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Abstract

This paper introduces an archetype mid-rise cold-formed steel (CFS) building that aids in assessing the limits of current structural solutions, particularly lateral force resisting systems, and also in the development of new CFS technologies. A unified archetype building provides a platform for comparing the performance of new lateral force resisting systems to existing ones. The study herein provides quantitative evaluation of the design limitations of a typical "complete" cold-formed steel building (i.e. only cold-formed steel based elements are used for all gravity and lateral force resisting systems) at different heights (4 through 20 stories) located in a high seismic zone. The primary focus is the seismic force resisting system, which is limited to shear wall systems detailed in AISI specifications. The archetype buildings are designed using ASCE7-10 for all required loads and load combinations; and the CFS framing systems are designed utilizing AISI specifications, particularly AISI-400-15. Limitations in the application of current specifications for designing mid-rise cold-formed steel buildings are provided, and the potential for further studies discussed.

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Introduction

Cold-formed steel (CFS) buildings are an effective solution for low and mid-rise structures (Schafer 2008, 2011). Robust structural and non-structural performance, as well as ease, efficiency, and economy of CFS construction are all favorable characteristics for mid-rise construction. However, the potential of CFS systems has not been fully realized in the building industry at this time, especially for commercial and multi-family residential applications where CFS solutions are at their most efficient.

Currently, CFS framing provides both the gravity and lateral load resisting system for low-rise buildings, but as building heights rise, other materials are often used for the lateral load resisting system such as reinforced masonry or concrete shear walls, mostly as core shear walls around the elevators or stair cases. Introducing multiple trades into the construction process can reduce the favorability and efficiency of CFS construction. Accordingly, a full archetype building using only CFS, representative of commercial and multi-family residential buildings, is needed to assess the limits of current structural solutions, particularly lateral load resisting systems. The archetype can also aid in the development and evaluation of new CFS technologies. New technologies may increase the performance of CFS buildings, and enable these building to be wholly constructed by systems similar to the ones indicated in the AISI S100 (general specification), S240 (framing design specification), and S400 (seismic specification). A unified CFS archetype building is essential for comparing the performance of new lateral load resisting systems to existing ones, and also to assess the limitations of current design methods and solutions.

To address these needs, an archetype building, representative of commercial and multi-family buildings, is selected. The building dimensions and loading assumptions are provided in detail to establish a unified suite of archetype buildings. The considered heights of the archetype building are 4, 6, 8, 10, 12, 15, 18 and 20 stories. These heights are selected in order to find the limits of current design and to shed light on different aspects of mid- and high-rise CFS structural design; including: shear capacity of the walls, stiffness or drift, chord stud and diaphragm design, hold-down and ties, and anchor rods. The height limit of the archetype building is reported based on each design limit state and the potential to improve the existing design methods or available construction details are discussed.

Archetype Building

A unified archetype building needs to be the representative of a particular construction method. Different architectural forms and performance requirements for buildings can result in quite complicated architectural shapes that may not be suitable for an archetype building. Accordingly, archetype buildings are typically simple buildings in term of geometrical shape, but they still represent a large number of buildings using a particular construction method. As an example, the full-scale two story archetype CFS building in the CFS-NEES project was designed as a small low-rise commercial building (see Fig. 1) with wholly CFS gravity and lateral load resisting systems, including ledger framing, lipped channel joists, OSB sheathed shear walls, built-up lipped channel chord studs, and OSB sheathed floors (see Fig. 2) (Schafer et al., 2014; Peterman et al., 2014; Peterman 2015). The designed buildings were subjected to extreme earthquake loads on a shake table with and without non-structural components, including non-structural sheathing (i.e. gypsum boards), interior drywalls, stairs, and exterior envelope (Peterman 2014).

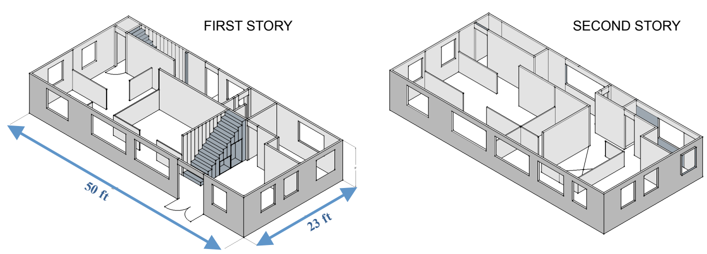


Fig. 1. Architectural drawings of the two-story archetype building in CFS-NEES. (Peterman 2014)

