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MODELLING WOOD STRUCTURAL PANEL PORTAL FRAME RESPONSE

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Abstract

In the 1980s, APA developed a portal frame concept, which can be site-built using standard sheathing and lumber, to create a semi-rigid moment frame. The advantage of portal frames is that they can resist relatively high lateral loads from narrow wall widths. A pair of the portal frames used for garage fronts is commonly used for prescriptive construction in the Pacific Northwest of the United States.

In the 2000s, extensive cyclic testing was conducted on this system such that design values could be determined for engineering applications. Additional prescriptive solutions were also developed, which included using the portal frames without the large holddown straps and using this portal on a raised floor system. Finally, the modern concrete codes, ACI-318 (2011), require one to consider the effects of cracked concrete on anchorage for use in areas subjected to significant seismic forces. This code requirement effectively reduced the capacities of the hold down straps in high seismic regions. Consequentially, in 2012, an additional series of full-scale wall tests were conducted by APA to confirm the effect from reduced strap capacity on the capacities of the portal frames.

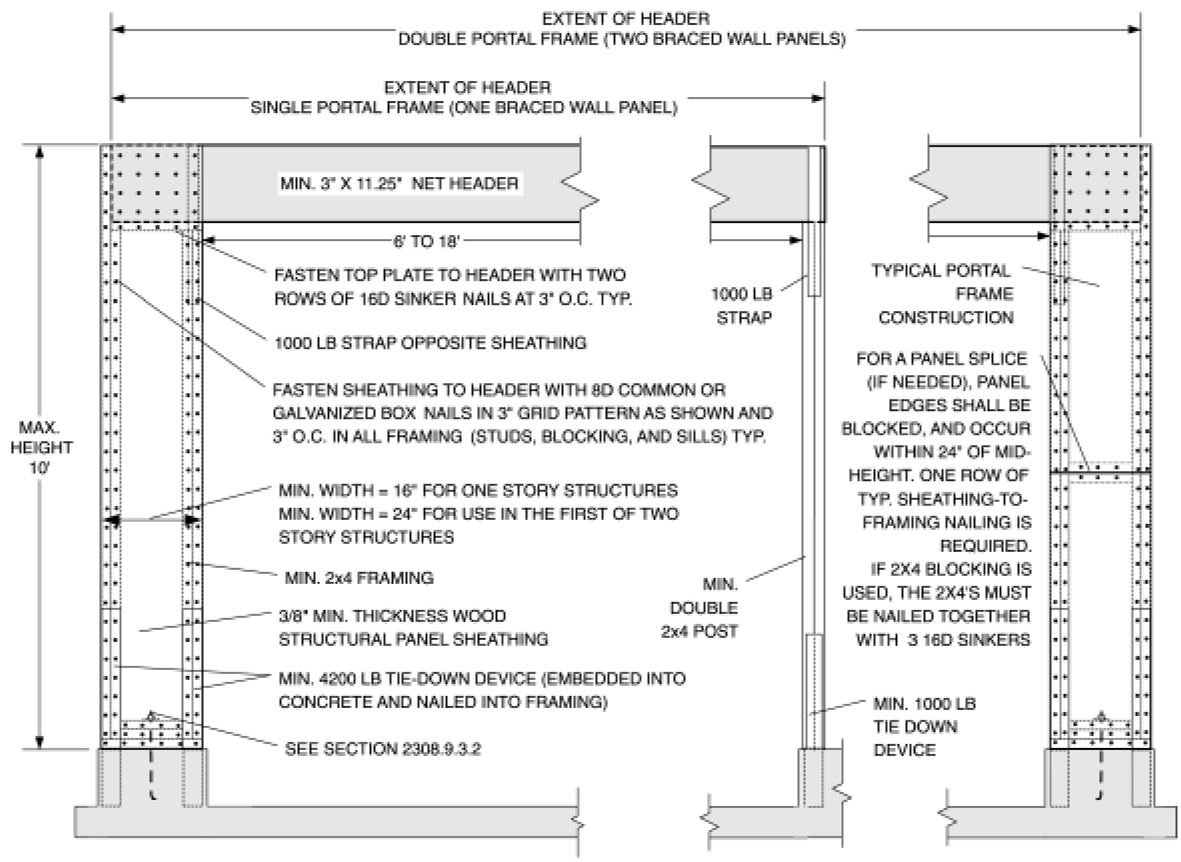
A simple principle of mechanics model was developed to predict the allowable stress design capacity of wood structural panel portal frames. Model predictions are compared to test results for 17 different portal frame configurations that have been tested throughout the years. Portal frame constructions investigated in this study range from 406 to 610 mm (16 to 24 in.) wide, 2.4 to 3.0 m (8 to 10 feet) tall, sheathed with OSB or plywood, and with no holddowns or with holddowns ranging from 3.0 to 21.2 kN (670 to 4,755 lbf) capacity at the base of the wall segment. Also investigated are portal frames built on raised wood floor assemblies with variable base of wall restraint configurations.

