

DEFLECTION GAP STUDY FOR COLD-FORMED STEEL CURTAIN WALL SYSTEMS

by

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B.S., Kansas State University, 2009

A REPORT

Submitted in partial fulfillment of the requirements for the degree

MASTER OF SCIENCE

Department of Architectural Engineering and Construction Science
College of Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

2009

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Abstract

Cold-formed steel has become a preferred building material for wall framing in many different types of structures. One of its main uses has been as non-structural members in curtain wall assemblies of structural steel framed buildings. In an exterior wall application, the main purpose of the curtain wall is to transfer out of plane loads to the steel frame while not supporting any superimposed gravity loads. Therefore, when the curtain wall is in the plane of the structural steel frame, the vertical deflection of the spandrel beam directly above the wall must be known to provide the appropriate deflection gap between the beam and the curtain wall so that gravity loads are not transferred to the wall.

Common practice is to size the gap for the deflection from 100% of the live load. In some cases, the deflection gap may be significant, and since this gap must also be provided in the exterior cladding of the wall, it creates a design issue for the architect. This report presents the results of an investigation into the feasibility of reducing the size of the deflection gap when the wall is located directly under the spandrel beam.

In this study, analytical models were developed for common design situations of curtain walls constructed of cold-formed steel studs in structural steel framed buildings. This study investigates two common stud heights combined with different floor live loads. Taking into account that wall studs have some available axial compressive strength, a procedure was developed to determine an appropriate reduction for the gap. Using an iterative process a relationship is made between the axial compressive strength of the stud and the amount of axial load the stud can support to establish a factor which gives the percentage the live load gap for 100% live load can be safely reduced by.

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Acknowledgements

I would like to thank Sutton Stephens for proposing the topic. His patience, support and guidance throughout the report cannot be overlooked. I would also like to thank Kimberly Kramer and Bob Condia, who were part of my committee, for their support in the subject matter. And finally, I would like to thank the Architectural Engineering department for the resources that were made available to me in completing this report.

Additionally, I would like to thank my family for their restless motivation and energy in finishing this report. Without their encouragement this report would not be.

1 INTRODUCTION

Structural steel building frame systems present some unique challenges to a designer. Unlike many common materials used in construction, metal buildings are integrated assemblies of many structural members and related accessories, all of which are custom configured by the engineer as required by the situation specific to the project (AISC, 2003). These integrated assemblies can vary from wall systems, floor systems and even connections of members.

As dead and live loads are applied to roof and floor framing systems an unavoidable vertical deformation will occur. Structural steel building framed systems typically do not have load bearing walls used to resist vertical deformations of the floor since the frame is design to resist all the forces applied to the building. In the case of building frame systems non-load bearing walls must be designed to accommodate vertical movement of the frame. A non-load bearing wall can also be said to be a non-structural member since it is independent from the primary building frame system. The 2006 *International Building Code* (ICC 2006) defines a non-load bearing wall as “Any wall that is not a load-bearing wall”, and its definition for a load-bearing wall is “Any metal or wood stud wall that supports more than 100 pounds per linear foot of vertical load in addition to its own weight”. In other words, a non-load bearing wall is expected to support very minimal axial loads. Also, the *North American Standard for Cold-formed Steel Framing- General Provisions* (AISI S200-07), defines a non-structural member as “A member in a steel framed system which is limited to a transverse (out-of-plane) load not more than 100 lb/ft², a superimposed axial load, exclusive of sheathing materials, of no more than 100 lb/ft, or a superimposed axial load of not more than 200 lbs”. This definition simply paraphrases the definition from the IBC2006. In fact, the IBC 2006 refers to the AISI Standards when designing with cold-formed steel. The inherit deflections from the roof or floors above these walls then must be carefully determined to design the non-bearing walls so that essentially no axial loading is applied to them.

Non-load bearing walls are commonly used on the exterior of structural steel building frame systems. As part of the building envelope, the exterior walls, known as curtain walls,

become a significant aspect of the building design. Since they are exposed to external conditions, they must be design to resist any lateral forces applied to them such as wind and seismic loads. Curtain walls are also limited in the amount of axial load it can support since they are a non-load bearing system. Typical bay spans in building frame systems can sometimes be significantly long and thus produce large vertical deflections grater then 1 inch. These deflections must be accounted for in the curtain wall system, and must be considered in design. To support this, the AISI S200-07 defines a curtain wall as “a wall that transfers transverse (out-of-plane) loads and in limited to a superimposed vertical load, exclusive of sheathing materials, of not more than 100 lb/ft, or a superimposed vertical load of not more than 200 lbs”. This definition states that a curtain wall assembly is considered to be a non-load bearing wall.

This report concentrates on the vertical deflections a cold-formed steel curtain wall system has to accommodate after it is installed in the building frame system. This vertical deformation is reflected as a deflection gap at the top of the curtain wall. The main goal of the study is to determine if a smaller deflection gap can be use for the curtain wall system in order to possibly reduce the vertical gap in the architectural exterior finishes of a building.

1.1 Background

Since the early 1900's, curtain walls have become more and more popular in architectural design for modern buildings. Two common types of curtain wall assemblies are used: glass system and cold-formed steel studs. The glass system provides for an appealing building as well as providing additional benefits such as day-lighting and climate control due to temperature transfers through the glass. The cold-formed steel stud system may have various exterior finishes such as stone and brick veneer, metal panels, louvers, etc (LGSEA, 2001) applied. Figures 1.1 and 1.2 show some examples of exterior curtain walls.



Figure 1.1: Glass curtain wall system



Figure 1.2: Stone Veneer curtain wall system

This report studies curtain walls with cold-formed steel studs. When considering cold-formed steel (CFS) framing, many possibilities in the assemblies and applications of the walls exist. Curtain walls are usually attached to the primary building frame of a buildings structure and therefore must be designed to accommodate any movement of the primary frame (LGSEA, 2001).

1.1.1 *Curtain Walls*

The purpose of a curtain wall is to resist air and water infiltration, transfer wind forces acting normal to the plane of the wall, seismic shear forces, and its own weight. For this report only; wind forces will be considered and it is assumed that the wall system is adequate for water and air infiltration and that seismic forces are treated similar to wind forces. Curtain walls require a vertical slip connection to accommodate the roof or floor deflections from the levels above so that axial are not transferred to the studs. Many components of a CFS curtain wall system must be considered to resist out-of-plane forces and building frame deflections. One component is the structural stud itself. C-shaped sections are most commonly used for studs in a curtain wall assembly for it geometric configurations and strength. They typically range from 33 mil to 68 mil in thickness and have a minimum yield strength of 33 ksi (LGSEA, 2001).