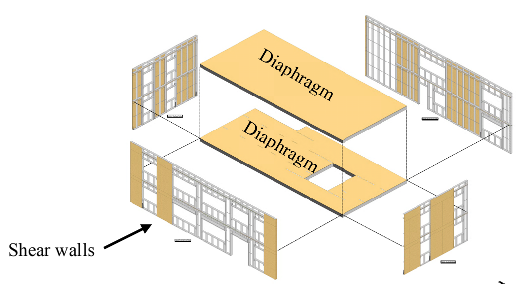
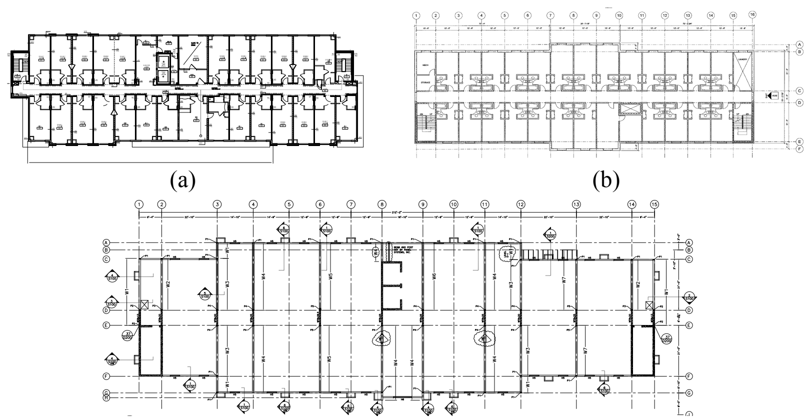


Fig. 2. Structural system of the two-story archetype building in CFS-NEES. (Peterman 2014)

Since the CFS-NEES building is relatively small (dimensions were about 50 ft × 23 ft in plan) to be considered as a mid-rise building (i.e. up to 20 stories or about 180 ft), a search has been performed to find larger candidates for the unified archetype building. Accordingly, a family of buildings has been found including hotels, residential buildings, and some commercial buildings that share a typical architectural plan. The plan includes repetitive rooms on both sides of a long hall way, two stair cases at the ends of the building, and a central elevator, as shown in Fig. 3. All perimeter walls, walls between rooms, and walls of the hallway are suitable places for placing gravity walls and lateral force resisting systems including shear walls or strap bracing.



(a) (b) (c)

Fig. 3. Typical hotel and residential building plans. (Courtesy of Nabil Rahman, DSi Engineering and Panel Systems Inc.)

Accordingly, a similar building plan is also provided in Example-1 of the IBC SEAOC Structural/Seismic Design Manual Vol. 2: Four story wood light-frame structure (IBC, 2012) and has been adopted as a typical plan applicable for CFS construction (see Fig. 4).

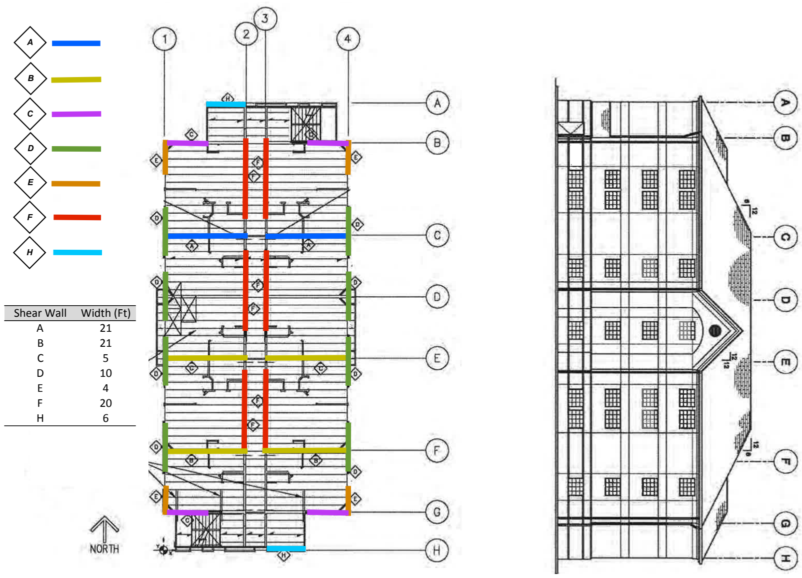


Fig.4. The unified archetype building plan, elevation and shear wall layout.

The buildings designed in this study (based on the unified archetype building) are sited in Irvine, CA (site class D) and are 116 ft × 48 ft in plan with a typical story height of 9.44 ft (Note, the original example for the archetype is a four-story building). To design the buildings with different amounts of stories, the following parameters were presumed in accordance to ASCE 7 (ASCE 2010): Importance Factor, I_e=1, Acceleration Parameter at short periods, S_s=1.39, Spectral Response Acceleration Parameter at a period of 1s, S_l=0.5, Short Period Site-Coefficient, F_a=1 and Long-Period Site Coefficient, F_v=1.5.