The paper provides a detailed theoretical basis for the model development as well as an expanded version of the table such that designers can reproduce these calculations for various portal frame configurations. The model predictions are compared to cyclic test data representing the 17 different wall assemblies. The average predicted allowable stress design capacity is within a few percent of the ultimate capacity divided by a factor of safety of 3.0 on average. The model is currently limited to predicting the capacity of portal frames. Additional refinements based on a database of cyclic test data might yield a suitable deflection prediction equation.

1. Introduction

In the 1980s, APA developed a portal frame concept, which can be site-built using standard sheathing and lumber, to create a semi-rigid moment frame, as illustrated in Figure 1. The advantage of portal frames is that they can resist relatively high lateral loads from narrow wall widths. A pair of the portal frames used for garage fronts is commonly used for prescriptive construction in the Pacific Northwest of the United States. Two widths of the portal frames, 406 mm (16 in.) and 610 mm (24 in.) and one height, 2.4 m (8 feet), were evaluated via monotonic racking tests. The general characteristics of the portal frames were as follows:

- Extended header over narrow pier
- Sheathing grid nailing in extended header to form a semi-rigid moment connection at top of pier
- Three bottom plates, which provide a semi-rigid moment connection with a grid of nails
- Hold down straps between concrete foundation and face of pier to form a semi-rigid connection.



For SI: 1 foot = 304.8 mm; 1 inch = 25.4 mm; 1 pound = 4.448 N.

FIGURE 2308.9.3.2
ALTERNATE BRACED WALL PANEL ADJACENT TO A DOOR OR WINDOW OPENING

Figure 1. Standard portal frame detail as published in 2012 International Building Code.

In the 2000s, extensive cyclic testing was conducted on this system such that design values could be determined for engineering applications. Additional prescriptive solutions were also developed, which included using the portal frames without the large holddown straps and combining the portal frames with homes that were fully sheathed, as well as using this portal on raised floor system. Finally, the modern concrete codes, ACI-318 (2011), require one to consider the effects of cracked concrete on anchorage for use in areas subjected to significant seismic forces. This code requirement effectively reduced the capacities of the holddown straps used for the engineered and prescriptive solutions for structures assigned a Seismic Design Category of C through E (based on the International Building Code). Consequentially, in 2012, an additional series of full-scale wall tests were conducted by APA to determine the effect from reduced strap capacity on the capacities of the portal frames.

2. Model Development

2.1 Overview

This paper presents a simple principle of mechanics model that was developed to predict the allowable stress design capacity of wood structural panel portal frames. The model treats the semi-rigid connections between the sheathing-to-header interface and the sheathing-to-sill plate interface as a fastener moment group. The tie-downs, when present, are treated as moment couples, adding to the capacity of the walls. The portal frame detail also uses a pier-to-header strap on the backside of the portal to increase out-of-plane stability. The addition of this strap is included in the model calculations. The model also accounts for shear capacity of the sheathing, the shear anchorage between the bottom plate and the foundation, and the shear nailing between the sheathing and the bottom plate of the walls. This model provides a method for one to calculate portal frame capacity for widths other than tested, as well as changing strap capacity.

The model was developed to predict the in-plane lateral racking strength, V, of a wood structural panel portal frame design. The general theory is provided in Equations 1-3 and Figure 2:

$$V = \text{Minimum of } V_{\text{moment couples}} \text{ and } V_{\text{shear strength}} \tag{1}$$

$$V_{\text{moment couples}} = (M_{\text{top}} + M_{\text{bottom}}) / H \tag{2}$$

$$V_{\text{shear strength}} = \text{Minimum of } v_{\text{panel}},\ v_{\text{nails}}, \text{ and } v_{\text{base connection}} \tag{3}$$

Where:

M_{top}	=	Minimum of: sheathing to header fastener moment capacity plus moment capacity due to header strap, or sheathing bending strength plus the moment capacity due to header strap
M_{bottom}	=	Holddown (tie down) strap capacity times wall width plus sheathing to sill plate nailing moment capacity
H	=	Wall height
v_{panel}	=	Wood structural panel shear-through-thickness strength
v_{nails}	=	Wood structural panel-to-framing shear capacity
$v_{base\ connection}$	=	Shear capacity due to base of wall connections to supporting structure

