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Optimum Base Connection of Columns in Steel Gable Frames

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Abstract: *Optimum base connections were found for steel gable or portal frames with different roof slope. Five locations, Detroit MI, Miami FL, Santa Barbara CA, St. Paul MN and Berlin NH, were chosen because they represented a variety of loading conditions. Multiple structures were designed for each location with span to height ratios from 1 to 10, and varying roof angles. Analysis was done for each case using ETABS. This provided patterns in stress efficiencies that the helped identify possible optimum base connection. Precise designs with the LRFD method were performed on both pin and fixed base connection for each span length in its optimum roof slope. It was found that for all locations and all geometries fixed base connection saves weight. The average savings was 20%. Primary factors in determining base connection in these situations were magnitude of loads and length of span.*

Keywords*: Gable frame design, Pre-fabricated frames, Portal steel frames, optimization, Pitch roof steel frame and pin base connection and fixed based connection.*

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I. INTRODUCTION

Large industrial buildings sometimes require floor plans that are free of columns. This allows for flexibility in changing uses, and freedom for large equipment to move around the floor. The structures are often made with gable-shaped moment frames with a steel cross-section that varies linearly along its length. The roofs of these structures are pitched because that reduces the costs of roofing materials and reduces leaks. The bases of columns may be pinned or fixed to the foundation.

Previous researchers have investigated optimization of steel gable frames with fixed or pinned connections. Hradil et. al. [1] developed a process for optimizing gable frames with a genetic algorithm and determines that Excel can be much quicker than ABAQUS in analyzing structures. However, the research produced no general guidelines usable by designers.

Previously, it has been shown that the LRFD and ASD can produce significantly different designs of steel gable frames [2]. LRFD and ASD are based on different philosophies of design and often don't produce the same resulting design. In general, LRFD produces more economical gable designs or is nearly the same as ASD. There are few situations where ASD produces significantly lighter weight steel gable frame designs. Additionally, it has been shown that in high snow regions, the difference between the methods is more dramatic [3]. In another study [4] it has been shown the optimum roof slope for gable frames with high snow is greater than a rise to run of 7 to 12 (30.2 degrees) because considering unbalanced snow is not required above that by ASCE 7 [5]. The length of the rafter is about a quarter more for highly pitched roofs than relatively flat ones, but they are generally more efficient because of shedding of unbalanced snow.

The purpose of this study is to find the least weight design for varying spans of steel gable frames considering the slope and base connection of the columns.

The costs of fixed column bases consist of the steel connection hardware and of the reinforced concrete foundation. A variety of means can be used to fix a column base to the foundation including end plates and angle cleats [6]. Changing a pinned connection to a fixed connection requires enhancement of the foundation. The costs of this are highly related to local conditions. For example, excavation costs vary based on soil conditions. Additionally, soil capacity determines the ability to resist overturning moments.

Many studies have investigated analysis of steel gable frames. Some have analyzed even the effect of bolts upon the rotational stiffness of the fixed connection. Verma [7] and Mahendran et. al. [8] showed that the end frames have lower deflections because of the effect of cladding on the end of the gable. Therefore, analyzing and designing a central frame is more typical of the whole structure.

II. METHODS

Katanbafnezhad and Hoback [4] used five cities to illustrate how the optimum roof angle changed due to various loading conditions as shown in Table 1. The cities selected there were chosen so that they demonstrated variety in snow, wind and earthquake loads. Each of those cities is an example of a case when one or more of the loads is high. There are more examples of varying levels of snow than for the other loads. This is because it was shown that gable frame designs are sensitive to snow levels [3]. Therefore, the same cities should be suitable for finding the optimum base connection detail for pre-fabricated gable frames. See Table 1 for the list of the locations and loads used. Seismic load was not controlling (NC) for most locations.

