (kN)		
				2400	2700	3000		2400	2700	3000	2400	2700	3000
L07035055550	G550	25.09	11.35	5.64	4.91	5.99	0.42	0.36	0.34	0.38	32.69	26.09	21.24
L07035075550	G550	40.84	21.41	9.03	8.04	9.45	0.81	0.59	0.53	0.67	43.65	34.84	28.36
L07035100550	G550	59.62	36.98	14.38	12.08	15.30	1.24	0.81	0.71	0.99	56.63	45.21	36.81
L07535120300	G300	45.27	32.59	19.61	16.99	20.63	0.99	0.85	0.81	0.90	77.93	62.22	50.65
L07535055550	G550	25.95	11.37	6.01	5.24	6.45	0.45	0.38	0.36	0.40	38.23	30.53	24.85
L07535075550	G550	42.25	21.47	9.54	8.52	10.98	0.87	0.65	0.58	0.74	51.08	40.78	33.19
L07535100550	G550	61.73	37.13	15.63	13.17	16.52	1.37	0.88	0.78	1.09	66.34	52.96	43.12
L09035100300	G300	42.06	24.20	17.07	15.63	18.00	1.02	0.87	0.83	0.92	101.13	80.74	65.73
L09035120300	G300	49.95	33.22	22.69	20.19	24.32	1.28	1.09	1.04	1.16	119.00	95.01	77.35
L09035055550	G550	28.54	11.43	6.89	6.06	7.49	0.54	0.45	0.43	0.48	58.07	46.36	37.74
L09035075550	G550	46.51	21.61	10.83	9.68	12.66	1.04	0.83	0.75	0.90	77.70	62.03	50.50
L09035100550	G550	68.04	37.48	17.98	15.96	19.70	1.78	1.14	1.00	1.41	101.13	80.74	65.73
C07035075550	G550	38.88	15.59	6.53	5.71	7.05	0.54	0.40	0.34	0.48	41.06	5.71	26.69
C07535075300	G300	27.68	11.60	6.92	6.09	7.42	0.42	0.36	0.34	0.39	48.08	38.38	31.25
C07535100300	G300	36.64	19.86	11.69	10.30	12.47	0.62	0.53	0.50	0.57	63.58	50.43	40.98
C07535120300	G300	43.70	27.68	16.15	14.25	17.10	0.78	0.68	0.64	0.73	74.67	59.62	48.53
C07535075550	G550	40.30	15.65	6.96	6.09	7.61	0.58	0.45	0.38	0.52	48.08	38.38	31.25
C07535100550	G550	59.26	28.87	11.80	10.30	12.81	1.03	0.67	0.57	0.81	63.06	50.34	40.98
C09035100300	G300	40.54	20.21	13.20	11.93	14.26	0.81	0.69	0.64	0.75	96.20	76.80	62.53
C09035120300	G300	48.39	28.31	18.38	16.30	19.79	1.02	0.88	0.82	0.95	114.08	91.08	74.15
C09035075550	G550	44.56	15.79	8.00	7.02	9.08	0.70	0.56	0.50	0.62	73.22	58.46	47.59
C09035100550	G550	65.58	29.22	13.72	12.02	15.52	1.33	0.88	0.74	1.07	96.20	76.38	62.53

Notes

a. One row of noggings at mid-height has been adopted for 2400 and 2700 mm high studs and 2 rows equally spaced for 3000 mm.

b. The effective length has been taken as 0.80 times the appropriate length as influence of the stiffness of the nogging and top and bottom plates has been allowed for. The connections need to be designed accordingly.

c. An additional 70 mm was added to the length of the stud to allow for timber flooring, gap to plasterboard, ceiling battens and lining.