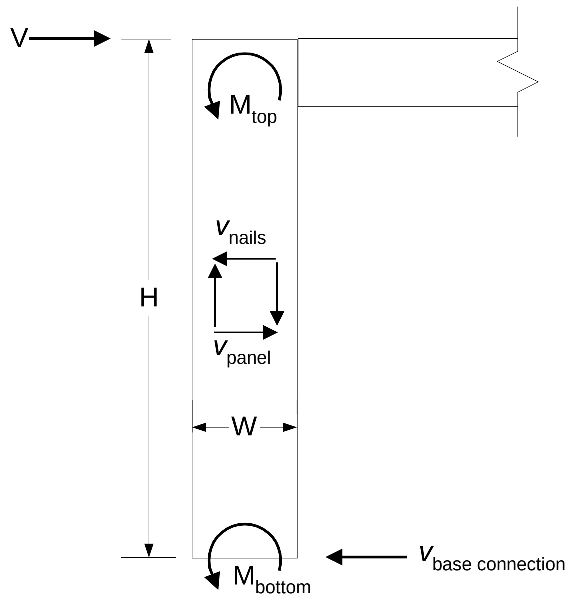


Figure 1. Principles of mechanics model to predict the strength of the wood structural panel portal frame.

2.2 Sheathing Fastener Moment Capacities

The fastener group moment capacities are calculated by first computing the polar moment of inertia of the fastener group. The single fastener allowable lateral load capacity is determined in accordance with the National Design Specification (NDS, 2012). Given the polar moment of inertia for the fastener group and the allowable single fastener lateral load capacity, the following formula is used to compute the allowable moment capacity of the connection:

$$M = Z'(J) / r \tag{4}$$

Where:

- Z' = single fastener allowable lateral load capacity per the NDS.
- J = polar moment of inertia
- r = distance to critical or average fastener.

The fastener group moment capacity can be computed using the average fastener or the critical fastener (that fastener located the furthest from the centroid of the fastener group). When using the distance to the critical fastener, the maximum moment is computed based on the assumption that the critical fastener will not exceed its allowable lateral load, and all other fasteners will be loaded to less than their allowable load.

When using the distance to the average fastener, the maximum moment is based upon a theoretical average fastener. The maximum moment of the fastener group is based on this fastener being stressed to its maximum allowable lateral load value. As a result of using the average fastener method, the moment capacity is increased at the expense of overstressing those fasteners that are further from the centroid of the fastener group than the theoretical average fastener. Because of this, it is necessary to check the load on the critical fastener to see if the computed overload can be tolerated. Given the trend in the U.S. to going to capacity design, for this paper, the average fastener method was used.

In this paper, there are 5 different fastener moment capacity cases calculated, as shown in Figure 3. A calculation example for the "(1) Header Fastener Moment" for both critical and average distances is provided in Appendix A.

