American Journal of Engineering Research (AJER)
e-ISSN: 2320-0847 p-ISSN : 2320-0936
Volume-9, Issue-6, pp-160-168
www.ajer.org
Research Paper Open Access

Pre-Fabricated Gable Frame Design in High Snow Regions-Comparison of LRFD and ASD

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ABSTRACT: *The LRFD and ASD methods do not produce the same designs. The biggest discrepancies are in cold regions with a high snow load because of different ways of handling the safety factors for snow. The results were shown for structures in Berlin, NH and Government Camp, OR, USA which are located in high snow regions. Precise practical designs of gable frames with a high snow load show that LRFD and ASD can produce results that vary by up to 41% with LRFD usually producing the lighter weight design for roof angles above a pitch of 7 to 12. Results were a function of other loads, roof angles, and span to height ratios. The implication is that designers in cold regions need to either be familiar with how the methods vary, or design with both methods in order to find the least weight design.*
KEYWORDS: *Gable frame design, Pre-fabricated frames, ASD, LRFD, tapered member, snow.*

Date of Submission: 28-05-2020 Date of acceptance: 14-06-2020

I. INTRODUCTION

Structural load prediction involves probabilistic analysis of likelihoods of naturally occurring phenomena such as snow, seismic events and wind. The building codes account for variability by applying overload factors that relate an expected load to a maximum load at a recurrence interval. For a snow load, the maximum expected snow for a 50 year recurrence is used [1]. The theories of how design codes handle this uncertainty differently will be discussed fully below.

Complicating the calculation of the design load is that the possibility of multiple occurrences of other maximum loads needs to be considered. Maximum roof live load will not occur at the same time as snow load because roof live load occurs during roof maintenance which is not possible when a roof is covered with snow. Wind and snow sometimes occur together during blizzards, but it is unlikely that the 50 year maximum for both would be coincident.

Failure of structures under snow load is more likely when the weight of snow has been underestimated, but making structures resilient so that they can withstand small localized failure without collapsing is a way to prevent many failures [2, 3].

Therefore, good design is important as well as having accurate predictions of snow load. ASCE 7-16 leaves many areas of the U.S. without estimated snow loads [1]. In those regions, a snow study needs to be performed. Comparisons of methods to conduct this sort of study have been shown by Bean et.al. [4].

The possibility of global climate change increasing snow load should be considered. However, McCauley et. al. haven't noticed significant changes in snow fall [5].

Balanced and unbalanced snow loads are the two different cases of snow load acting on sloped roofs in snowy regions. Balanced snow is placed uniformly on all the spans in roof, but unbalanced snow is not a uniform distribution of this load because of how wind loads move snow from the windward side to the leeward side of a roof. According to the code, unbalanced snow need not be considered for a roof angle less than 2.38 and greater than 30.2 degrees. It has been found that in the case of gable frame roofs that are low sloped, code requirements for snow might be missing an important situation of drifting on the eaves [6]. However, consideration of unbalanced snow loading partly matches this load situation.

Different types of load cases have been defined by codes to consider the effects of balanced and unbalanced snow on the structures [1]. These load cases may control the design if the amount of the snow load compared to roof live load is significantly different. Previous work [7] has shown that in locations with heavy snow the snow load cases control design in all geometries for both the Load and Resistance Factor Design

(LRFD) and Allowable Stress Design (ASD) methods [1]. Although, both methods are similar in many aspects, they are based on different philosophies of design and often don't produce the same resulting design. In some situations, such as extreme loading or geometry, there are significant differences between them. For example, this was previously investigated in the area around Lake Tahoe, NV that has a ground snow of 5.75 kPa (120 psf) [7]. In that case, based on stress levels, the LRFD method was up to 30% more efficient in the design of Pre-Fabricated gable frames. Since regions with high snow have been identified as places where the design methods significantly disagree, then this should be investigated further. Additionally, the first study was based only on stress differences found between the two methods. A detailed design should be done to validate that variations in stress ratios result in weight differences, and this will ensure that all other code requirements are met such as lateral drift, deflection, and seismic design limits.