In general, the structural details of the building including lateral force resisting system (LFRS), and gravity framing is selected to be similar to the CFS-NEES archetype building. Accordingly, ledger framing is assumed and the LFRS mainly consists of Type I OSB (7/16 in.) sheathed shear walls, as designated in Fig. 4. Each shear wall is anchored by hold-downs at the ends only on the foundation, and ties or strap at floor levels are used to provide chord stud continuity. The parameters R (Response Modification Coefficient), Ω_o (Over-Strength Factor) and C_d (Deflection Amplification) were determined to be 6.5, 3 and 4, respectively, per ASCE 7 Table 12.14-1 as Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets. The maximum structural height of this type of building is 65 ft, which is equivalent to a 7-story building. Notably, the height limitation is not enforced in this study to find the limitations of current solutions. The effective seismic weight was based on the estimated weights of roof, floor and exterior walls (see Tables 1 and 2). A 1200 lb allowance for rooftop MEP has been included per each 2 stories (i.e., 2400 lb for a 4-story building).

Table 1. Unit Weight of the dead load and live load

Roof Weights:		**Floor weights:**	
Roofing + re-roof	5.0 psf	Flooring	1.0 psf
Sheathing	2.5	Lt.wt. concrete	14
Trusses	3	Sheathing	2.8
Insulation + sprinklers	2.5	I-joist	4.0
2layers gyp + misc	7	2 layers gyp + misc	8.2
Dead load	**20.0 psf**	**Dead load**	**30.0 psf**
Live load	**20.0 psf**	**Live load**	**40.0 psf**

Table 2. Effective seismic weight of the 8 story building

			Dead Load (included in the effective weight)				Live Load (Not included)	
Level	Height of each floor	Assembly	Unit Wt (psf)	Area (ft^2)	Weight (kips)	Story Wt (kips)	Unit Wt (psf)	Story Wt (kips)
Roof	14.75 (to the centroid of the roof)	Roof	20	5288	105.8	**157.3**	20	105.76
		Ext wall	15	1350	20.3		-	
		Int wall	10	2644	26.4			
		Rooftop MEP			4.8		-	
Typical floor Floor	9.44	Floor	30	5288	158.6	**258.0**	40	211.52
		Ext wall	15	3100	46.5		-	
		Int wall	10	5288	52.9		-	

Note: The vertical part of the wall in the last floor is assumed to be 8.25'. Half of the interior and exterior walls assigned to the upper floor and half to the lower floor.

In this study, design of the gravity load framing system has not been explicitly included. However, certain steps have been provided to satisfy the important seismic requirements of the LFRS; including: shear wall analysis and design for shear demands, controlling the lateral drift of the structure; design of chord studs, hold-downs, and anchor bolts for the applied demands; and analysis and design of the diaphragms. Notably, gravity load effects do need to be considered, and are considered, in chord stud and hold-down demands. All elements have been designed using LRFD load combinations in ASCE 7 and LRFD design methods in AISI-S100 and AISI-S400. Due to the symmetric geometry of the structure and for simplicity in the analysis, accidental eccentricity has not been considered in the archetype design, although it is mandated in ASCE 7. The accidental eccentricity would modestly change shear demands on the shear walls far from the rigidity center of the building and would need to be considered in the future.

Table 3. Shear wall stiffness of the first floor of the 8-story building

			Stiffness portion (%)					
Axis line	**Shear wall**	**b (ft)**	**k_1**	**k_2**	**k_3**	**k_4**	**Stiffness (lb/in.)**	**Relative stiffness of shear walls in the story (%)**
A	H	6	9.2	17.4	46.0	27.4	6621	4.4
B	C	5	9.0	14.3	49.7	27.0	5436	3.7
C	A	21	7.5	49.9	20.2	22.4	18976	12.7
E	B	21	7.5	49.9	20.2	22.4	18976	12.7
F	B	21	7.5	49.9	20.2	22.4	18976	12.7
G	C	5	9.0	14.3	49.7	27.0	5436	3.7
H	H	6	9.2	17.4	46.0	27.4	6621	4.4
1	E	4	8.8	11.1	53.9	26.2	4222	2.0
1	D	10	9.1	28.7	35.2	27.0	10900	5.1
2	F	20	7.7	48.5	21.0	22.8	18419	8.6
3	F	20	7.7	48.5	21.0	22.8	18419	8.6
4	E	4	8.8	11.1	53.9	26.2	4222	2.0
4	D	10	9.1	28.7	35.2	27.0	10900	4.0

Note: Stiffness potion of the walls. k_1: Cantilever effect, k_2: Sheathing shear deflection, k_3: Nonlinear deflection, k_4: Anchors deflection. k_1 to k_4 are representing four terms added together in AISI-S400-15 Eq. E1.4.1.4-1.