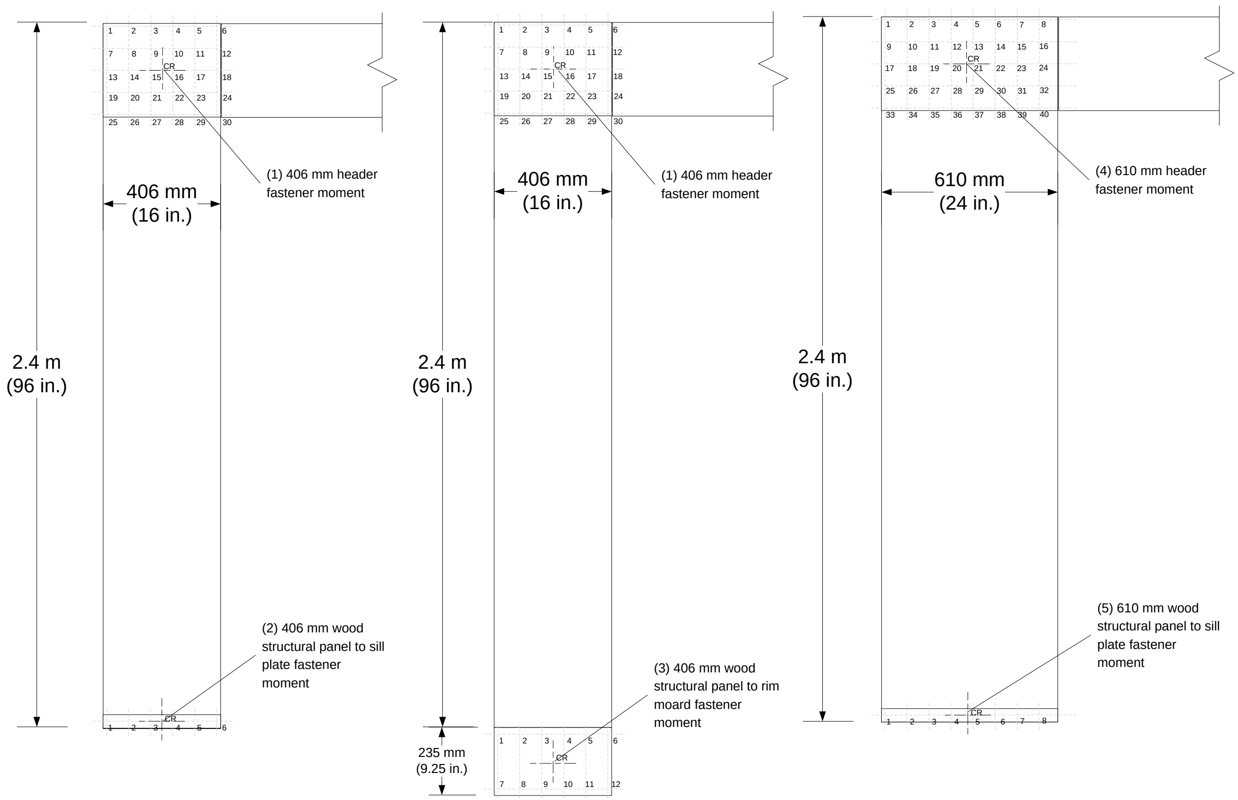


Figure 2. The five different fastener moment capacity cases

2.3 Calculation Procedure

The calculation procedure simply follows Equations 1 - 4. A complete example calculation for calculating the capacity of Wall #1 is provided in Appendix B. Material properties for the wood structural panels (plywood and OSB) are taken from the Plywood Design Specification (APA, 1998), Panel Design Specification, PDS (APA, 2012a), and APA Performance-Rated Rimboard (APA, 2009). The individual fastener properties, nails and anchor bolts are taken from the NDS (2012). The holddowns (tie downs), header strap, and other framing anchors are taken from manufacturers' catalogues at the time that the tests were conducted.

3. Calculation Results

The calculated results are completed for 17 different walls that have been tested at APA, as summarized in Table 1 (APA, 2002; 2003a; 2003b; 2004; 2006; 2012b; and 2012c). Following the calculation procedures previously described, Table 2 provides a summary of the calculated values compared to the ultimate strength values divided by 3. In this report, the factor of 3 is used as the safety factor, or margin, between ultimate strength and "allowable" design value. Safety factors ranging from 2.5 to 3 have historically been used with wood shear wall assemblies, and the value of 2.8 is currently used in the product standard PS-2 (US-DOC, 2010) for wood structural panel (WSP) shear walls. All sheathing thicknesses were either 9.5 mm (3/8 in.) or 11 mm (7/16 in.), as shown in Tables 1 and 2. Note that for Wall #10, the sheathing was 9.5 mm (3/8 in.) plywood which has an effective thickness of 3.9 mm (0.155 in.), based on the provisions of Section 2.6, and published allowable bending values from the Plywood Design Specification (APA, 1998). The OSB edgewise design values were conservatively based on the values published for rimboards (APA, 2009). The shear through thickness values for both OSB and plywood were based on the Panel Design Specification (APA, 2012)

The tabulated values for the holddown capacities for Walls 1 – 10 were based on manufacturer's literature that was current at the time of testing. For Walls 11 – 17, Simpson Strong-Tie STHD10RJ holddowns were used for all tests. The tabulated capacity of these holddowns, when all 28 nails were used, was 21.2 kN (4,755 lbf). This number was based on non-cracked concrete when used as a "mid-wall". APA did not cast these straps into concrete, hence cracking and concrete edge distances were not considered as an issue. Addition tests of these portals were conducted by varying the strap capacity by reducing the number of nails to 20 nails and 17 nails, which was intended to simulate cracked concrete in high seismic areas. By using a simple ratio of the number of nails, the tested strap capacities were 15.1 and 12.9 kN (3,400 lbf and 2,890 lbf), respectively.

The wall series tested in Walls 1 – 10 were based on the sequential phase displacement method (SEAOSC, 1997), using a first major event (FME) equal to 30.5 mm (1.2 in). Walls 11 – 17 were tested in accordance with the CUREE protocol (ASTM, 2009), with a delta equal to 61 mm (2.4 in.).

