PROTOTYPE TESTING PROCEDURE FOR FLANGE HUNG POTIUS PANEL

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ABSTRACT

The new Arts and Media building at the Nelson Marlborough Institute of Technology (NMIT) is nearing completion. The building boasts some of the latest technology in timber engineering including post tensioned shear walls, timber/concrete composite beams and stressed shin panels.

The primary structure was fabricated by Hunter Laminates using Nelson Pine LVL, manufactured from locally grown pine. All the timber used in the primary structure was grown, manufactured and fabricated within 100 km from where it was erected.

POTIUS double 'T' stressed skin panels were proposed for the project. Irving Smith Jack Architects Ltd had specific requirements for the panel support detail. The panels were to be hung between the bearers, they did not want visible fixings and the solution had to be cost effective.

STRESSED SKIN PANEL DESIGN

POTIUS Building Systems Ltd has been manufacturing stressed skin panels in Nelson for three years using Nelson Pine LVL. The panels are designed in accordance with widely published stressed skin panel theory.

The configuration varies from one project to another. For the NMIT building 360 x 90 mm LVL webs are rigidly connected (glued) to a 1200 x 36 mm skin in a double 'T' configuration. The undersides of the panels are exposed, for architectural reasons, so the joists are oversized to provide the required thirty minute fire resistance in accordance with NZS 3603 Cl 9.4.5. A non-structural concrete topping is placed in-situ.

FLANGE HUNG CONNECTION

A flange hung panel system (Figure 1) was proposed for supporting floor panels. The skin of the panel cantilevers beyond the panel web to bear on the supporting beam. In this case the maximum distance the skin can cantilever before it fails in bending, is approximately 160 mm. The configuration therefore requires the transfer of the vertical shear force from the skin to the web of the panel. This results in a tension force in the web, perpendicular to the grain.

There is no value in NZS 3603 for the tensile capacity perpendicular to the grain for timber (or LVL). There is also no methodology to readily determine how much of the shear force will transfer into the web.

Nelson Pine had some very limited test results for LVL in tension perpendicular to the grain. Calculations were carried out assuming that the entire reaction force will transfer to the web. To get a preliminary indication of the capacity of the connection it was also assumed that the area in tension is the width of the joist multiplied by the distance from the end of the joist to the point where the skin fails in bending. Multiplying this area by the tension perpendicular to grain value supplied by Nelson Pine

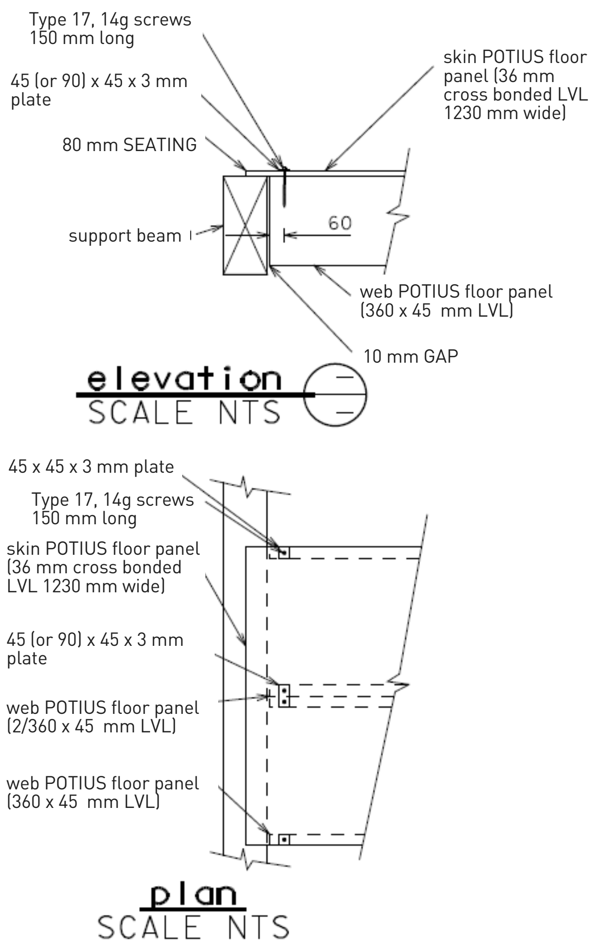

Figure 1. Flange hung panel system.