Design of OSB Sheathed Shear Wall Systems

OSB sheathed Type I shear walls (E1.3.1.1 in AISI-S400) have been sheathed on either one or both sides, and detailed with hold-down and anchors at each end of the wall segment. To distribute the lateral force between shear walls, the relative stiffness of the shear walls are estimated in Table 3 using the design deflection method provided in AISI-400-15 section E1.4.1.4. The lateral force at each story level is distributed between the walls based on the associated relative stiffness.

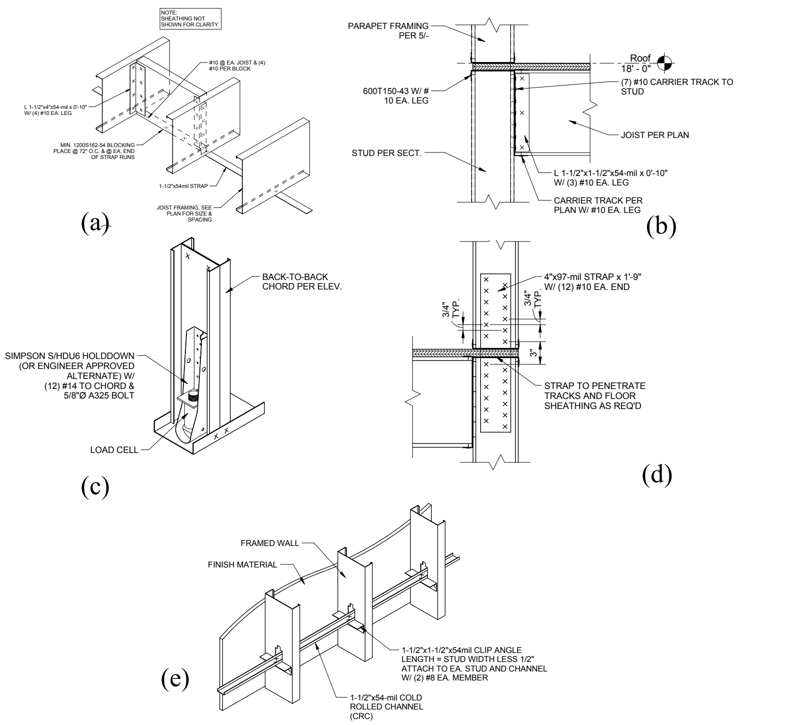


Fig. 5. Typical details of the CFS-NEES archetype building applicable to the unified archetype building: (a) Joist blocking and strapping detail; (b) Ledger-frame construction method; (c) Built-up chord studs and hold-downs; (d) Chord stud ties at the story level; (e) Bridging detail of the wall studs (Peterman 2015, Madsen et al. 2011).

To meet strength requirements, the maximum thickness of the chord-studs and top and bottom tracks are assumed 97 mil, and 68 mil, respectively. However, in many cases lower thickness can be satisfactory for the design. To provide the required shear strength of the shear walls, different perimeter fastener spacing was selected (see Table 4). For instance, for perimeter #10 fasteners at 2 inches on center, the nominal shear strength of a one-sided CFS framed shear wall with 7/16" OSB sheathing and appropriately sized chord studs is 3080 (lb/ft), where the thickness of the studs and tracks are more than 68mil per Table E1.3-1 of AISI-S400 to provide the required chord stud capacity as explained later (see Table 7). It should be noted that the chord stud thickness requirements sometimes contradicts the strength requirements and further investigation is required due to the limitation it places on current design. The capacity of the wall can be increased to two fold of the current capacity by adding a similar sheathing to the other side of the wall. All shear walls in this study are selected as having sheathing on both sides. As shown in Table 4, the ratio of $v/(\phi v_n)$ (shear demand to nominal shear strength) for all buildings and all different shear walls is satisfactory, using nominal shear capacities in the design specification. Notably, the design could be more optimized for some walls, but the results shows the basic applicability of the current design method for design of the archetype buildings.