4. Discussion of Results

As shown in Table 2, the calculated results are very close to the tested results divided by a safety factor of 3 for a variety of tested boundary conditions. The predicted capacity agreed well with the tested capacities with the range of errors in predictions varied from -15% to +20%. Additional studies to this observation are being investigated. One observation is that, the wall configuration with dimensions of 610 mm x 2,440 mm (24 in. x 96 in.) was tested with four different holddown capacities (Walls 6, 13, 14 and 17). The tested lateral capacities ranged from 6.6 kN – 7.6 kN (1,476 lbf – 1,716 lbf). However, the predicted capacities ranged from 6.0 kN - 7.9 kN (1,363 lbf – 1,771 lbf). It can be observed that the wall capacities are not as sensitive to changes in strap capacity as the predictions are sensitive. This one wall configuration accounts for prediction errors ranging from -15% to +16%. It is possible that the moment at the bottom of the walls were being over-predicted, since the straps were being treated as one hundred percent effective moment couples. Due to the fact that the wood bottom plates are being subjected to compression perpendicular-to-grain, it is likely that the moment couples are indeed not fully effective. One might consider adding an empirical factor for reducing the "effectiveness" of the strap capacities, since the straps are almost certainly not one hundred percent effective. Regardless, on average, the model is providing reasonable results, and the prediction errors may not be too great for designers, especially given the large factors of safety used in adjusting the ultimate test values to allowable capacities.

Table 1. Summary of walls analyzed and APA Test Report Referenced

Wall #	Description	APA Test Report Reference
1	406 mm x 3,050 mm (16 in. x 120 in.) portal frame with 18.7 kN (4,200 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 9.5 mm (3/8 in.) OSB	T2003-11: Tests 1 and 2
2	406 mm x 2,440 mm (16 in. x 96 in.) portal frame with 18.7 kN (4,200 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 9.5 mm (3/8 in.) OSB	T2002-46: Test 3
3	406 mm x 2,440 mm (16 in. x 96 in.) portal frame with 18.7 kN (4,200 lbf) hold down, 10.7 kN (2,400 lbf) header strap, 9.5 mm (3/8 in.) OSB	T2002-46: Test 9
4	610 mm x 3,050 mm (24 in. x 120 in.) portal frame with 18.7 kN (4,200 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 9.5 mm (3/8 in.) OSB	T2003-11: Tests 3 and 4
5	610 mm x 2,440 mm (24 in. x 96 in.) portal frame with 18.7 kN (4,200 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 9.5 mm (3/8 in.) OSB	T2002-46: Test 5
6	610 mm x 2,440 mm (24 in. x 96 in.) portal frame with 18.7 kN (4,200 lbf) hold down, 10.7 kN (2,400 lbf) header strap, 9.5 mm (3/8 in.) OSB	T2002-46: Test 10
7	406 mm x 2,440 mm (16 in. x 96 in.) portal frame without hold down, 4.4 kN (1,000 lbf) header strap, 11 mm (7/16 in.) OSB	T2006-29: Test 9
8	406 mm x 2,440 mm (16 in. x 96 in.) portal frame on a raised floor with 3.0 kN (670 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 9.5 mm (3/8 in.) OSB	T2004-38: Test 8
9	406 mm x 2,440 mm (16 in. x 96 in.) portal frame on a raised floor with 235 mm (9.25 in.) WSP overlap on rim board, 4.4 kN (1,000 lbf) header strap, 9.5 mm (3/8 in.) OSB	T2004-38: Test 10
10	406 mm x 2,440 mm (16 in. x 96 in.) portal frame without hold down, 4.4 kN (1,000 lbf) header strap, 9.5 mm (3/8 in.) plywood	T2006-29: Test 6
11	406 mm x 2,440 mm (16 in. x 96 in.) portal frame with 12.9 kN (2,890 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 11 mm (7/16 in.) OSB	T2012-23 & T2012-24: Two replications
12	610 mm x 3,050 mm (24 in. x 120 in.) portal frame with 12.9 kN (2,890 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 11 mm (7/16 in.) OSB	T2012-23 & T2012-24: Two replications
13	610 mm x 2,440 mm (24 in. x 96 in.) portal frame with 21.2 kN (4,755 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 11 mm (7/16 in.) OSB	T2012P-24 Three replications
14	610 mm x 2,440 mm (24 in. x 96 in.) portal frame with 12.9 kN (2,890 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 11 mm (7/16 in.) OSB	T2012P-23 Three replications
15	406 mm x 3,050 mm (16 in. x 120 in.) portal frame with 21.2 kN (4,755 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 11 mm (7/16 in.) OSB	T2012P-24 Two replications
16	406 mm x 3,050 mm (16 in. x 120 in.) portal frame with 12.9 kN (2,890 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 11 mm (7/16 in.) OSB	T2012P-23 Two replications
17	610 mm x 2,440 mm (24 in. x 96 in.) portal frame with 15.1 kN (3,400 lbf) hold down, 4.4 kN (1,000 lbf) header strap, 11 mm (7/16 in.) OSB	Unreported Two replications