Table 1. Locations and loads used

Location	Dead Load (psf)	Roof Live (psf)	Ground Snow (psf)	Wind Speed (mph)	Earthquake			
					S_s	S_1	F_a	F_v
Detroit, MI	20	20	20	90	NC	NC	NC	NC
St. Paul, MN	20	20	50	115	NC	NC	NC	NC
Berlin, NH	20	20	100	108	NC	NC	NC	NC
Miami, FL	20	20	0.00	170	NC	NC	NC	NC
Santa Barbara, CA	20	20	0.00	93	2.19g	0.79g	1	1.7

The column height and space between frames were a constant 20 feet for all cases. The gable spans were 20, 60, 100, 150, and 200 feet because wind load changes with the ratio of span to height and span to width. Roof angles were 10, 15 and 30.3 degrees because of changes in the wind and snow loads with the roof angle and these slopes were previously shown as an optimum roof slope for these locations [4]. For determining the unbraced length of the compression flange, the space between purlins was assumed to be 3 feet. See the frame geometry in Figures 1 and 2. One typical frame in the middle was designed and is shaded in Figure 2. The total width of the structure needed to be set at a constant value for consistency. This building width influences the calculation of the leeward side wind load. There was no particular reason to pick one width over another so 20' was chosen which the width of only one segment is. A checkbyKatanbafnezhad & Hoback [2] found that the result only varied by about 1% when the width was changed, so in most cases the results are not significantly dependent on it. The definitions of the cross-sections are shown in Figure 3. The foundation is in Figure 4.