Table 4. $v/(\phi v_n)$ ratio of the first floor

Axis	Wall	W (ft)	Thickness of OSB Sheathing (Number of sheathing)	Story							
				4	6	8	10	12	15	18	20
				(number of sheathing)×V_n (lb/ft)							
				(2) 1230	(2) 1850	(2) 1850	(2) 2310	(2) 2310	(2) 2310	(2) 3080	(2) 3080
				Fastener spacing at panel Edges (in)- Screw #10							
				6	4	4	3	3	3	2	2
A	H	6	(2) 7/16"	0.69	0.75	0.97	0.84	0.89	0.95	0.79	0.77
B	C	5	(2) 7/16"	0.67	0.70	0.96	0.83	0.88	0.94	0.79	0.77
C	A	21	(2) 7/16"	0.56	0.58	0.80	0.69	0.73	0.78	0.66	0.64
E	B	21	(2) 7/16"	0.56	0.58	0.80	0.69	0.73	0.78	0.66	0.64
F	B	21	(2) 7/16"	0.56	0.58	0.80	0.69	0.73	0.78	0.66	0.64
G	C	5	(2) 7/16"	0.67	0.70	0.96	0.83	0.88	0.94	0.79	0.77
H	H	6	(2) 7/16"	0.69	0.72	0.97	0.84	0.89	0.95	0.79	0.77
			(2) 7/16"								
1	E	4	(2) 7/16"	0.23	0.25	0.34	0.30	0.31	0.34	0.28	0.28
1	D	10	(2) 7/16"	0.52	0.54	0.74	0.64	0.67	0.72	0.60	0.59
2	F	20	(2) 7/16"	0.40	0.41	0.56	0.49	0.52	0.56	0.47	0.46
3	F	20	(2) 7/16"	0.40	0.41	0.56	0.49	0.52	0.56	0.47	0.46
4	E	4	(2) 7/16"	0.23	0.25	0.34	0.30	0.31	0.34	0.28	0.28
4	D	10	(2) 7/16"	0.52	0.54	0.74	0.64	0.67	0.72	0.60	0.59

According to ASCE 7 the seismic story drift shall be limited to 0.025h for this type of structure, where h is the story height. Drift was determined based on AISI 400-15, including the drift resulting from cantilever actions of the wall, shear deformation of the sheathing, nonlinear deformation of the wall resulted from fastener nonlinear behavior, and hold-down and anchor deformation. The resulting lateral drift is amplified by C_d (Deflection Amplification) and compared to the 0.025h, per ASCE 7. As, shown in Table 5, all archetype buildings can satisfy the drift limitations using the current design methods. The provided drift ratios in Table 5 are calculated based on the stiffness of shear walls as provided in Table 3 for the 8-story building (as a sample). However, the available methods may not consider the actual behavior of a tall cantilever wall, where the stiffness has been separately calculated for each story and improvements may be needed in design for this case.

Table 5. Drift ratio for archetype buildings

Number of Stories	4	6	8	10	12	15	18	20
Maximum Drift	0.011	0.012	0.017	0.015	0.016	0.017	0.015	0.015

Chord Stud Design

Chords studs are primarily designed for axial load demands, including gravity loads and tension/compression induced by lateral demands, particularly earthquake in this study. Using sheathing on one side of the chord studs will result in eccentric axial loads demands and chord studs need to be designed for combined axial load and bending moments. However, this eccentric bending moment need not be considered, when both sides of the wall are sheathed.

Chord studs are assumed to be back-to-back lipped channels, and the maximum practical size of the chord stud is considered to be a (rather large) back-to-back 800S259-97 (AISI-S200-12 designation). For higher demands it is common to use more studs packed together, although the behavior and the load paths for stud packs are not well studied. In addition, the choice to allow up to 800S studs implies wall thickness that may require architectural changes from current practice, but are intended to illustrate the potential of such deeper studs.

Table 6 summarizes chord stud demand analysis and design for the first story of the 4-story building. Chord studs of the first story include the gravity and seismic forces of the above stories. The chord studs have been designed for LRFD load combinations and also for expected seismic load combinations. For all chord studs, the expected seismic load combination governed the design.

Table 6. Chord stud demand analysis and design of the first story of the 4-story building (Note: gravity and seismic forces include th effect of the above stories)

Axis	SW	$P_n{}^1$ (kip)	v (lb/ft)	v_n (lb/ft)	P_{seis} (kip)	P_{DL} (kip)	P_{LL} (kip)	$P_u{}^2$ (kip)	$P_{u\text{-}amp}{}^3$ (kip)	$P_{u\text{-}exp}{}^4$ (kip)	LRFD $P_u/\phi P_n$	Expected Min (3,4) / P_n
A	H	86.3	1015	2460	26.8	0.73	0.56	28.09	81.70	66.3	0.383	0.768
B	C	86.3	993	2460	26.2	0.73	0.56	27.52	79.98	66.2	0.375	0.768
C	A	86.3	834	2460	22.1	0.73	0.56	23.34	67.45	66.4	0.318	0.769
E	B	86.3	834	2460	22.1	0.73	0.56	23.34	67.45	66.4	0.318	0.769
F	B	86.3	834	2460	22.1	0.73	0.56	23.34	67.45	66.4	0.318	0.769
G	C	86.3	993	2460	26.2	0.73	0.56	27.52	79.98	66.2	0.375	0.768
H	H	86.3	1015	2460	26.8	0.73	0.56	28.09	81.70	66.3	0.383	0.768
1	E	86.3	292	2460	7.7	3.91	3.01	14.62	30.03	71.8	0.199	0.348
1	D	86.3	774	2460	20.5	3.91	3.01	27.37	68.28	71.9	0.373	0.791
2	F	86.3	588	2460	15.5	4.82	3.71	24.08	55.18	73.6	0.328	0.639
3	F	86.3	588	2460	15.5	4.82	3.71	24.08	55.18	73.6	0.328	0.639
4	E	86.3	292	2460	7.7	3.95	3.038	14.69	30.09	71.9	0.200	0.349
4	D	86.3	774	2460	20.5	3.95	3.038	27.44	68.34	72.0	0.374	0.792