Table 2. Summary of the calculated value and the tested values for the average fastener method.

			Step 1. V based on moment couples														Step 2. V based on shear strength							Step 3.		
			M_{bottom}				M_{top}										v_{panel}		v_{nails}							
			Tie down strap		WSP to Sill		Header Fastener	Sheathing M				Header Strap					Shear through thickness		Nail	studs					Tested Lateral Capacity / 3	Compared to Tested[d]
	Width	Height		M[a]	M	M_{bottom}	M	type	Fb	t	M		M[b]	M_{top}	$V_{moment\ couples}$	Fvtv	V	Z		V	$v_{base\ connection}$	$v_{shear\ strength}$	V[c]		
#	(mm)	(mm)	(kN)	(kN-mm)	(kN-mm)	(kN-mm)	(kN-mm)	--	(kPa)	(mm)	(kN-mm)	(kN)	(kN-mm)	(kN-mm)	(kN)	(N/mm)	(kN)	(N)	#nails/m	(kN)	(kN)	(kN)	(kN)	(kN)	--
1	406	3048	18.7	6169	449	6618	2726	OSB	4137	9.5	1735	4.4	1638	3374	3.28	27.1	17.7	316	32.8	6.74	8.54	6.74	3.28	3.23	2%
2	406	2438	18.7	6169	449	6618	2726	OSB	4137	9.5	1735	4.4	1638	3374	4.10	27.1	17.7	316	32.8	6.74	8.54	6.74	4.10	3.94	4%
3	406	2438	18.7	6169	449	6618	2726	OSB	4137	9.5	1735	10.7	1735	3471	4.14	27.1	17.7	316	32.8	6.74	8.54	6.74	4.14	4.21	-2%
4	610	3048	18.7	9965	809	10774	4458	OSB	4137	9.5	3905	4.4	2542	6447	5.65	27.1	26.5	316	32.8	10.11	8.54	8.54	5.65	5.38	5%
5	610	2438	18.7	9965	809	10774	4458	OSB	4137	9.5	3905	4.4	2542	6447	7.06	27.1	26.5	316	32.8	10.11	8.54	8.54	7.06	7.43	-5%
6	610	2438	18.7	9965	809	10774	4011	OSB	4137	9.5	3905	10.7	3905	7810	7.62	27.1	26.5	316	32.8	10.11	8.54	8.54	7.62	6.56	16%
7	406	2438	0.0	0	462	462	2803	OSB	4137	11.1	2025	4.4	1638	3663	1.69	28.9	18.8	325	32.8	6.93	9.25	6.93	1.69	1.69	0%
8	406	2438	3.0	984	0	984	2803	OSB	4137	9.5	1735	4.4	1638	3374	1.79	27.1	17.7	316	32.8	6.74	8.48	6.74	1.79	1.68	7%
9	406	2438	0.0	0	1029	1029	2726	OSB	4137	9.5	1735	4.4	1638	3374	1.81	27.1	17.7	316	32.8	6.74	8.58	6.74	1.81	1.70	6%
10	406	2438	0.0	0	399	399	2419	PLY	11376	3.9	1973	4.4	1638	3611	1.64	9.3	6.0	280	32.8	5.98	9.25	5.98	1.64	1.65	0%
11	406	2438	12.9	4245	462	4707	2803	OSB	4137	11.1	2025	4.4	1638	3663	3.43	28.9	18.8	325	32.8	6.93	8.54	6.93	3.43	3.89	-12%
12	610	3048	12.9	6857	831	7688	4584	OSB	4137	11.1	4556	4.4	2542	7098	4.85	28.9	28.2	325	32.8	10.39	8.54	8.54	4.85	5.71	-15%
13	610	2438	21.2	11282	831	12113	4584	OSB	4137	11.1	4556	4.4	2542	7098	7.88	28.9	28.2	325	32.8	10.39	8.54	8.54	7.88	7.63	3%
14	610	2438	12.9	6857	831	7688	4584	OSB	4137	11.1	4556	4.4	2542	7098	6.06	28.9	28.2	325	32.8	10.39	8.54	8.54	6.06	7.15	-15%
15	406	3048	21.2	6984	462	7446	2803	OSB	4137	11.1	2025	4.4	1638	3663	3.64	28.9	18.8	325	32.8	6.93	8.54	6.93	3.64	3.05	20%
16	406	3048	12.9	4245	462	4707	2803	OSB	4137	11.1	2025	4.4	1638	3663	2.75	28.9	18.8	325	32.8	6.93	8.54	6.93	2.75	2.76	0%
17	610	2438	15.1	8067	831	8898	4584	OSB	4137	11.1	4556	4.4	2542	7098	6.56	28.9	28.2	325	32.8	10.39	8.54	8.54	6.56	6.92	-5%
																								average =	0%