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Table 4 Minimum properties for Type A Stud

Property	Nominal length (mm)			Units
	2400	2700	3000	
ΦN_t	40	40	40	kN
ΦN_s	23	23	23	kN
ΦN_c	15	13	16	kN
N_{ox}	60	50	40	kN
ΦM_{sx}	0.70	0.70	0.70	kNm
ΦM_{bx}	0.62	0.58	0.68	kNm

Table 5 Maximum roof load widths for Type A stud (wind classification N2)

Stud Type	Stud Height (mm)	Required Minimum Stud-Plate Connection Capacity (kN)	Rafter / Truss Spacing (mm)					
			600	900	1200	600	900	1200
			Maximum Roof Load Width (mm)					
			Sheet Roof			Tile Roof		
Type A			Stud Spacing 450					
	2400	8.63	17176	11451	8588	9606	6404	4803
	2700	5.58	11097	7398	5549	6207	4138	3103
	3000	6.42	12773	8515	6386	7144	4762	3572
Type A			Stud Spacing 600					
	2400	6.50	12940	8626	6470	7237	4825	3619
	2700	3.10	6172	4115	3086	3452	2301	1726
	3000	3.22	6405	4270	3203	3582	2388	1791

Notes:

Roof self weight including cladding, ceiling, light weight insulation, battens (NASH Standard Appendix E3):

- Steel sheeting 0.15 kN/m^2
- Tiled roof 0.70 kN/m^2
- Truss self weight 0.10 kN/m^2

$K_c = 0.8$

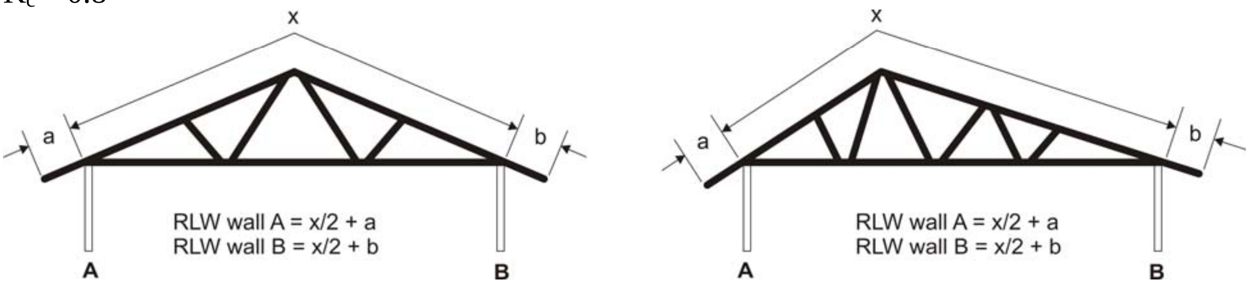

Figure 2 Roof load width definitions

Table 5 also gives the minimum connection downward capacity. This can be transferred through bearing at the end of the studs or by connectors. Typical connectors include screws, rivets, clinches and welding. Often rivets and screws are combined with dimples to provide a flat surface and to increase the capacity of the connection. Table 6 gives the capacity of various screws where connecting the two sheets of the same thickness and grade together and this can be used to determine the number of screws required in a connection. When bearing is used to

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transfer the force enough mechanical fasteners are required to resist any uplift forces. This particularly applies for steel roofs and could conservatively be taken as the force given.

Table 6 Design capacity of a screwed connection in shear (ΦV_b) (kN / screw) (NASH 2008)

Grade of Steel	**G300**			**G500**	**G550**		
Thickness of Sheet (mm)	0.8	1.0	1.2	1.2	0.55	0.75	1.0
Screw designation							
6 gauge	0.96	1.34	1.76	2.69	0.66	1.26	2.16
8 gauge	1.05	1.46	1.92	2.94	0.72	1.38	2.37
10 gauge	1.12	1.56	2.06	3.14	0.77	1.48	2.53
12 gauge	1.20	1.67	2.20	3.37	0.82	1.58	2.71
14 gauge	1.28	1.79	2.36	3.60	0.88	1.69	2.90

7. Conclusion

The approach outlined in this paper makes it possible for relatively common standard steel sections to be selected and used in low-rise building construction applications without specific engineering design. It also facilitates the mixed use of proprietary and standard sections in the one structural project. Finally, it establishes a framework which may be applied to the development of generic performance tables for other structural components.

8. Acknowledgements

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