Nominal axial capacity of (2) 800S250-97; [2] $(1.2+0.2S_{DS})P_{DL}+0.5P_{LL}+P_{seis}$;
[3] $(1.2+0.2)P_{DL}+0.5P_{LL}+\Omega_E P_{seis}$, where $\Omega_E=\Omega_o=3.0$; [4] $(1.2+0.2)P_{DL}+0.5P_{LL}+P_{exp}$; ϕ=0.85.

Per AISI-S400 requirements, chord studs should be sized for the expected strength of the shear wall, but need not exceed the load effect including seismic loads with overstrength. Increasing the height of the building could increase the overturning moment on the shear walls. Accordingly, (2) 800S250-97, can only meet the requirements for 4-story building and for taller buildings a higher capacity member is required for the chord studs, as shown in Table 7. Thus chord stud capacity is an immediate and important limiting factor for taller CFS buildings. Currently, either packs of CFS studs, or HSS sections have been used to work around this limitation.

Table 7. Chord stud and hold-down design summary

Number of Stories	4	6	8	10	12	15	18	20
Max chord stud size	800S250-97	Size>800S250-97						
Max hold-down size	S/HD158	Size>S/HD158						

Tie and Hold-down Design

Ties are used to transfer chord stud forces through the building floors. Hold-downs connect the chord studs to the foundation. Both ties and hold-downs need to be designed for the expected strength of the shear wall, but need not exceed the load effect including seismic loads with overstrength, per AISI-S400.

Table 8. Hold-downs demand analysis and design of the first story of the 4-story building (Note: gravity and seismic forces include th effect of the above stories)

										LRFD	Expected
Axis	SW	T_n[1] (kip)	# of H-downs	T_{seis} (kip)	P_{DL} (kip)	P_{LL} (kip)	T_u[2] (kip)	$T_{u\text{-}amp}$[3] (kip)	$T_{u\text{-}exp}$[4] (kip)	$T_u/\phi T_n$	Min (3,4) / T_n
A	H	42.4	2	26.8	0.73	0.56	25.81	79.42	62.51	0.43	0.737
B	C	42.4	2	26.2	0.73	0.56	25.30	77.77	62.58	0.42	0.738
C	A	42.4	2	22.1	0.73	0.56	20.09	64.20	59.18	0.34	0.697
E	B	42.4	2	22.1	0.73	0.56	20.09	64.20	59.18	0.34	0.697
F	B	42.4	2	22.1	0.73	0.56	20.09	64.20	59.18	0.34	0.697
G	C	42.4	2	26.2	0.73	0.56	25.30	77.77	62.58	0.42	0.738
H	H	42.4	2	26.8	0.73	0.56	25.81	79.42	62.51	0.43	0.737
1	E	42.4	2	7.7	3.91	3.01	4.20	19.60	49.78	0.07	0.231
1	D	42.4	2	20.5	3.91	3.01	16.56	57.46	57.25	0.28	0.675
2	F	42.4	2	15.5	4.82	3.71	10.25	41.35	55.85	0.17	0.487
3	F	42.4	2	15.5	4.82	3.71	10.25	41.35	55.85	0.17	0.487
4	E	42.4	2	7.7	3.95	3.038	4.17	19.57	49.75	0.07	0.231
4	D	42.4	2	20.5	3.95	3.038	16.53	57.43	57.22	0.28	0.674

[1]Nominal capacity of one S/HD158 (a Simpson Strong-Tie product); [2]$(0.9\text{-}0.2S_{DS})P_{DL}+T_{seis}$; [2]$(0.9\text{-}0.2S_{DS})P_{DL}+\Omega_E T_{seis}$; , where $\Omega_E=\Omega_o=3.0$; [3]$(0.9\text{-}0.2S_{DS})P_{DL}+T_{exp}$; $\phi=0.7$.