Another important component is bridging. Bridging in CFS design is needed to prevent in-plane buckling and twisting of the member when subjected to out-of-plane lateral forces such as wind. Wind forces normally control the design on a curtain wall system either for strength, bending, or serviceability, deflection. Bridging is used to reduce lateral torsional buckling either permanently or until sheathing is applied. Bridging in a CFS wall is usually provided by horizontal U-shaped channels that run through pre-punched wed holes of the studs and usually placed at third point along the CFS studs vertical height (LGSEA, 2001). Figure 1.3 illustrates the use of bridging.

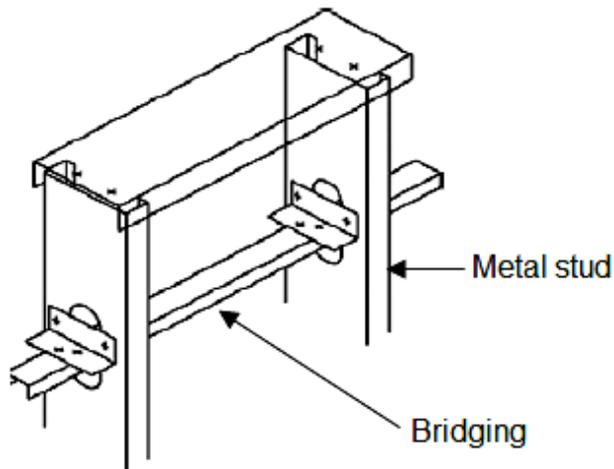


Figure 1.3: Bridging detail in a CFS stud system (Rahman, 2003)

[With permission from Nabil Rahman]

The use of bridging helps reduce the unbraced length of the studs for lateral torsional-flexural buckling. For the purposes of this study, the CFS studs analyzed will have bridging spaced out at third points but not greater than 4'-0". This study considers axial compressive forces transmitted from the building frame therefore bridging also braces the weak axis for overall buckling in compression (LGSEA, 2001).

1.2 Objectives

There does not seem to be a consensus on the method to determine the actual magnitude of the floor or roof deflection that should be considered for the design of the slip track connection detail for the curtain wall. Most commonly, a deflection gap between the top of the wall and steel frame is sized for the full live load deflection of the floor or roof system. This may be overly conservative (AISC, 2003). The dead load supported by a beam has to be accounted for, however since most of it will be present prior to constructing the curtain wall; therefore, the gap due to dead load will be ignored in this study. Live load on the other hand, will produce deflection after the curtain wall is in place and must be considered. The purpose of this report is to demonstrate the feasibility of reducing the deflection gap due to the live load applied on the building. By reducing the deflection gap it allows for a similar reduction in the

vertical gap for the architectural finishes for the wall. With the anticipated reduction in the deflection gap other benefits can arise.

One of the main benefits of reducing the deflection gap would be a decrease in the steel cost. Designing for the full 100% of the live load deflection needed may require a thicker CFS stud than if designing for a reduced deflection; thus the cost of steel is minimized. A second benefit is having the studs delivered to the site already pre-cut to the required length and thus eliminates the need to cut any studs on the field; which allows for the potential of having a faster construction time. A final benefit in the reduction of the deflection gap is the possibility of also having a reduction in the size of the control joints in the architectural finishes. It would seem that less magnitude of movement in the joint will increase the durability of the joint.

In this study the deflection gap is obtained by developing common design situations for curtain walls constructed of cold-formed steel studs in structural steel framed buildings. This study investigates two common wall heights for buildings; ten foot and twelve foot, with different live loads applied to the spandrel beam, such as 50 psf and 80 psf. Taking into account the wall studs have some available axial compressive strength, a procedure was developed to in the study to determine an appropriate reduction for the gap. Using an iterative process a relationship is made between the axial compressive strength of the stud and the amount of axial load the stud can support to establish a factor that when applied gives the percentage the live load gap for 100% live load can be safely reduced by.

2 SPANDREL BEAM DESIGN

In a structural steel building frame system, the exterior beam that spans from column to column at each level is known as the spandrel beam. It usually supports the floor or roof framing onto the beam and the exterior wall panel. The spandrel beam plays an important role in the support of the exterior cladding of a structure. Figure 2.1 & 2.2 show locations of a spandrel beam in a typical building frame system and a typical section through the spandrel beam.