The purpose of this study is to investigate differences between ASD and LRFD in regions with extreme snow loading for Pre-Fabricated Gable Frames.

II. COMPARISONS BETWEEN ASD AND LRFD

ASD and LRFD are two design methods for steel structures. ASD is older, and LRFD has been developed within the past 30 years. Since these two methods have differences in how they handle safety factors, then under varying loading conditions and geometry either method could be preferred because of relative weight savings. The relationship between design strength and applied load for ASD method is expressed as follows:

$$\frac{\emptyset R_n}{\gamma_i} \geq \sum Q_i \qquad (1)$$

Where:
$\emptyset$ =Resistance factor
γ_i =Overload Factor
R_n =Nominal resistance
Q_i =Load

An assumption with this method is that all loads have the same variability. With this concept in mind, the entire variability of the loads, γ, is placed on the strength side of Equation 1.

For the LRFD method, the relationship between design strength and applied load is expressed as follows:

$$\emptyset R_n \geq \sum \gamma_i Q_i \qquad (2)$$

The simple statement of this equation would be, the design strength, $\emptyset Rn$, provided by the resulting design must be at least equal to the sum of the applied factored service loads, $\sum \gamma_i Q_i$. The subscript i refers to each type of load such as Dead, Live, Earthquake and Wind. The term γ_i depends on the variability of the load.

Another difference between these two methods appears in a load's factor. See Table 1 for load combinations for ASD and LRFD. The table is according to ASCE but it is expanded to cover all possible loading situations like balanced and unbalanced snow load, wind suction or compression [1].

As shown in Table 1 some load combinations have no correspondence in other methods; however, some combinations have similarities. Comparison between similar load combinations can be done by formulations.

In regions with high snow load, the load cases for snow become much more likely to govern for gable frame design. According to ASCE, unbalanced snow load (S_{ub}) only needs to be considered for roof pitches less than 7 on 12 which equals 30.2 degrees [1]. Therefore, for roof angles over 30.2 degrees, LRFD and ASD are likely to be controlled by cases 4D and 6D in Table 1, respectively. The notation for Table 1 is in the appendix. Algebraic manipulation is done below so that these cases can be easily compared. Bending moments control in gable frames, so the resistance factor for flexure is used. For LRFD:

$$1.2D + W_c + 0.5S_b = 0.90R_n \qquad (3)$$

Dividing both sides by 0.90 gives:

$$1.333D + 1.111W_c + 0.556S_b = R_n \qquad (4)$$

For ASD:

$$D + 0.45W_c + 0.75S_b = \frac{R_n}{1.67} \tag{5}$$

If both sides are multiplied by 1.67:

$$1.67D + 0.751W_c + 1.252S_b = R_n \tag{6}$$

Dividing Equation 4 by Equation 6, and dividing the top and bottom by 1.67D gives the ratio of LRFD to ASD strengths required:

$$\beta = \frac{0.8 + 0.66\frac{W_c}{D} + 0.33\frac{S_b}{D}}{1 + 0.45\frac{W_c}{D} + 0.75\frac{S_b}{D}} \tag{7}$$

When the roof angle is low, then unbalanced snow load is more likely to control, and the controlling cases are likely to be 3C and 3B for LRFD and ASD, respectively. Those cases are compared below. For these load cases β can be formulated as:

$$\beta = \frac{0.8 + 0.3323\frac{W_c}{D} + 1.064\frac{S_{ub}}{D}}{1 + \frac{S_{ub}}{D}} \tag{8}$$