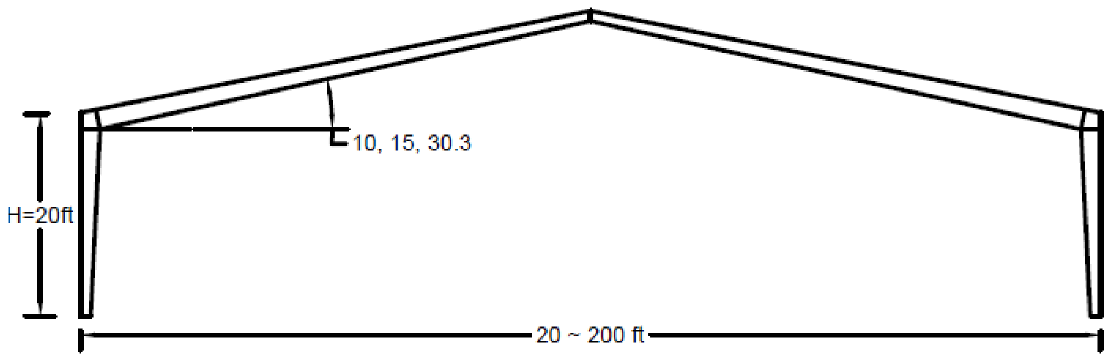

Fig. 1. Typical Frame

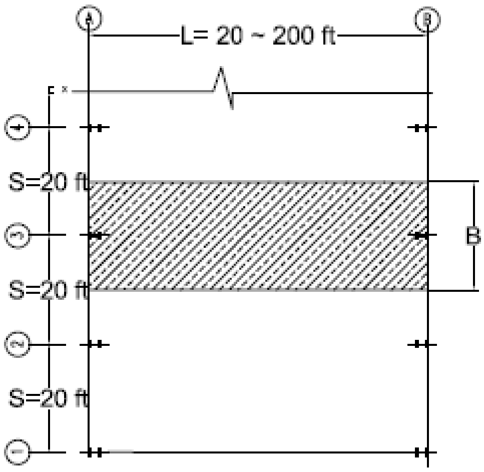

Fig. 2. Plan View

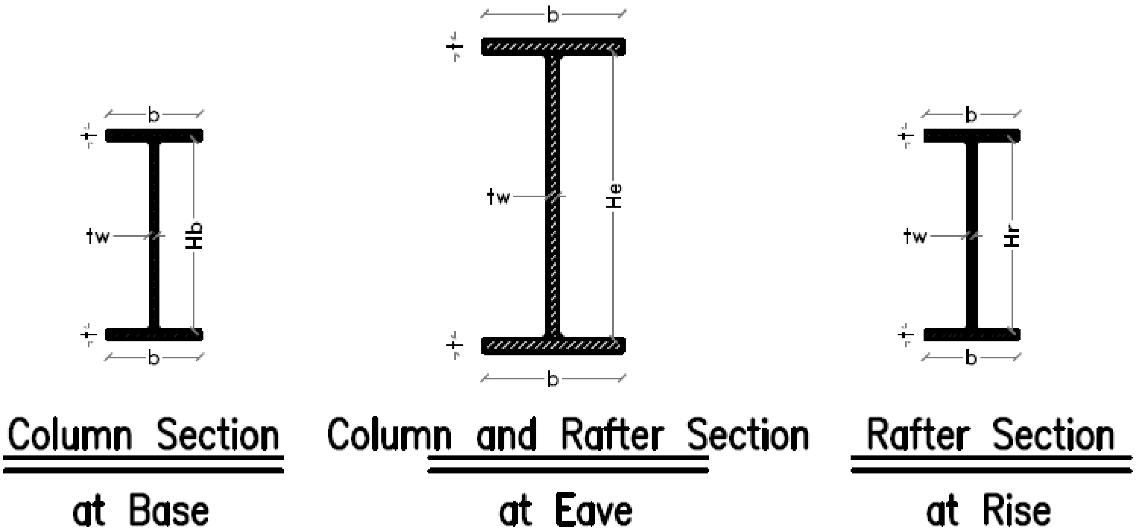

Fig. 3. Definition of Cross-Sections

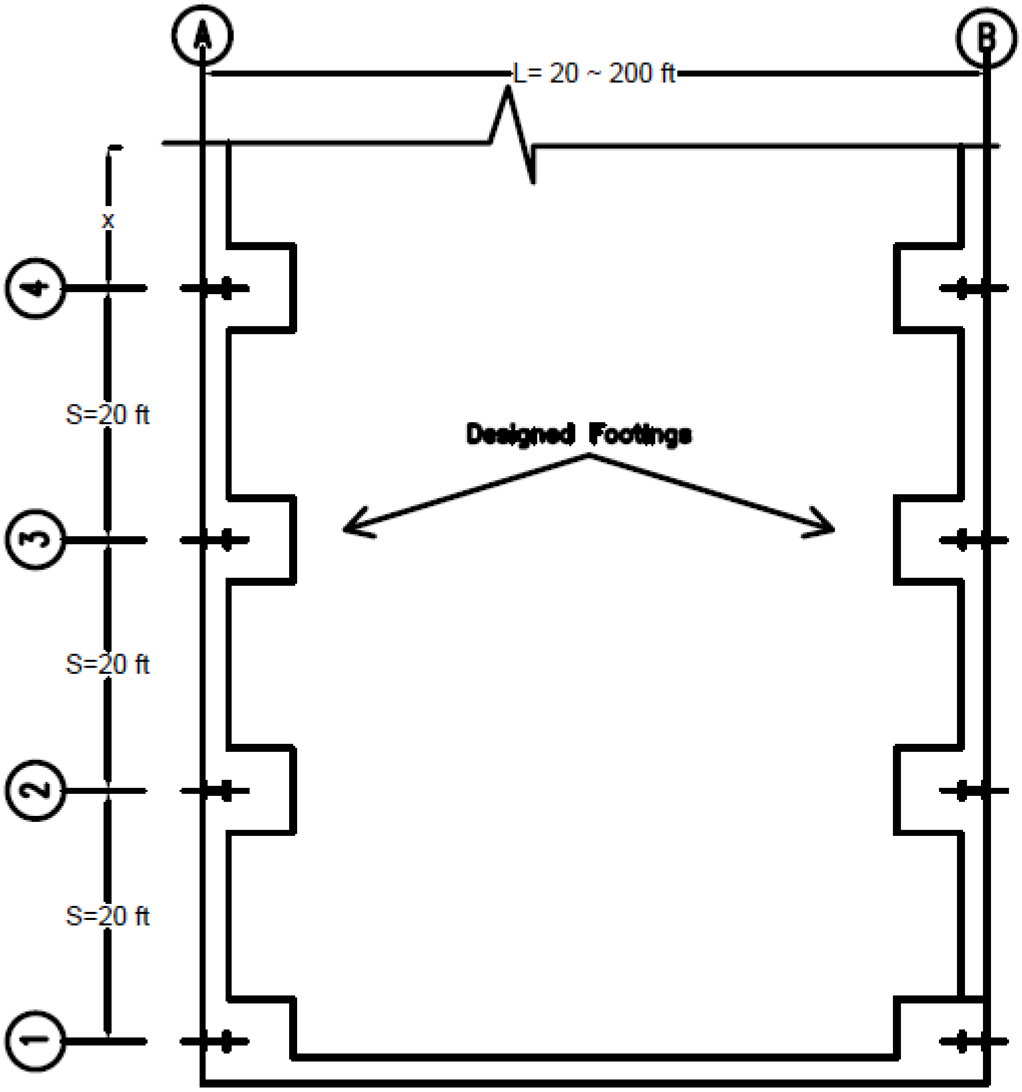

Fig. 4. Foundations

To follow common fabrication practice, the column and rafter were defined as non-prismatic member when base connection was pin. Webs are assumed be linearly tapered and flanges are assumed be constant. Therefore, the major axis moment of inertia will vary non-linearly in the column and rafter. Columns were chosen prismatic when their bases were fixed.

The members were designed to keep stress ratio in the members close to one and satisfy lateral frame displacement limit of 0.025h. Also, all section satisfied compact section limitation according to AISC [9]. Therefore, there are two different sections were used for pin and fixed base connection for each case.

The moment frame considered was as an ordinary moment frame. The site class and seismic design category were assumed to be D. The importance factor was taken at 1. The surface roughness category was considered exposure C. The roof slope condition assumed was an unobstructed slippery surface and considered as a warm roof. For wind loading, the directional procedure was used from ASCE 7-16 [5]. Site Class D is used for each city so that comparisons between locations can be made. However, conditions at actual project sites may vary from the hypothetical. Additionally, Exposure C was used for wind, but that doesn't mean the predominant exposure in the area is that type. The direct method was used in frame analysis. The ASTM standard A572 high- strength steel, grade 50 has been used for design members (Fy=50 ksi,Fu=65 ksi, ,Fye=55 ksi,,Fue=71.5 ksi,,E=29,000 ksi).