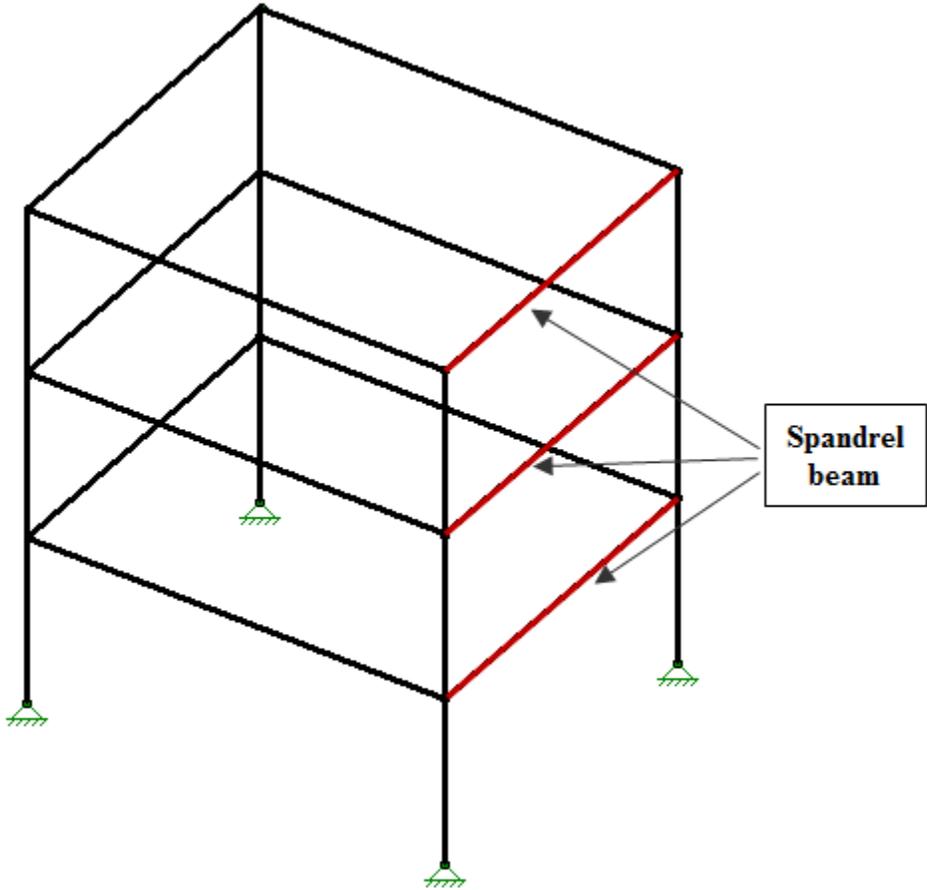


Figure 2.1: Metal Building Frame

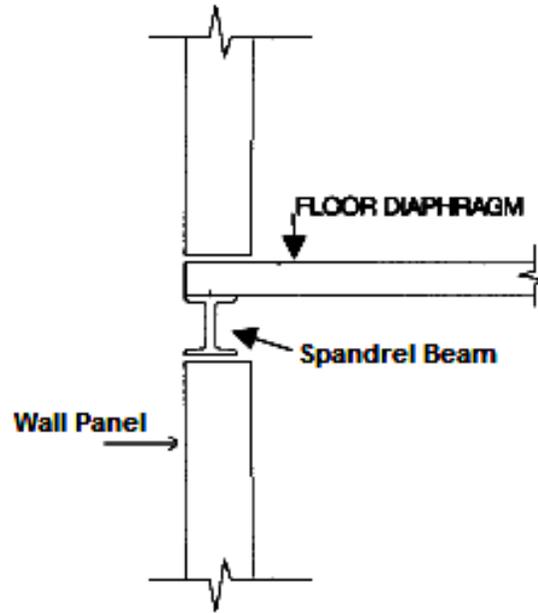


Figure 2.2: Spandrel Beam Detail

For a spandrel beam the probability of the having the full live load applied to it should be considered. The chance that even 75% of the live load is accumulated in the perimeter of the building is small (AISC,2003). Noting that in any type of building, the areas close to the walls, especially if windows are located in the walls, will typically not have a lot of activity by people around them. With this, the feasibility of reducing the deflection gap due to live load is even more plausible (AISC, 2003).

Considering that the spandrel beam will resist gravity loads and lateral loads it must be braced to prevent rotation due to lateral loads applied to the bottom flange. Also, if the bottom flange of the beam is braced at discrete points along its length, then the beam must also be checked for bending and torsion between the support points.

2.1 Analysis

The spandrel beam is part of the floor or roof system that must be of sufficient size to support the self weight and superimposed load with a limited deflection. Deflection limits are

provided in the IBC 2006 Table 1604.3. Live load deflection of the spandrel beam must be accounted for in the design of the deflection gap at the top of the wall. For this report, wall openings are ignored and the spandrel beam live load deflection is accommodated in the slip connection of the curtain walls (AISC, 1997).

Two ways the curtain wall can be attached to the primary frame of the structure are typically used. One is the head-of-wall condition shown in Figure 2.3. This is when the wall studs extend from the top of the floor slab to the bottom of the spandrel beam. The other way is when the wall is attached using the bypass condition. This is when the wall studs pass outside the primary building frame and are connected by a vertical clip attached to the edge of the slab or beam (AISC, 1997). Figure 2.4 shows the bypass condition.

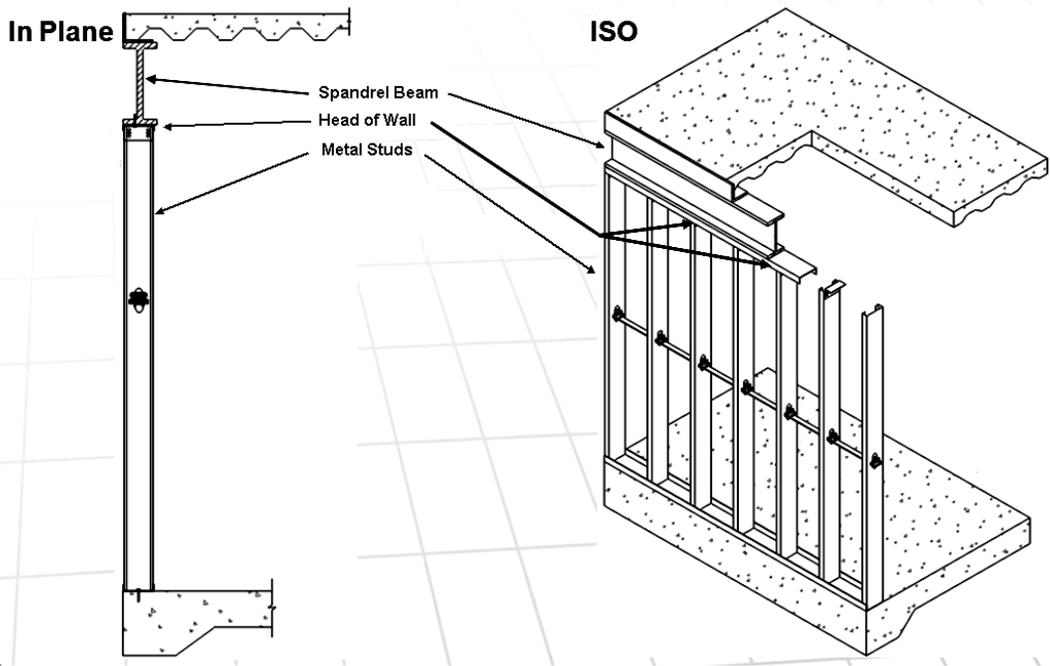


Figure 2.3: Head-of-Wall Condition

[With permission from Nabil Rahman and The Steel Network
<www.steelnetwork.com/steel_framing_products>]