Table 1. ASCE 7-16 Expanded Load Combination as Analyzed

LC	LRFD	LC	ASD
1	*1.4D*	**1**	*D*
2A	$1.2D+0.5L_r$	**2**	*D+L*
2B	$1.2D+0.5S_b$	**3A**	$D+L_r$
3A	$1.2D+1.6Lr+0.5W_s$	**3B**	$D+S_{ub}$
3B	$1.2D+1.6L_r+0.5W_c$	**4A**	$D+0.75L_r$
3C	$1.2D+1.6S_{ub}+0.5W_c$	**4B**	$D+0.75S_b$
4A	$1.2D+W_s+0.5L_r$	**5A**	$D+0.6W_s$
4B	$1.2D+ W_c +0.5L_r$	**5B**	$D+0.6W_c$
4C	$1.2D+ W_s +0.5S_b$	**6A**	$D+0.45W_s+0.75L_r$
4D	$1.2D+W_c+0.5S_b$	**6B**	$D+0.45W_c+0.75L_r$
5A	$0.9D+W_s$	**6C**	$D+0.45W_s+0.75S_b$
5B	$0.9D+W_c$	**6D**	$D+0.45W_c+0.75S_b$
6	$1.2D+E_h+0.2S_b$	**7A**	$0.6D+0.6W_s$
7	$0.9D+E_h$	**7B**	$0.6D+0.6W_c$
		8	$D+0.7E_h$
		9	$D+0.525E_h+0.75S_b$
		10	$0.6D+0.7 E_h$

If β is greater than 1.0, then ASD produces a lower stress design and is likely to be the most efficient design process. The opposite is true when β is less than 1.0. Equation 7 and 8 rely upon other loads beyond the snow load. Therefore, β can't be found from only the snow load, but it is necessary to know or assume the ratios of W_c/D, and S_b/D or S_{ub}/D.

Additionally, since Equations 7 and 8 are based on flexure controlling, when other internal forces like shear or axial loads are controlling, the equations will have to be modified.

Despite the difficulty in applying equations 7 and 8, some general observations about the equations can be made. When a balanced snow load controls, in Equation 7 the ratio S_b/D is weighted 0.75 and 0.33 by ASD and LRFD, respectively. That means that if wind and dead load were insignificant, LRFD could produce a design with 0.33/0.75 = 0.44 as much stress. Therefore, it could be roughly twice as efficient. When an unbalanced snow load controls, in equation 8 the ratio S_b/D is nearly the same for LRFD and ASD. Therefore, β depends more on relationships between the loads. In the results below it will be shown for different geometries (roof angle and span) which method is preferred. So, realizing which method is preferred would provide a more economical solution. This is not easy without the exact analysis and design of the structure.

III. METHODS

First, locations with high ground snow are selected. Then, Pre-Fabricated Gable Frames are analyzed using LRFD and ASD for roof angles of 10 to 45 degrees and span to height rations (L/H) of 1 to 10. In the second part, cases are selected to be redesigned with new sections for column and rafter and total weight of frame computed and compared.

It was determined that two locations would be necessary to illustrate how variation in commonly controlling loads influences whether ASD or LRFD would be preferred for design of pre-fabricated gable frames. Locations were selected because they had high ground snow. They were Government Camp, OR and Berlin NH. The seismic load was not controlling for these locations. Since Government Camp, OR is in an area that would require a case study for the wind load, the wind speed was assumed be 209 km/h (130 mph). The snow load there is 15.37 kPa (321 psf). In Berlin, NH, the snow and wind loads are 4.79 kPa (100 psf) and 174 km/h (108 mph), respectively. Both sites had a dead and roof live load of 0.96 kPa (20psf) each. Despite having very high snow loads, it is possible that with certain geometries, other loads might control, so all load cases will be checked.

It was necessary to have many gable spans because wind load changes with the ratio of span to height and span to width. The gable spans were in 6.1 m (20 ft) increments up to 61m. Roof angles were 10, 15, 20, 25, 30, 35 and 45 degrees because of changes in the wind and snow loads with angle. The column height and space between frames were a constant 6.1 m (20 ft) for all cases. The column base support was hinged. For determining the unbraced length of the compression flange, the space between purlins was assumed to be 0.91 m (3 ft). 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 building does not significantly change the wind load in gable frames, so it was set to a constant value of 6.1 m (20 ft) for consistency. A check found that the result only varied by about 1% when the width was changed.