(a) Hold down M = strap capacity times width - 76.2 mm

(b) Header strap moment capacity = strap capacity times width - 38.1 mm, but shall not exceed sheathing moment capacity

(c) V = minimum of V based on moment couples and V based on shear strength

(d) Comparison is: (V/tested)-1 x 100%

5. Limitations

The model presented is confirmed to be generally accurate for strength design. However, it does not include racking deflection. At present, racking deflection information can be obtained from the empirical data available in the original reports (APA, 2002; 2003a; 2003b; 2004; 2006; 2012b; and 2012c) or a deflection model could be developed. However, such a model could be rather complex.

The combined effects of vertical and lateral loads have also not been investigated in this study. It is theorized that the minimum required header stiffness "worst case" (a double 38.1 mm x 286 mm (nominal 2x12) with clear span of 5.6 m (18 ft)) provides sufficient rigidity under allowable vertical loads that it does not impart significant moment into the wall segment. On the other hand, the larger deformations associated with design lateral loads do impart moment (header fastener moment in Tables 2) into the header. Similar treatment of combined lateral and vertical loads can be seen in design information for prefabricated wood portal frame segments from Simpson Strong Tie (2012) and TrusJoist (2012).

6. Summary and Conclusion

A principle of mechanics model is presented to determine the strength of wood structural panel portal frames. Details of the calculations, including complete sample calculations are provided. The analytical model compares very well to the test results for a range of portal frame constructions.

7. Acknowledgements

The initial principle of mechanics model was developed by Zeno A. Martin, P.E., S.E. as a staff engineer at APA – The Engineered Wood Association.

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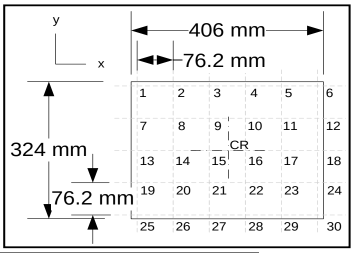
Appendix A - A calculation example for the header fastener moment

Fastener group moment capacity calculation (SI units)

Z =	325	N/nail per NDS
C_D =	1.6	
Z' =	520	N/nail
Width =	406.4	mm



Longest moment arm (r_{max}) =	244 mm	
Critical fastener moment calculation =	**1824** kN-mm	($M = Z' \times J / r_{max}$)
Average moment arm (r_{ave}) =	159 mm	
Average fastener moment arm =	**2803** kN-mm	($M = Z' \times J / r_{ave}$)
Load on critical fastener =	798 N	($Z = M \times r_{max} / J$)

Fastener	x (mm)	y (mm)	dx (mm)	dy (mm)	dx^2 (mm^2)	dy^2 (mm^2)	dx^2+dy^2 (mm^2)	r (mm)
1	0	305	-191	152	36290	23226	59516	244
2	76	305	-114	152	13064	23226	36290	191
3	152	305	-38	152	1452	23226	24677	157
4	229	305	38	152	1452	23226	24677	157
5	305	305	114	152	13064	23226	36290	191
6	381	305	191	152	36290	23226	59516	244
11	0	229	-191	76	36290	5806	42097	205
12	76	229	-114	76	13064	5806	18871	137
13	152	229	-38	76	1452	5806	7258	85
14	229	229	38	76	1452	5806	7258	85
15	305	229	114	76	13064	5806	18871	137
16	381	229	191	76	36290	5806	42097	205
21	0	152	-191	0	36290	0	36290	191
22	76	152	-114	0	13064	0	13064	114
23	152	152	-38	0	1452	0	1452	38
24	229	152	38	0	1452	0	1452	38
25	305	152	114	0	13064	0	13064	114
26	381	152	191	0	36290	0	36290	191
31	0	76	-191	-76	36290	5806	42097	205
32	76	76	-114	-76	13064	5806	18871	137
33	152	76	-38	-76	1452	5806	7258	85
34	229	76	38	-76	1452	5806	7258	85
35	305	76	114	-76	13064	5806	18871	137
36	381	76	191	-76	36290	5806	42097	205
41	0	0	-191	-152	36290	23226	59516	244
42	76	0	-114	-152	13064	23226	36290	191
43	152	0	-38	-152	1452	23226	24677	157
44	229	0	38	-152	1452	23226	24677	157
45	305	0	114	-152	13064	23226	36290	191
46	381	0	191	-152	36290	23226	59516	244
CR	190.5	152.4				J =	856450	