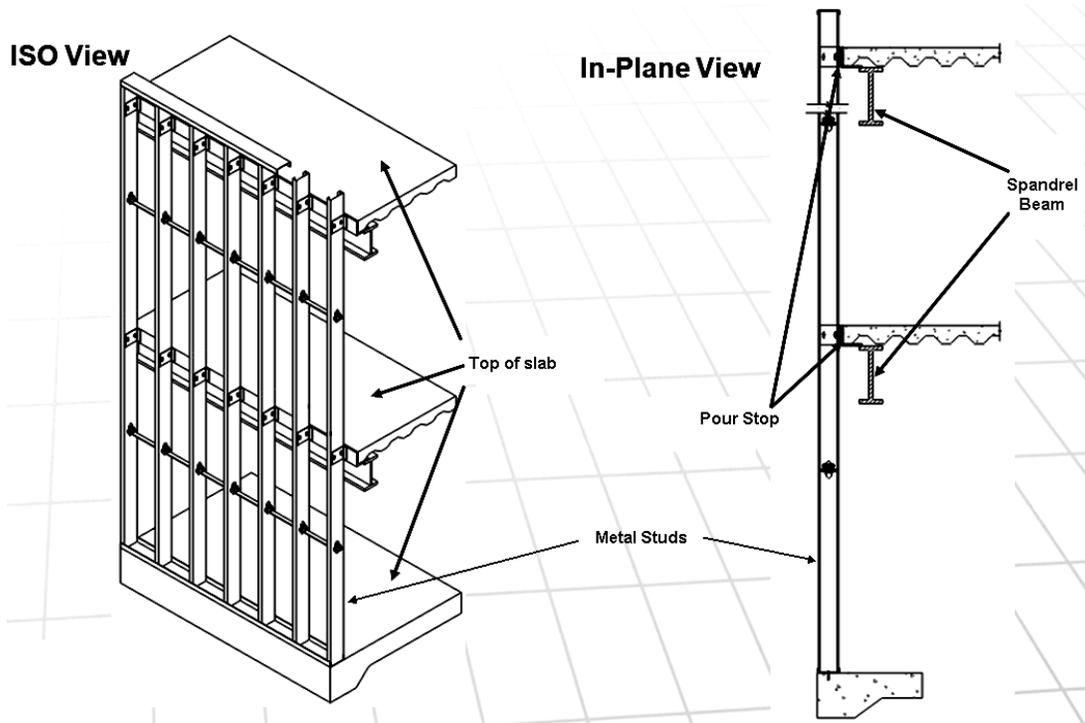


Figure 2.4: Bypass Condition

[With permission from Nabil Rahman and The Steel Network
www.steelnetwork.com/steel_framing_products]

These two common conditions are appropriate for curtain walls in the building frame systems and each brings different approaches to the design of the spandrel beam. The bypass condition bears eccentrically on the spandrel beam since the reaction is not at the beam centerline, causing torsion that must be resisted by the beam. Figures 2.5 shows a free body diagram of the forces applied to the spandrel beam under the bypass condition (MSC, 2007).

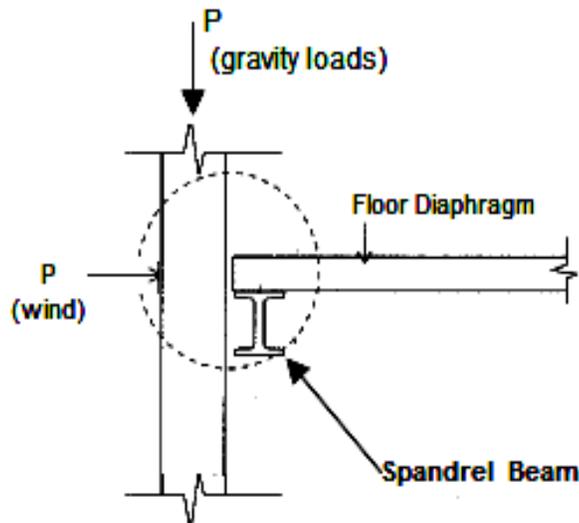


Figure 2.5: Free Body Diagram with Bypass Condition

The head-to-wall condition is where the wall studs are located closer to the beam centerline and does not introduce significant torsion. The head-of-wall attachment is at the bottom of the spandrel beam; whereas the bypass condition is more flexible in its attached location. The bypass attachment can be made at the bottom of the spandrel beam or to the top of the floor supported by the spandrel beam.

2.2 Design Standards for Spandrel Beams

Regardless of the connection type, when analyzing a spandrel beam many factors must be considered since different load and detailing conditions all have an impact on the design. Strength design, serviceability, and constructability have to be considered for any situation. The following is a list of design considerations (MSC, 2007).

Strength Design:

- Superimposed gravity floor or roof loads (dead, live, snow, rain).
- Lateral loads (wind, seismic) for weak-axis bending and torsion.
- Eccentric façade load that produce torsion.

Serviceability:

- Deflection due to gravity loads (dead, live)
- Rotation of beam from eccentric loads
- Long term creep of composite floor systems
- Lateral displacement of the structural frame

Constructability:

- Depth limits of beam from plenum spaces
- Interferences from walls openings
- Interferences from mechanical and electrical equipment
- Flange width limitations for edge distances

2.3 Bracing Conditions for Curtain Wall Attachment

The manner in which the curtain wall assembly is connected to the spandrel beam, in a larger part also determines the bracing system required for the beam. For instance, the head-of-wall condition can be designed to transfer lateral forces directly into the floor diaphragms. In this case no additional bracing system is required since there is no significant eccentricity at the connection as shown in Figure 2.6. But, since wall panels are not always attached as a head-of-wall condition a bracing system could be introduced to provide a load path lateral forces into the floor diaphragm from the of the wall panel; this bracing is known as a kicker and is shown in Figure 2.7. In either case, the bracing system eliminates torsional loading of the spandrel beam. Also, “torsional bracing may be provided at eccentric load points to reduce or eliminate the torsional effect” (AISC, 1997). This simply means that if there is a point load placed along the floor system, a distance away from the spandrel beam that significantly influences torsion, a diagonal brace (kicker) must be provided. This is true regardless of the attachment condition of the curtain wall system.