For designing members, AISC 360-16 was used [9]. Analysis and design has been done by ETABS 17.

For three locations, Miami FL, Santa Barbara CA and Berlin NH analysis and design of the foundation has been done. These locations represent high earthquake load, wind load and high snow load.

ACI 318-14 was used for foundation design. Concrete compressive strength, f'c, assumed be 4 ksi. Fy and Fu for foundation's rebar are 60 and 90 ksi respectively (ASTM A615 Grade 60). Soil allowable stress for all cases and locations assumed be 2 ksf. Similar to frame design, foundation of one of the middle frames considered for investigation and design. Reported rebar's weight and concrete volume in tables 3 to 5 are for 2 single footings. Minimum depth of footing assumed be 3.5'. Analysis and design of foundation has been done by SAFE 2016.

III. RESULTS

All combinations of roof angle and span ratio were analyzed. The stress utilization based on the initial size is shown in Table 2 for Detroit. Katanbafnezhad and Hoback [3] found that the stress ratio was a very accurate predictor of the lowest weight design. Katanbafnezhad and Hoback [4] found that when two similar angles have the same stress utilization, the lower angle is optimal because higher angles have higher lengths and material weights. Since the length difference is not significant but usually only a couple percentages the stress utilization alone is a good indicator of the optimum weight roof slope.

Pinned and fixed connections in Table 2 have the same optimum roof angle for all span ratios. It is the same for all other locations for this paper.