Ties (straps in Fig. 5 d) can be provided by flat plated connected to the web of the chord studs via required screws. There is no specific limitation for sizing the ties and accordingly, design of these elements is not reported herein. Notably, using short straps may result in block shear failure of the connection in tension. Moreover, the shear lag between the flanges and web of the chord stud needs to be studied further for large-scale applications. The alternative of using continuous tie rods is possible, but not detailed in the archetype herein.

Table 8 summarizes hold-down demand analysis and design for the first story of the 4-story building. Hold-downs have been designed for LRFD load combinations and also for expected seismic load combinations. For all chord studs, the expected seismic load combination governed the design.

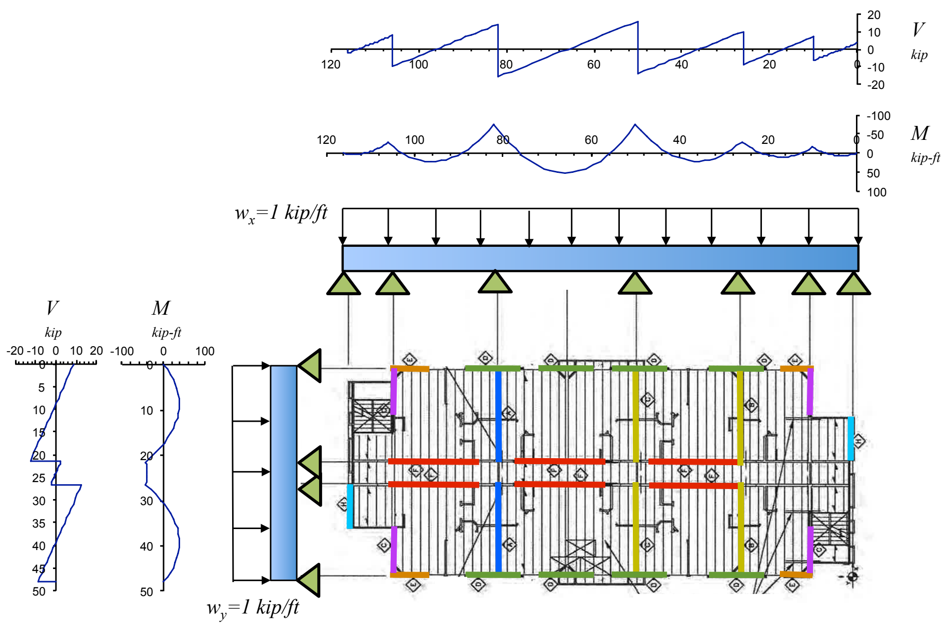


Fig. 6. Diaphragm analytical model for a unit distributed load of w=*1 kip/ft.*

Diaphragm Design

Floor diaphragms have been designed for the diaphragm design force, F_p, considering the minimum and maximum limitations, as required by ASCE 7. The diaphragm design force is applied as a distributed load (horizontal line load, w_x, and w_y) and the diaphragm is analyzed as a continuous beam on multiple supports, as shown in Fig. 6. The resulting maximum shear and moment of the beam is used to design the diaphragm shear and diaphragm chord design, respectively. Notably, the analysis shown in Fig. 6 has been provided for a unit distributed load of 1 kip/ft and the results can be used for all buildings by scaling the associated shear force and bending moment to the applied demands on the diaphragms, such as w_x and w_y.

The diaphragm design method is implemented in Table 9 for analysis and design of the 8-story building. The diaphragm has been designed using nominal shear capacity of 768 lb/ft provided in AISI-S400 as blocked 3/8 in. OSB floor sheathing and screw spacing at diaphragm boundary edges and at all continuous panel edges equal to 6 inches.

Table 9. Diaphragm analysis and design for the 8-story building

Long Direction	F_p (lb)	w (lb/ft)	V_{max} (kip)	M_{max} (kip-ft)	v_u (lb/ft)	v_n[1] (lb/ft)	$v_u/\phi v_n$	Chord Force (kip)
Roof	44111	380	6.1	29.1	127.0	768	0.28	0.61
8th	66759	576	9.2	44.0	192.1	768	0.42	0.92
7th	61384	529	8.5	40.5	176.7	768	0.38	0.84
6th	56550	488	7.8	37.3	162.8	768	0.35	0.78
5th	51903	447	7.2	34.2	149.4	768	0.32	0.71
4th	49847	430	6.9	32.9	143.5	768	0.31	0.68
3rd	49847	430	6.9	32.9	143.5	768	0.31	0.68
2nd	49847	430	6.9	32.9	143.5	768	0.31	0.68
Short Direction								
Roof	44111	919.0	11.7	39.9	101.1	768	0.22	0.34
8th	66759	1390.8	17.8	60.3	153.1	768	0.33	0.52
7th	61384	1278.8	16.3	55.5	140.8	768	0.31	0.48
6th	56550	1178.1	15.0	51.1	129.7	768	0.28	0.44
5th	51903	1081.3	13.8	46.9	119.0	768	0.26	0.40
4th	49847	1038.5	13.3	45.0	114.3	768	0.25	0.39
3rd	49847	1038.5	13.3	45.0	114.3	768	0.25	0.39
2nd	49847	1038.5	13.3	45.0	114.3	768	0.25	0.39

[1]AISI-S400-15. Table F2.4-1. ϕ=0.6.