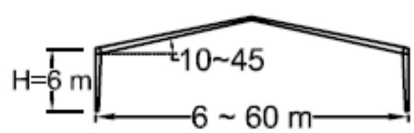

Fig.1. Typical Frame

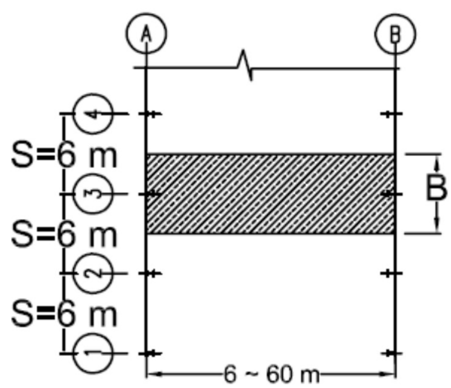

Fig.2. Plan View

To follow the common fabrication practice for gable frames, columns and rafters are defined as non-prismatic members. Webs are assumed to be linearly tapered and flanges are assumed to be constant. Therefore, the major axis moment of inertia will vary non-linearly in the column and rafter. The members are the same for all locations and spans in Part One. The non-prismatic sections were defined the same for all geometries and locations in Part One. The initial cross-sections were the largest at the eave for both the column and rafter, and less at the base and ridge. The flanges were all 38.1 cm (15 in) with thickness of 2.54 cm (1 in). The webs varied from 35.6 cm (14 in) at the base and eave to 76.2 cm (30 in) at the eave with a thickness of 0.953 cm (0.375 in). In Part Two, which is redesigning a selected case, sections were changed and defined to keep the stress ratio in the members close to 1 to satisfy the lateral frame displacement limit.

The frame was 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 [1]. Site Class D was used for each city so that comparisons between locations could 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. Moments in columns were expected to govern since the frames do not have significant axial loads from cranes or other attachments. Consequentially, axial analysis and determining the K Factor bear little on the final result. The ASTM standard A 572 high- strength steel, grade 50 has been used for design members Fy = 345 MPa (50 ksi), Fu = 448 MPa (65 ksi), Fye = 379 MPa (55 ksi), Fue = 493 MPa (71.5 ksi).

For designing members AISC 360-16 was used [8].Analysis and design was done by ETABS 17.

IV. RESULTS

Table 2 shows detailed results for Government Camp, OR for a roof slope of 25 degrees. The height is constant at 6.1 m (20 ft). Only the controlling load cases were shown. A total of 280 analyses were performed for Part One of study, so this level of detail can't be shown for each.

Table 2. Detailed Results for Government Camp, OR at 25 degrees

L/H	Method	Stress Ratio and controlling LC				Ave. Stress	β
		Column	LC	Rafter	LC		
1	ASD	0.127	6D	0.114	6D	0.121	1.18
	LRFD	0.158	3C	0.135	3C	0.147	
2	ASD	0.401	3B	0.371	3B	0.386	1.1
	LRFD	0.445	3C	0.414	3C	0.43	
3	ASD	0.819	3B	0.781	3B	0.8	1.07
	LRFD	0.881	3C	0.843	3C	0.862	
4	ASD	1.299	3B	1.278	3B	1.289	1.06
	LRFD	1.375	3C	1.36	3C	1.368	
5	ASD	1.828	3B	1.832	3B	1.83	1.05
	LRFD	1.919	3C	1.937	3C	1.928	
6	ASD	2.382	3B	2.29	3B	2.336	1.05
	LRFD	2.498	3C	2.409	3C	2.454	
7	ASD	2.965	3B	2.872	3B	2.919	1.05
	LRFD	3.102	3C	3.014	3C	3.058	
8	ASD	3.565	3B	3.473	3B	3.519	1.04
	LRFD	3.724	3C	3.637	3C	3.681	
9	ASD	4.176	3B	4.087	3B	4.132	1.04
	LRFD	4.357	3C	4.273	3C	4.315	
10	ASD	4.797	3B	4.71	3B	4.754	1.04
	LRFD	4.999	3C	4.919	3C	4.959	

In Equations 7 and 8, the expected controlling load cases were compared to find the variable β. However, in some geometries and loadings, other cases control. Therefore, β is generalized to compare between whichever load combination controls. A value above one means LRFD is more expensive than ASD by that ratio, so ASD is preferred, and conversely when the value is less than one.