Table 3 to 7 show the results for each city. Confirming previous studies, it was seen that higher roof angles are preferred in regions with high snow.

According to the tables, for all cases having fixed base columns gives lower weight of overall structural steel (column, rafter and rebar.) The biggest difference in weights is for Miami with a L/H =1. In this case, the strong wind loads caused high lateral displacements. To limit that, the moment of inertia of the members had to be increased. However, with the fixed case, lateral displacements were lower and the members were controlled by strength.

In many locations, gravity loads are more significant than lateral loads. However, unbalanced snow load can cause lateral deflections. Several scenarios show that the frames with fixed bases are 20% more efficient than frames with pinned bases. A fixed base column is more efficient to carry moment loads because the moment diagrams are distributed more evenly and because lateral displacements are reduced. When the base connection is pinned the moment is zero at base and is maximum at top however, when the base connection is fixed the moments are more equal at the base and the top.

The percentage of weight savings varies significantly between cases. Part of the reason for the variation is that all designs are made practical by rounding to realistic available sections. Therefore, the results have a small unpredictable variation.

IV. CONCLUDING REMARKS

With the current code and methods, generally for all cases columns with a fixed base have the least weight design of pre-fabricated gable frames compared to pinned base columns. The fixed options were between 14 to 42 percent lower cost for a range of structures representing a profile of the United States. Generally, the most common spans of gables have savings of 20% if a fixed base is used. Foundations that provide fixity require more concrete and rebar, but the overall amount of steel is decreased. The cost of concrete is insignificant compared to that of steel, so having fixed bases reduces the total cost of building.

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Table 2. Stress ratios for all scenarios in Detroit. pg= 20psf, wind=90mph

L/H	θ = 10	θ = 15	θ = 20	θ = 25	θ = 30	θ = 35	θ = 45
1	0.043	0.046	0.053	0.058	0.064	0.068	0.082
	0.048	0.050	0.058	0.063	0.070	0.075	0.089
1.5	0.061	0.064	0.076	0.082	0.089	0.095	0.113
	0.059	0.062	0.073	0.082	0.091	0.099	0.121
2	0.091	0.094	0.109	0.118	0.125	0.134	0.156
	0.088	0.091	0.105	0.111	0.117	0.128	0.158
2.5	0.129	0.131	0.151	0.161	0.168	0.179	0.206
	0.129	0.129	0.148	0.155	0.160	0.168	0.200
3	0.183	0.179	0.200	0.210	0.216	0.228	0.259
	0.178	0.177	0.198	0.205	0.208	0.217	0.245
3.5	0.247	0.239	0.255	0.264	0.268	0.281	0.315
	0.234	0.231	0.254	0.259	0.261	0.270	0.295
4	0.320	0.305	0.321	0.321	0.324	0.337	0.374
	0.298	0.291	0.316	0.318	0.317	0.325	0.352
4.5	0.400	0.378	0.392	0.383	0.383	0.396	0.435
	0.369	0.356	0.382	0.380	0.376	0.384	0.410
5	0.487	0.455	0.467	0.447	0.444	0.457	0.497
	0.446	0.426	0.451	0.445	0.438	0.444	0.471
7.5	1.004	0.900	0.885	0.815	0.773	0.773	0.799
	0.905	0.822	0.837	0.799	0.765	0.757	0.762
10	1.641	1.421	1.358	1.225	1.120	1.110	1.117
	1.464	1.283	1.295	1.189	1.120	1.092	1.070