Discussion

Dimensions and loading conditions of a unified archetype building are provided to help assessing the current design practice for mid-rise wholly cold-formed steel buildings and are intended to be used to evaluate novel structural systems for CFS construction.

According to ASCE 7 the maximum height permissible for light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets is 65 ft. However, this limitation is not considered here in an effort to find the limitations of the current practice and to provide an archetype for innovative lateral load resisting system. Based on available solutions buildings up to 52.51 ft (4 stories, with typical story height of 9.44 ft) were found to be possible and 200 ft (20 stories, with typical story height of 9.44 ft) plausible with only minor improvements in technology or design.

Shear capacity of the OSB sheathed shear walls provided in AISI-S400 could provide enough capacity for mid-rise buildings. As a measure of the amount of shear walls in the building, there is 1 ft of Type I shear wall per 35 ft^2 of the building in the unified archetype building (note: CFS-NEES building had 1 ft of Type I shear wall per 39 ft^2. This shows the archetype building has slightly more shear wall per plan area of the building). Obviously, providing less shear walls may lead to higher required shear capacity for individual shear walls and the capacity may be limited by the limitations of the design standard itself.

The deflection equation in AISI-S400 does not consider the overturning effect in the multistory buildings and the equation is essentially provided for a one-story building (shear wall). Accordingly, a more mechanic based analytical model is required for multistory building to consider the system effects.

Overturning moment at the base of the shear walls is a serious concern for multistory buildings that can affect design of chord studs, hold-down and anchor bolts. Moreover, the required demands on the foundations imply using mat or deep strip reinforced concrete foundations. The results shows that for even a 4-story building, the chord studs are to be built-up lipped-channels as large as 800S250-97. For higher demands more studs (stud packs) should be used. However, providing ties and hold-downs for more that two lipped channels is challenging. Using Type II shear walls may alleviate high axial demands on the chord studs; however, the load path and design method provided in the design standard for Type II shear walls has not practically examined for multistory building and more studies are required to understand the performance of these shear walls.

A similar issue exists for the design of hold-downs, and anchor bolts, as well as the design for bearing. Available hold-downs are barely enough for a 4-story building (we just examined Simpson strong tie, herein and not independently designed/engineered hold-downs). The required associated anchor bolt would be also larger than the available anchors. High compressive loads of the chord studs may need a separate baseplate to spread the load over the concrete foundation and the common bottom track may not be enough.

Diaphragm design showed that the intermediate shear walls (those that are not at the ends of the buildings) can effectively reduce shear and chord demands of the diaphragms. A simplified model, consisting of a continuous beam on multiple supports, is considered to analyze the diaphragms for in plane actions and the results shows that diaphragms are not critical in design and may function as rigid diaphragms.

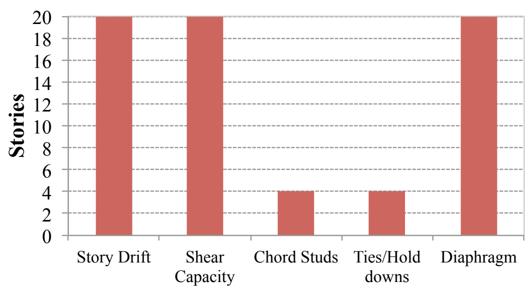


Fig.7. Summary of the design results: maximum number of stories per design limit states

Fig. 7 has summarized the result for all design buildings. Accordingly, providing high capacity chord studs, hold-downs, and anchors is required for enabling mid-rise and high-rise CFS constructions. Additionally, mechanics-based analytical models are required to model multi-story buildings and consider system effects.

Conclusions

Assessing current cold-formed steel (CFS) framing standards for mid-rise applications through a unified archetype building frame work sheds light on the potentials and limitations of the current practice to enable multistory CFS construction. Incorporating system effects in the analysis and design of mid-rise buildings in addition to high capacity shear walls that need high capacity chord studs, hold-downs, and anchors is needed to bring the efficiency of complete CFS construction (all systems framed from CFS) to mid-rise construction.

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