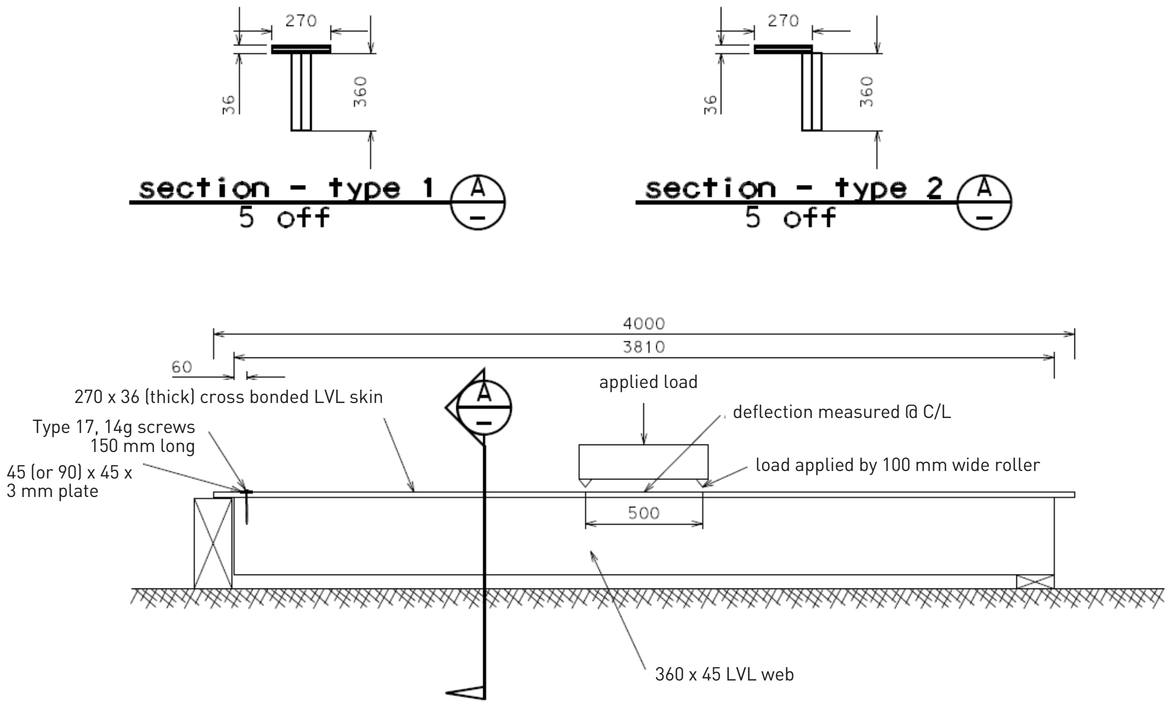

Figure 2. Proposed testing configuration

gives an approximation of the capacity of the flange hung panel with no additional strengthening.

In this instance the capacity of the connection, after load factors were applied, was not adequate. There is also concern that, should the connection fail it would be a brittle 'unzipping' mechanism.

Preliminary testing confirmed this showing that the timber failed in tension (perpendicular to grain). Unexpectedly, it was the bottom veneer of the skin that failed. It was therefore proposed to use 150 mm type 17 screws to transfer the force into the joist.

TEST PROCEDURE

The testing procedure was prepared in accordance with NZS1170:2002 Appendix B.

The maximum width of unit that would fit in the testing equipment available was 270 mm. Therefore the test unit configuration used was a 270 x 36 mm skin and 90 x 360 mm web. This was, if anything, a slightly conservative representation of the actual configuration proposed (and therefore allowed for the possibility of penetrations through the skin for services). Two type 17, 14 g screws per 90 mm web will be installed with a 90 x 45 x 3 mm steel pate acting as a washer (Figure 2). Hunters manufactured the test units.

Panel test units were 3.5 m long. This was to induce stresses in the panel similar to the in service situation where there is no moment at the support, but there will be some horizontal shear force in the connection that may have an effect on the connection capacity. The horizontal shear stress is proportional to the vertical shear and not related to the moment in the panel so it was not necessary to test full length units.

Photo 1. Testing facility

Figure 2 shows the proposed testing configuration. Photo 2 shows the actual testing facility set up at Nelson Pine.

The required ultimate limit state (ULS) load at the support is 15.5 kN. This equated to an applied load of 31 kN. The width of web per joist is 600 mm in the panels used in the NMIT building but only 270 mm in the test units. The primary concerned was the ultimate limit state, however mid-span deflection was recorded during tests.

Each unit was loaded to failure. The load was applied at a rate of approximately 4 kN/min to 41.6 kN (ULS load multiplied by k_t, refer Appendix A for k_t derivation) and held at this load for 2 min while a visual inspection was