Table 3. Preferred column base connections for Miami, FL

Member	Dim (in)	L/H									
		1		3		5		7.5		10	
		pin	fixed	pin	fixed	pin	fixed	pin	fixed	pin	fixed
Column	Hb	7	15	9	22	18	33	13.5	40	18	45
	Ht	25	15	30	22	33	33	45	40	45	45
	b	7	6.5	8.25	7.5	8.5	9	11	9	15	10
	t	0.5	0.375	0.5	0.5	0.75	0.5	0.75	0.75	1	1
	tw	0.375	0.25	0.375	0.25	0.375	0.375	0.5	0.5	0.5	0.5
Rafter	He	22.5	15	30	22	33	33	45	40	45	45
	Hr	8.5	4	6	6	18	9	13.5	8	18	15
	b	4	4	5	6	8.5	5	10	7	15	10
	t	0.25	0.25	0.375	0.375	0.5	0.5	0.75	0.75	1	1
	tw	0.25	0.25	0.375	0.25	0.375	0.375	0.5	0.5	0.5	0.5
Lateral Dis (in)		6.00	2.46	6.01	1.53	5.99	1.41	5.72	1.31	5.87	1.06
Roof Slope		10		10		15		15		15	
Frame's Weight (lbs)		2146	1473	4206	3412	9328	7430	19851	16117	38384	30384
Rebar's weight (lbs)		126	126	176	240	353	632	952	1140	1928	3074
Concrete Volume(ft^3)		63	63	63	63	112	112	328	568	686	776
Total steel Weight		2272	1599	4382	3652	9681	8062	20803	17257	40312	33458
Saved weight %		42.1%		20.0%		20.1%		20.5%		20.5%	
Preferred Base Support		Fix		Fix		Fix		Fix		Fix	

Note: For 200 ft span with fixed connection, higher allowable stress for soil is needed(qall> 2.5 ksf)

Table 4. Preferred column base connections for Santa Barbara, CA

Member	Dim (in)	L/H									
		1		3		5		7.5		10	
		pin	fixed	pin	fixed	pin	fixed	pin	fixed	pin	fixed
Column	Hb	6	11.5	8.5	22	9	33	10	33	15	35
	Ht	22	11.5	27	22	35	33	35	33	45	35
	b	6	5.25	7.5	7	9	8.25	10	13	15	15
	t	0.375	0.375	0.5	0.5	0.5	0.5	1	0.75	1	1
	tw	0.25	0.25	0.375	0.25	0.5	0.375	0.5	0.375	0.5	0.5
Rafter	He	15	10	22	22	33	33	35	33	45	35
	Hr	4	4	8	8	9	6	10	10	15	15
	b	4	3	7	5	9	5.25	10	10	15	15
	t	0.25	0.25	0.5	0.5	0.5	0.5	1	0.75	1	0.75
	tw	0.25	0.25	0.25	0.25	0.375	0.375	0.5	0.375	0.5	0.5
Lateral Dis (in)		5.81	2.73	4.41	0.87	3.13	0.45	2.75	0.31	1.79	0.27
Roof Slope		10		15		15		15		15	
Frame's Weight (lbs)		1384	1136	3972	3271	8133	6706	20228	16344	36915	30400
Rebar's weight (lbs)		195	227	195	226	416	533	874	1700	1330	3150
Concrete Volume(ft^3)		63	63	63	63	140	140	140	270	240	280
Total steel Weight		1579	1363	4167	3497	8549	7239	21102	18044	38245	33550
saved weight %		15.8%		19.2%		18.1%		16.9%		14.0%	
preferred Base Support		Fix		Fix		Fix		Fix		Fix	