Tables 3 to 4 list the controlling load combination by method for each location and geometry. The parameter β indicates which method is preferred and by how much. It is seen that when the roof angle is below 30.2 degrees, where unbalanced snow needs to be considered, ASD usually provides for lower stress in the roof members since β is above 1. However, as seen in Table 4, LRFD sometimes controls for long flat roofs if their snow load is not extreme. Between 30 and 35 degrees there is a change in controlling load cases and in the β ratio of relative efficiency.

Comparing the locations in Tables 3 and 4, it is seen that higher snow for Table 3 causes ASD to be more strongly favored for low sloped roofs, and LRFD for higher sloped roofs. This illustrates how the differences between the methods are more prevalent in extreme loadings.

In almost every analysis, snow controlled the design. However, Berlin, NH had only 4.79 kPa (100 psf) of snow load, and with high roof angles Table 4 shows that Case 3B controls in the LRFD method. That means that live load controls there. This occurs because with high pitch, unbalanced snow need not be considered, and balanced snow is less on sloped roofs.

Table 3. Beta and Controlling Load Cases for Government Camp, OR

L/H	method	θ = 10	θ = 15	θ = 20	θ = 25	θ = 30	θ = 35	θ = 45
1	ASD	6D	6D	6D	6D	6D	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.06	1.11	1.15	1.18	1.21	0.91	1
2	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.09	1.08	1.11	1.1	1.11	0.71	0.8
3	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.04	1.05	1.06	1.07	1.08	0.65	0.75
4	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.03	1.04	1.05	1.06	1.07	0.63	0.73
5	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.02	1.03	1.04	1.05	1.06	0.61	0.71
6	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.03	1.03	1.04	1.05	1.03	0.62	0.7
7	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.03	1.03	1.04	1.05	1.05	0.61	0.71
8	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.03	1.03	1.04	1.04	1.05	0.68	0.72
9	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.03	1.03	1.03	1.04	1.05	0.63	0.69
10	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.03	1.04	1.03	1.04	1.06	0.62	0.72

The snow controlled the gable design when the sloped roof snow was around twice as high as the roof live load per area. The previous study found that when snow was above 5.75 kPa (120 psf), it controlled for all geometries [7], but in this study it is seen that it doesn't always control at 4.79 kPa (100 psf). Therefore, the snow dominates starting at about 5.75 kPa (120 psf) so it could be labeled as being a snow-controlling region.

V. DESIGN

In Part 2 of the study the selected cases are redesigned with non-prismatic sections. Two cases are selected for further investigation based on the results from Table 3. The first case is L/H =1 with a roof slope of 30 degrees. In this case, ASD gives a lower stress by about 21 percent. The second case is L/H =7 with a roof slope of 35 degrees. In this case, LRFD gives approximately 39 percent less stress.

Table 5 shows the resulting sections for efficient design. Cross-section sizes were chosen with engineering judgement based on practical sizes available. In English units the dimensions are whole inches for widths and thicknesses are commonly available. Table 6 shows the results of the analysis. Up to a 3% overstress was allowed in the final designs. The frame weight is from two beams and two columns. The lateral displacements were checked and ruled acceptable for each controlling wind load case using a limit of 0.025h.

Results of redesigning selected cases in Part 2 of this study shows that the stress ratio comparison in part one is approximately accurate and according to that, designers could expect actual results similar to that comparison.

VI. CONCLUSION

With current methods, generally LRFD produces more economical designs in high snow regions. However, there are some situations such as with a roof angle less than 30.2 degrees where ASD can be preferred. With a roof angle of less than 30.2 degrees, the results of both methods are similar to each other, but with ASD controlling more commonly. ASD was found to give up to a 21 percent lighter weight design. For roof angles greater than 30.2 degrees, LRFD was found to give up to a 41 percent lighter weight design. The variation depended upon the level of the snow load and geometry. The significant differences between the methods show that in cold regions designers have a strong economic advantage to design structures using both methods and choosing the best. These differences are present in structures in other regions too, but are not as dramatic.