CR = center of rotation

dx = x distance from fastener to center of rotation

dy = y distance from fastener to center of rotation

Appendix B - A calculation example for the portal frame capacity

Wood portal frame design value capacity by analysis. Example calculation for wall #1. Sheathed with 9.5 mm OSB (SI Units).

$Width = 406$	mm
$Height = 3048$	mm
$Tie_{down.strap} = 18.7$	kN, tie down strap allowable design value
$M_{WSP.to.sill} = 449$	kN-mm, determined from fastener group moment capacity calculation
$M_{header.fastener} = 2726$	kN-mm, determined from fastener group moment capacity calculation
$Fb_{WSP} = 4137$	kPa, allowable beding strength of OSB per APA publication W345
$t = 9.5$	mm, effective thickness of wood structural panel
$Strap_{header} = 4.45$	kN, header strap allowable design value
$Fvtv = 27.1$	N/mm, panel shear through the thickness from APA publication D510
$Z = 316$	N, from 2012 NDS Table 11Q for 8d common nails and 9.5 mm OSB
$n = 32.8$	number of nails per meter (based on 2 rows spaced at 76 mm o.c.)
$v_{base.connection} = 5.338 \cdot 1.6$	kN, value from 2012 NDS Table 11E for 15.9 mm anchor bolt bearing on 3 bottom plates. The 1.6 is the load duration factor from NDS Table 2.3.2.

Step 1. Lateral load capacity, V, based on moment couples

Moment capacity at bottom of portal frame wall segment, M_{bottom}:

$$M_{bottom} = Tie_{down.strap} \cdot (Width - 76.2) + M_{WSP.to.sill}$$

$M_{bottom} = 6616$ kN-mm

Note: the 76.2 mm is subtracted to sum moment about tie down strap centerline.

Moment capacity at top of portal frame wall segment, M_{top}:

Note: the 38.1 mm is subtracted to sum moment about strap centerline.

$$M_{WSP} = Fb_{WSP} \cdot \frac{\left(t \cdot Width^2\right)}{6 \cdot 10^6} \cdot 1.6$$

$M_{WSP} = 1728$ kN-mm

$$M_{header.strap} = \min\left[Strap_{header} \cdot (Width - 38.1), M_{WS}\right.$$

$M_{header.strap} = 1637$ kN-mm

$$M_{top} = \min\left(M_{WSP}, M_{header.fastener}\right) + M_{header.strap}$$

$M_{top} = 3365$ kN-mm

Portal frame lateral load capacity based on moment couples, $V_{moment\ couples}$:

$$V_{moment.couples} = \frac{\left(M_{bottom} + M_{top}\right)}{Height}$$

$V_{moment.couples} = 3.27$ kN

Step 2. Lateral load capacity, V, based on shear strength

Panel shear capacity, v_{panel}:

$$v_{panel} = \frac{Fvtv}{10^3} \cdot 1.6 \cdot Width \qquad v_{panel} = 17.6 \quad kN$$

Nail shear capacity, v_{nails}:

$$v_{nails} = Z \cdot 1.6 \cdot n \cdot \frac{Width}{10^6} \qquad v_{nails} = 6.73 \quad kN$$

Note: the 1.6 is the load duration factor from NDS Table 2.3.2.

Portal frame lateral load capacity based on shear strength, $V_{shear\ strength}$:

$$V_{shear.strength} = \min(v_{panel}, v_{nails}, v_{base.connection}) \qquad V_{shear.strength} = 6.73 \quad kN$$

Step 3. Lateral load capacity, V, based on minimum of moment couples and shear strength

Predicted portal frame lateral load capacity:

$$V = \min(V_{moment.couples}, V_{shear.strength}) \qquad V = 3.27 \quad kN$$

Note: the average ultimate value based on testing / 3 = 3.22 kN APA Report T2003-11.