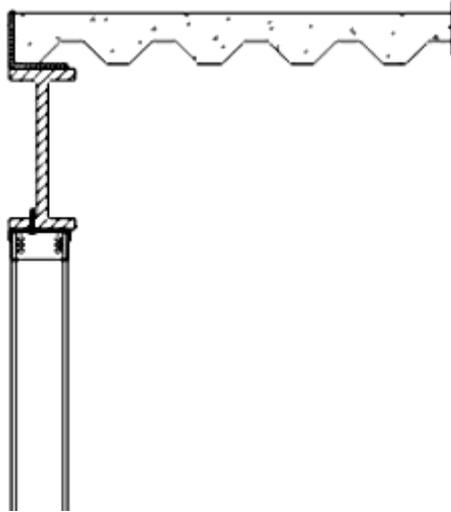


Figure 2.6: Bracing for Head-of-Wall Condition

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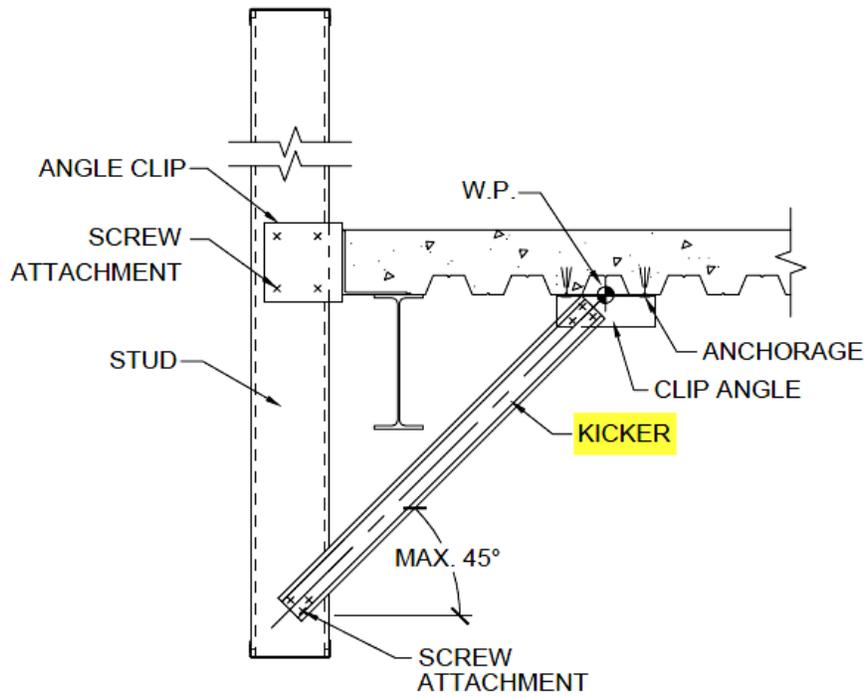


Figure 2.7: Bracing Detail (SSMA)

[With permission from Steel Stud Mfrs. Assn. (SSMA), Glen Ellyn, IL]

Another consideration for the design of the spandrel beam is applying a camber to the beam; “the pre-deforming of a member so that, in a loaded state, it more nearly approximates its theoretical presumed shape” (Ricker, 1989). A camber is applied to accommodate dead and live load deflections and is placed at the mid-span of the member; where the maximum deflection will occur. Cambering a steel member can significantly change the stress properties of the member, if the camber is large enough. Due to a possible change in steel properties and the torsional loads present on a spandrel beam; it is common practice not to camber them (Ricker, 1989). The conditions where it is reasonable to have camber at a spandrel beam is when it has a large span (greater than 20 feet) with a large the vertical deflection due to dead and live loads and a substantially large dead load applied to the beam (MSC, 2007 & Ricker,1989). In a typical metal frame building system, since it is not common to have camber in the spandrel beam and will be ignored for the purposes of this report.

This report is limited to the head-to-wall condition since the curtain wall is attached directly under the spandrel beam. The deflection of the spandrel beam due to gravity loads is what the deflection gap connection at the top of the curtain wall must accommodate. Therefore, looking at the head-of wall attachment system to the spandrel beam may allow for a more precise analysis results.

3 COLD-FORMED STEEL STUD WALLS

Cold-formed steel studs are often used in an integrated system with a structural steel building frame as part of non-load bearing walls systems. Typically used for curtain wall systems in structures that have a very symmetrical configuration, such as office buildings and warehouses, where repetition is desired (LGSEA, 2001). Repetition in a structure allows for faster construction and gives less room for errors. Cold-formed steel studs are a lightweight material and considered to be relatively thin, which brings about some distinct design limitations. Generally cold-formed sections are shaped and formed from flat sheets, and when formed at room temperature, changes the original properties of the steel. The thickness of the CFS studs are usually less than 1/8 inch (3 mm) thick, which means they generally have a predominate failure mode of buckling. Since CFS wall studs can be relatively tall, limitations for slenderness, bracing and serviceability must be checked (AISC, 2003).

Although, there are often many factors that must to be checked, designing a wall with lightweight material like cold-formed steel have certain advantages. The advantages of using cold-formed steel stud wall systems are numerous: the recyclable nature of the material, low weight, ease of erection for installment, high strength, custom shapes, post buckling strength, and element stiffening characteristics (AISC, 2003).

3.1 Design Standards

Cold-formed steel wall stud design standards have been developed by the American Iron and Steel Institute (AISI) Committee on Framing Standards (COFS). These Standards have been written based on the *North American Specifications for the Design of Cold-Formed Steel Structural Members* (AISI S100-07). The standards that covers wall stud design is the *North American Standard of Cold-Formed Steel Framing - Wall Stud Design* (AISI S211-07); which provides technical information and requirements for the design of wall studs made from cold-formed steel. It applies the relevant sections of AISI S100-07, including load combinations specific to wall studs, bracing requirements and connection requirements. The International Building Code (IBC) has since adopted these standards and thus has become a requirement.

3.1.1 Load Combinations

The load combinations used in AISI S211-07 comply with the *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-05). For this study the strength design factored load combinations (LRFD) are used. From the list of the LRFD basic load combinations, the governing ones for wall studs in this study are (2) and (4):

$$(2) \quad 1.2(D+F+T) + 1.6(L+H) + 0.5(L_r \text{ or } S \text{ or } R)$$

$$(4) \quad 1.2D + 1.6W + f_1 L + 0.5 (L_r \text{ or } S \text{ or } R)$$

ASCE 7 allows f_1 to be 0.5 when the live load is less than 100 psf. In load combination (4) this f_1 factor can be applied if the design loads due to the building occupancy are less than 100 psf. For this study the 0.5 factor is used since the live load to be applied is for an office building, which is less than 100 psf (ASCE 7-05). With the elimination of the notations for the loads that are not present, the load combinations simplify to:

$$(2) \quad 1.2D + 1.6L$$

$$(4) \quad 1.2D + 1.6W + 0.5L$$

Load combination (2) is used when analyzing the stud due to axial loads only and load combination (4) is used when analyzing the stud due to combined axial compression and bending. In equation (4) the wind load is applied according to AISI S211-07. CFS wall stud are sized due to bending alone, and the more critical wind loading must be used. Components and Cladding (C&C) wind loads, out-of-plane with the wall stud, are the most critical and is used to size the CFS stud. On the other hand, The Main Wind Force Resisting System (MWFRS) wind loads are used when examining the studs due to combined axial compression and bending. This is because the studs are already sized for the worst case wind load. Also because, when checking combine loads it considers the whole framing system instead of individual components (AISI, 2007d).