Table 5. Preferred column base connections for Berlin, NH

Member	Dim (in)	L/H									
		1		3		5		7.5		10	
		pin	fixed	pin	fixed	pin	fixed	pin	fixed	pin	fixed
Column	Hb	6	20	8	22.5	12	32	15	33	15	30
	Ht	26	20	33	22.5	33	32	45	33	45	30
	b	6	6.25	8	7.5	8.5	8.75	9.5	8.5	15	12
	t	0.5	0.375	0.5	0.5	0.75	0.5	0.75	0.75	1	1
	tw	0.375	0.25	0.375	0.25	0.375	0.375	0.5	0.5	0.5	0.5
Rafter	He	22.5	14	30	22.5	33	27	45	28.5	45	24
	Hr	4	4	6	6	12	12	18.5	15	15	15
	b	4	4	4	3.25	8.25	6	5	7	15	15
	t	0.25	0.25	0.25	0.375	0.5	0.375	0.75	0.75	1	1
	tw	0.25	0.25	0.375	0.25	0.375	0.375	0.5	0.375	0.5	0.375
Lateral Dis (in)		2.89	0.55	5.81	1.41	5.92	1.06	5.85	1.31	5.97	1.51
Roof Slope		10		30.3		30.3		30.3		30.3	
Frame's Weight (lbs)		2026	1609	4278	3187	9409	7451	16599	13655	40946	34052
Rebar's weight (lbs)		145	179	300	353	500	550	610	800	1178	1430
Concrete Volume(ft^3)		63	63	112	112	200	216	240	336	378	630
Total steel Weight		2171	1788	4578	3540	9909	8001	17209	14455	42124	35482
saved weight %		21.4%		29.3%		23.8%		19.1%		18.7%	
preferred Base Support		Fix		Fix		Fix		Fix		Fix	

Table 6. Preferred column base connections for Detroit, MI

Member	Dim (in)	L/H									
		1		3		5		7.5		10	
		pin	fixed	pin	fixed	pin	fixed	pin	fixed	pin	fixed
Column	Hb	8	8	9	20	9	33.5	9	33.5	12	38
	Ht	22.5	8	33.5	20	33.5	33.5	33.5	33.5	45	38
	b	6.25	6	8	8	9	9	11	10	12	10
	t	0.375	0.375	0.5	0.5	0.75	0.5	1	1	1	1
	tw	0.25	0.25	0.375	0.25	0.375	0.375	0.375	0.375	0.5	0.5
Rafter	He	14	8	24	20	33	31	11	30	45	33.5
	Hr	4.5	4	7	5	9	10	33.5	8	12	11
	b	4.5	4	6	5	9	5	11	8	12	11
	t	0.25	0.25	0.5	0.5	0.5	0.5	1	1	1	1
	tw	0.25	0.25	0.25	0.25	0.375	0.375	0.375	0.375	0.5	0.375
Lateral Dis (in)		5.33	3.07	4.04	1.01	3.44	0.64	3.73	0.65	3.83	0.73
Roof Slope		10		15		15		15		15	
Frame's Weight (lbs)		1435	1111	4213	3480	8882	7390	20099	16619	31648	26293
Preferred Base Support		Fix		Fix		Fix		Fix		Fix	
Saved weight (%)		29.16%		21.06%		20.19%		20.94%		20.37%	

Table 7. Preferred column base connections for St. Paul, MN

Member	Dim (in)	L/H									
		1		3		5		7.5		10	
		pin	fixed	pin	fixed	pin	fixed	pin	fixed	pin	fixed
Column	Hb	6	10	9	22.5	16	27	15	28.5	30	30
	Ht	22.5	10	33.5	22.5	33	27	45	28.5	45	30
	b	5.75	6	8	8	8.5	9	10	10	13	12
	t	0.5	0.375	0.5	0.5	0.75	0.5	0.75	0.75	1	1
	tw	0.25	0.25	0.375	0.25	0.375	0.375	0.5	0.375	0.5	0.5
Rafter	He	22.5	10	24	22.5	33	27	45	28.5	45	24
	Hr	4	4	6	6	16	9	20	13.5	30	15
	b	4	4	6	6	7.5	7	8	10	11	15
	t	0.25	0.25	0.5	0.5	0.5	0.5	0.75	0.75	1	1
	tw	0.25	0.25	0.375	0.25	0.375	0.375	0.5	0.375	0.5	0.375
Lateral Dis (in)		5.12	3.07	3.43	0.74	6	1.58	5.85	1.68	6.03	1.28
Roof Slope		10		15		30.3		30.3		30.3	
Frame's Weight (lbs)		1580	1209	4621	3868	9709	8003	20463	16980	37660	28850
Prefeed Base Support		Fix		Fix		Fix		Fix		Fix	
Saved weight (%)		30.69%		19.47%		21.32%		20.51%		30.54%	