Table 1. Test Results

Crack opening (mm)	Test Load (kN)									
	test 3	**test 3a**	**test 4**	**test 4a**	**test 5**	**test 5a**	**test 6**	**test 6a**	**test 7**	**test 7a**
0.5	15	13.45	16.6	18.8	12.6	21.3	15.31	17.14	17.7	
1	23.4		23.47	21.1	18	23.4	19.01	21.93	21.68	8.95
1.5		21.5	26	26.4	23.42	26.1	26.37	23.58	25.5	12.7
2	34.9	26.4	29	28.9	27.3	27.3		25.96	28.13	15.51
2.5	40.5	31.1	32.44	32.5	29.6	32.6	33	28.93	33.19	20.14
3		33.4	34.8	37	33.3	34.6		31.39	35.33	23.44
3.5	41.62	36		40.6	37	37.7	37.14	34.98	41.61	29.55
4		38	39.2		39	41.6		37.18		31.3
4.5		39.3						40.3		34.41
5		41.6						41.62		37.37
5.5			40.1				41.59			38.96
6										41.63
Failed	49.74	53.81	41.3	50.05	48.25	53.92	44.71	45.13	50.23	48.11
deflection @ SLS	1	1.8	1	1.2	1.5	1	1	1.5	1.3	3

carried out. It took about five minutes to get to the ULS load. The load was then increased to failure at the same rate.

The predicted failure mode was either horizontal splitting on the web at the bottom of the fasteners or fastener withdrawal.

RESULTS

As the load was applied a split formed at the connection between the flange and the web, starting at the end of the web. The size of this 'crack opening' was measured as the load was increased to failure load. The results are as shown in Table 1.

All ten units failed by the same mechanism; flexural failure of the flange at or around the screw fasteners (refer Photo 2).

Photo 2. Unit failure

ANALYSIS

The average failure load was 48.5 kN (Std Deviation 3.89, Coefficient of variation 8.2%).

The pre-determined k_t value needed to be adjusted. The value derived in the testing procedure was based on data for tension perpendicular to the grain, whereas the failure mechanism was flexure in the skin. An adjustment was made based on the test population coefficient of variation, to 1.21. This means the ultimate limit state test load was revised from 41.6 to 37.51 kN and similarly the SLS test load changed from 23.4 to 21.1 kN.

All test results exceeded the required breaking load (37.51 kN). The average crack opening between the flange and the web at the support at the SLS test load was 1.43 mm.

DISCUSSION

It was apparent during the testing that there was an initial splitting, between the web and flange, before the fastener took up any load. This occurred at an average load of 15.69 kN. This equates to a load in the connection of 7.8 kN, approximately 10% under the maximum (unfactored) serviceability limit state loading.

Test 7 showed larger deflections than the rest of the test data. Inspection of this unit revealed that the glue joint at the support was not fully bonded. The webs were made with two pieces of 360 x 45 LVL laminated together. The edge was not redressed before the flange was glued on. It is likely therefore that one of the pieces of 360 x 45 was slightly high, halving the bond area. In the NMIT panels a single 360 x 90 will be used so this will not occur.

In the NMIT building the skin will be double the width of the test units. This will increase the stiffness and strength of the flange, subsequently increasing the ultimate capacity of the system (as the failure mechanism was bending of the flange) and reducing its flexibility, therefore reducing the separation between the flange and the web.

CONCLUSION

The proposed flange hung panel system exceeds the required capacity and exhibits acceptable deflections at serviceability limit state loading.

APPENDIX A - DERIVATION OF k_t

As the assumed failure mode is tension in the web, perpendicular to the grain the coefficient of variation has been derived from testing carried out by Andy Van Houtte on this LVL property. The results from this testing and derivation of the coefficient of variation is summarised in Table 2.

The value of k_t for 10 samples and a coefficient of variation of 15%, from Table B1 in NZS 1170, is 1.34.

Table 2. Derivation of k_t.

No.	Area (mm^2)	Ultimate load (kN)	Stress (MPa)
1	1444	1.42	0.98
2	1384	1.97	1.42
3	1357	1.46	1.07
4	1348	1.94	1.44
5	1471	1.67	1.13
6	1304	1.67	1.28
7	1384	1.56	1.13
8	1392	1.40	1.01
Ave			1.18
Std Deviation			0.18
Coeff of variation			15%

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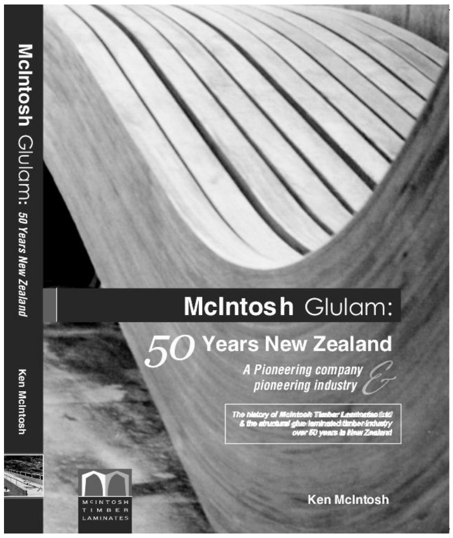

This is a unique publication for the forest and wood products industry. It covers pioneering work undertaken by McIntosh Timber Laminates (MTL), developing engineered timber – Glulam – for the New Zealand construction industry. It is as much a history of a company, as the development and promotion of a structural building product.

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* *Previously Published in e.nz magazine.*