3.1.2 Member Design

In AISI S211-07, either an all steel design or a sheathing braced design can be used for the design of the studs. If considering a sheathing braced design, it must be assumed that the same sheathing is used on both sides of the wall. The sheathing must also be connected to the top and bottom horizontal tracks to prevent lateral and torsional buckling of the studs. In the case of an all steel design the contribution from any structural bracing due to sheathing is ignored. For this study an all steel design was used with bridging used at discrete locations to brace the studs and provide lateral and torsional support (AISI S211-07).

Axially loaded wall studs are required to be supported at the top and bottom horizontal tracks for support against rotation and horizontal displacements. In the all steel system the design must comply with AISI S100-07 Sections C4.2 and D4 (a). When designing for combined bending and axial loads the interaction equations in Section C5 of AISI S100-07 must be satisfied. These equations are:

$$\frac{\bar{P}}{\phi_c P_n} + \frac{C_{mx} \bar{M}_x}{\phi_b M_{nx} \alpha_x} + \frac{C_{my} \bar{M}_y}{\phi_b M_{ny} \alpha_y} \leq 1.0 \quad (\text{Eq. C5.2.2-1})$$

$$\frac{\bar{P}}{\phi_c P_{no}} + \frac{\bar{M}_x}{\phi_b M_{nx}} + \frac{\bar{M}_y}{\phi_b M_{ny}} \leq 1.0 \quad (\text{Eq. C5.2.2-2})$$

$$\frac{\bar{P}}{\phi_c P_n} + \frac{\bar{M}_x}{\phi_b M_{nx}} + \frac{\bar{M}_y}{\phi_b M_{ny}} \leq 1.0 \quad (\text{Eq. C5.2.2-3})$$

These are the two design criteria that were used for this study. Other design considerations not considered in this study, for simplifications purposes, include shear design and web crippling; all which must comply with the AISI S100-07 specifications (AISI S211-07). This report assumes that these criteria's are satisfied.

3.1.3 Connection Requirements

In CFS wall stud design, two main connection locations must be designed properly to have an adequate system. One location is the stud-to-track connection at the base of the stud.

For this connection two conditions are evaluated: (1) wall studs that are not adjacent to wall openings and (2) wall studs that are adjacent to wall openings. The first connection condition is the one which this study is concerned with. The next section of this report takes a closer look into the deflection track connection and covers current design considerations both structurally and architecturally.

4 DEFLECTION GAP

The main purpose of the deflection connections is to allow for the deflection of the primary buildings structure to occur without transferring any axial loads to the wall studs. The deflection gap provides a separation between the curtain wall system and the primary metal frame system in a head-of-wall condition. Figure 4.1 shows a typical detail of this connection using a single track connection. For the single track configuration, bridging must be located a distance that is at least "...twice the distance between sheathing connectors" (AISI S211-07). And it must be within 12 inches of the deflection track (CFSEI, 2009). These constraints are due to the attachments condition of the deflection gap. The top track of the wall and the wall studs are not attached to each other. The track overlaps the wall studs and flanges from the track support the wall studs. Figure 4.1 also shows the how the track overlaps the wall stud. Bridging at the top provides additional support to keep the system in place as well providing lateral support. Therefore, the bridging location cannot be too far from the top.

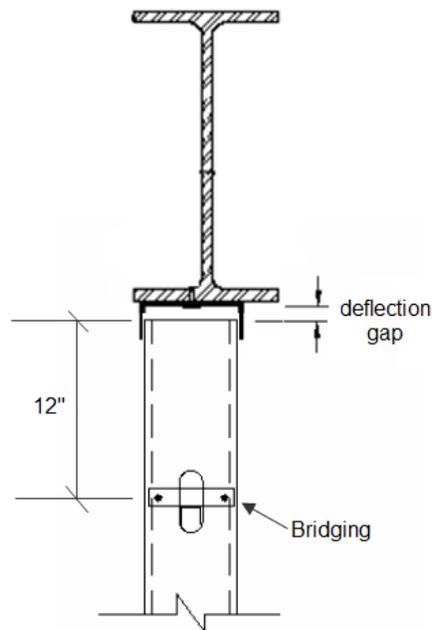


Figure 4.1: Deflection gap detail (CFSEI, 2009)

[With permission from Cold-Formed Steel Engineers Institute (CFSEI), Don Allen]

4.1 Current Deflection Gap Design Methods

There are four common types of vertical deflection connections used to accommodate the live load deflection of the spandrel beam above the wall studs. The first is the slotted clip angle connected to the stud and the head-of-wall track shown in Figure 4.2 (a). The second is a slotted track connected to the stud shown in Figure 4.2 (b). The third connection is a single deep leg track with no attachment to the stud shown in Figure 4.2 (c). Finally, the fourth connection is a double track assembly with no attachment between the interior and exterior tracks shown in Figure 4.2 (d) (Rahman, 2005).

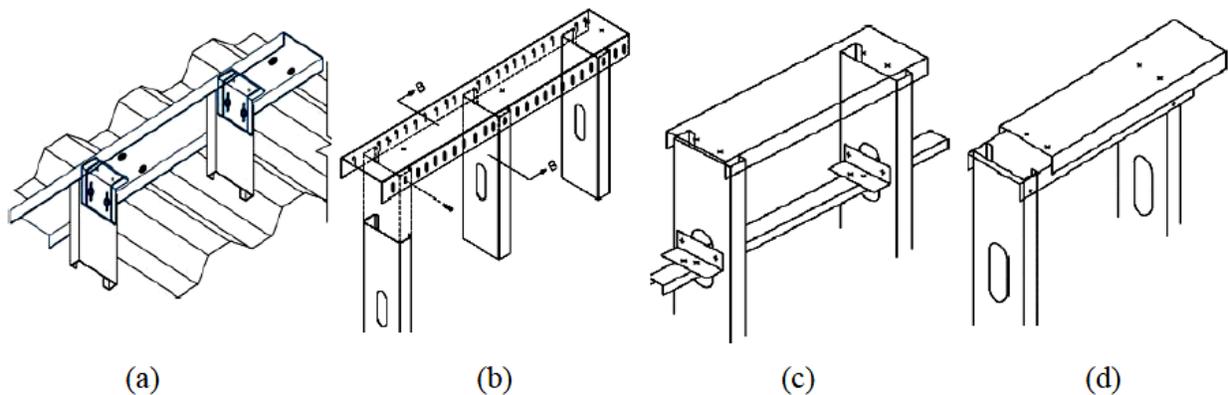


Figure 4.2: Deflection Gap Connections (Rahman, 2005)

[With permission from Nabil Rahman and The Steel Network
<www.steelnetwork.com/steel_framing_products>]

The most commonly used connections of these are the single deep leg track connection and the double track assembly. The connection that is assumed in this study is the single deep leg slip track connection (Rahman, 2005). When using a single deep leg slip track, it is preferred that the thickness of the track and the wall studs are the same. When the track and stud thicknesses are equal an increase in strength due to a synergistic condition that is present (AISI, 2007). This is required for any of the deflection gap connection shown. Considering that the track and the stud are not attached to each other, the flange of the track should be long

enough to contain the wall studs in place and to transfer any forces caused by lateral loads to the building frame through the spandrel beam. In Figure 4.3 a more detailed connection of the single slip track is shown (SSMA, Jan. 2000).