Table 4. Beta and Controlling Load Cases for Berlin, NH

L/H	method	θ = 10	θ = 15	θ = 20	θ = 25	θ = 30	θ = 35	θ = 45
1	ASD	6D	6D	6D	6D	6D	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.05	1.02	1.02	1.03	1.03	1.05	1.11
2	ASD	6D	6D	6D	6D	6D	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.02	1.04	1.04	1.04	1.04	0.84	0.94
3	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	4D	4D
	β	1.01	1.01	1.03	1.06	1.07	0.78	0.88
4	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	3B	3B
	β	0.99	0.99	1.01	1.03	1.05	0.76	0.86
5	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	3B	3B
	β	0.98	0.99	1	1.02	1.03	0.76	0.86
6	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	3B	3B
	β	0.98	0.98	1	1.01	1.03	0.76	0.86
7	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	3B	3B
	β	0.97	0.98	0.99	1.01	1.02	0.76	0.86
8	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	3B	3B
	β	0.97	0.97	0.99	1	1.02	0.76	0.86
9	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	3B	3B
	β	0.97	0.97	0.99	1	1.01	0.76	0.86
10	ASD	3B	3B	3B	3B	3B	6D	6D
	LRFD	3C	3C	3C	3C	3C	3B	3B
	β	0.96	0.96	0.99	1.01	1.02	0.76	0.86

VII. NOTATION

The following symbols are used in this paper:

β: ratio of strengths required for LRFD versus ASD.
Ø: Resistance factor
γ_i: Overload Factor
R_n: Nominal resistance
Q_i: Load
F_y: yield stress
B: frame width
b: plate width
t: plate thickness
h: plate height

D: Dead Load
L: Live Load
L/H: frame span over column height
L_r: Roof Live Load
S_b: Balanced Snow Load
S_{ub}: Unbalanced Snow Load
W_c: Wind Load compression on windward roof
W_s: Wind Load suction on windward roof

Table 5. Final section for selected cases in Part 2 of the study

θ	L/H	Method	Member	Position	Flange		Web	
					b (cm)	t (cm)	h (cm)	t (cm)
30	1	ASD	Column	Base	20.3	1.3	20.3	0.6
				Eave	20.3	1.3	50.8	0.6
			Rafter	Eave	20.3	1	40.6	0.6
				Ridge	20.3	1	20.3	0.6
		LRFD	Column	Base	20.3	1.9	20.3	0.6
				Eave	20.3	1.9	38.1	0.6
			Rafter	Eave	20.3	1.3	38.1	0.6
				Ridge	20.3	1.3	20.3	0.6
35	7	ASD	Column	Base	45.7	3.2	45.7	1.3
				Eave	45.7	3.2	101.6	1.3
			Rafter	Eave	45.7	3.2	101.6	1.3
				Ridge	45.7	3.2	45.7	1.3
		LRFD	Column	Base	30.5	2.5	50.8	1.3
				Eave	30.5	2.5	114.3	1.3
			Rafter	Eave	30.5	2.5	114.3	1.3
				Ridge	30.5	2.5	50.8	1.3

Table 6. Final design results for selected cases in Government Camp, OR

L/H	θ	Method	Column			Rafter			Frame weight (kg)	Preferred method	Less weight
			Weight (kg)	Stress Ratio	LC	Weight (kg)	Stress Ratio	LC			
1	30	ASD	346	1.03	6D	157	1.03	6D	1006	ASD	20.80%
		LRFD	447	0.983	3C	189	1.01	3C	1271		
7	35	ASD	1810	1.03	3B	7731	1.03	3B	19081	LRFD	41.10%
		LRFD	1209	0.856	4D	4409	0.83	4D	11235		

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Naser Katanbafnezhad & Alan Hoback. "Pre-Fabricated Gable Frame Design in High Snow Regions- Comparison of LRFD and ASD." *American Journal of Engineering Research (AJER),* vol. 9(06), 2020, pp. 160-168.