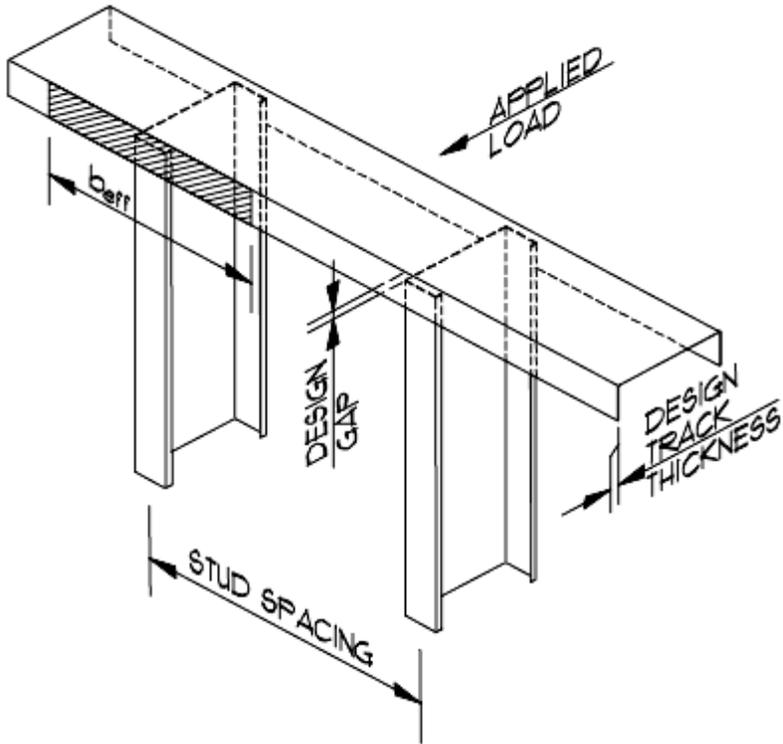


Figure 4.3: Single Deep Leg Slip Track Connection for Deflection Gap (SSMA, 2000a)
[With permission from Steel Stud Mfrs. Assn. (SSMA), Glen Ellyn, IL]

The Steel Stud Manufacturers Association (SSMA) has established recommendations for determining the proper length of the flanges of the track. For a one story building application, the length of the track flanges should be, at a minimum, equal to 1 inch plus the designed gap of the deflection. For all other building applications the track flanges should be, at minimum, equal to 1 inch plus twice the design gap of the deflection (SSMA, 2000). Even though the track and the wall studs are not connected, the track is connected to the primary building steel frame system; in this case to the bottom of the spandrel beam. Thus the track will deflect with the

beam and the calculated designed deflection gap must be enough to accommodate the vertical movement of the beam without bearing on the wall studs (Gerloff; Huttetmaier; Ford, 2004).

4.2 Deflection Gap Effect on Architectural Wall Finishes

The exterior finishes do not always impact the performance of the curtain wall. But, the behavior of the curtain wall does impact the exterior wall finishes. The affect of wall finish materials such as gypsum wallboard, exterior stucco or siding, brick veneer, etc. on the out-of-plane behavior varies with the magnitude of the force applied, but will behave similar to the curtain wall it is attached to. This behavior must be taken into account by the architect. It can be assumed that the exterior finishes add stiffness to the curtain wall and therefore reduces drift limits to the structure, but this is not necessarily the case and is never considered design engineer. Influences of the curtain wall stud behavior can cause cracking in the architectural finish to the point of failure if not taken into account. Typically the architect assigned to the specified project provides this coordination for the curtain wall design.

The deflection gap in the curtain wall system is accommodated in the exterior finishes of the wall by movement joints. A movement joint plays several roles for the purpose of the façade requirement. The primary intent of the movement joint is to control any stresses induced into the façade by the building such as deflections. Having a reduction in the deflection gap may help reduce the movement joint in the exterior wall finish. Since this report investigates the feasibility of reducing the gap, it may be of use to the architect in their design coordination. By doing so, it may prevent cracking in the wall finish and maintain integrity of the wall system. But, the movement joint serves as other purposes for the architect that are take into account. The joint also acts as a water sealant joint to prevent any water from penetrating into the wall cavity. Another role is it acting as an expansion joint for any thermal expansion the wall façade may undergo (Goldberg, 1998).

5 PROBLEM STATEMENT

5.1 Objective

The purpose of this study is to demonstrate the feasibility of reducing the deflection gap required due to live load applied to the spandrel beam at the top of the curtain wall system. A typical office building that has a metal building frame as its system is analysed with different loading situation on the spandrel beam. Varying wall heights are also considered for the analysis. The critical location of the studs, are the ones at the spandrel beam mid-span. At this point along the wall the greatest deflection on the spandrel beam will occur and where the first loading, if any, will be transferred to studs.

Considering the wall stud is able to support some axial load due to its axial compression strength, calculations can be made to determine exactly how much the deflection gap can be reduced. The stud can be analyzed for combined axial and bending and a relationship made to its capacity due to axial load only. The goal is to obtain a gap reduction for the stud size and thickness that would normally be selected as a curtain wall.

5.2 Scope

To study the deflection gap for a head-of-wall condition, a structural steel building frame system was chosen. An office building occupancy was selected since multistory steel frame buildings are many time constructed for office spaces. A 90 MPH wind speed was chosen because most of the United States is in the 90 MPH zone. The building is assumed to be at least four stories high which is typical for many office buildings.

The floor plan of the study building was divided into six different bay sizes with different framing options; all which were analyzed. The bay sizes are shown in Figure 5.1: (a) 25'X30' with two joists equally spaced, (b) 30'X30' with two joists equally spaced, (c) 30'X30' with three joists equally spaced, (d) 40'X30' with three joists equally spaced and (e) a 40'X30' with a one way concrete slab as the floor system. The total height of the building was chosen at 44-ft high. The bottom two floors are 12-ft high and the top two floors are 10-ft high. This allows for a 10-

ft and a 12-ft minus the depth of the spandrel beam and the depth of the floor system, stud to be evaluated. To simplify the analysis the full floor to floor height was used. The walls analyzed are the exterior curtain walls at the second and third levels of the building. Figures 5.1 and 5.2 show the plan of the analyzed building and a typical bay elevation. In the analysis two common live load conditions for office buildings were studied; a 50psf and an 80psf live load. These loads were obtained from the IBC 2006 Table 1607.1 for Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Load. The occupancy for office buildings was chosen. The wall studs will be analyzed when only 50% of the live load is applied to see if the stud is adequate and then when 100% of the live load is applied to see how much it fails by. Knowing that the wall studs will not be able to carry the full load, adjustments in the percentage of live load applied were made to determine the exact percentage of the load the stud can support before it fails. This was obtained by dividing the adjusted live load the stud is able to support before it fails by the nominal capacity of the stud.

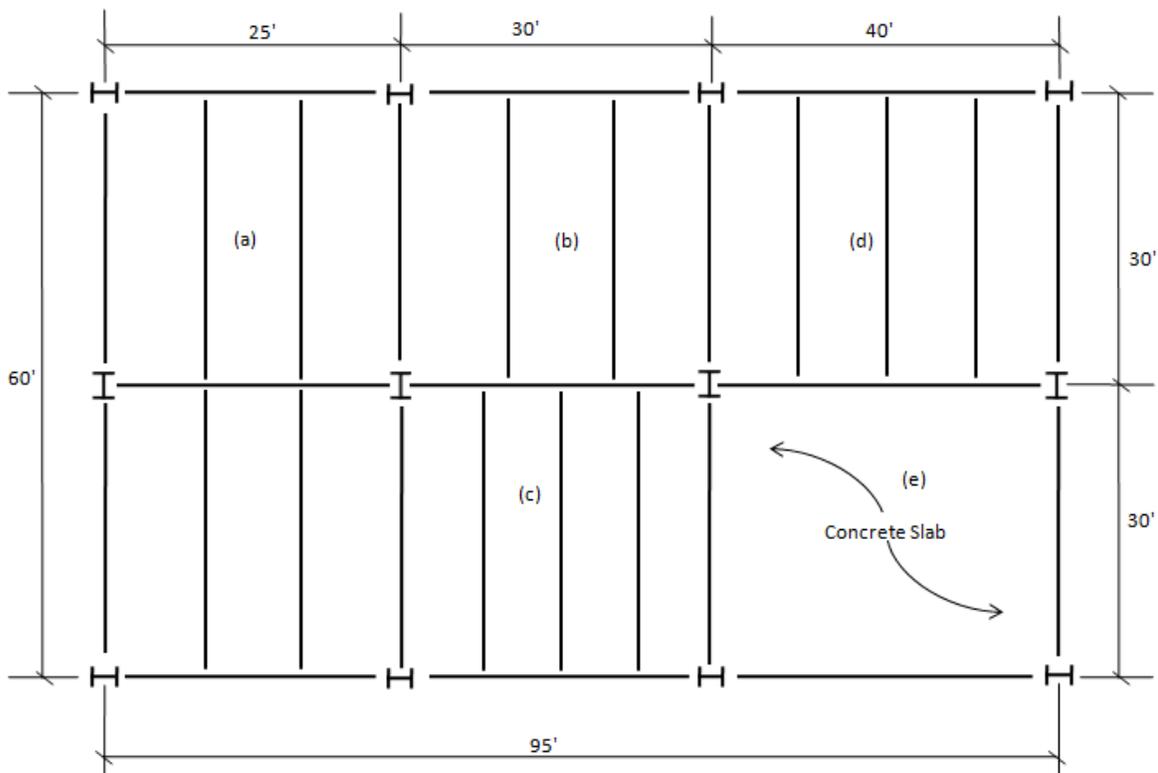


Figure 5.1 – Proposed Building Plan

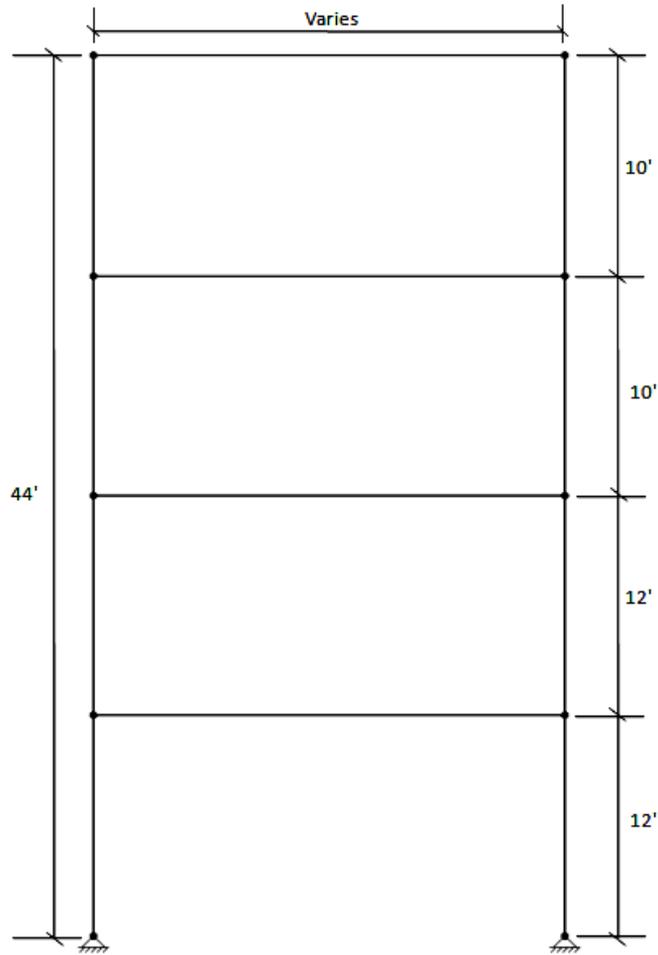


Figure 5.2 – Typical Bay Elevation Plan

Other design considerations for this study are:

- Building exposure category B for wind
- No roof overhangs on the building
- No parapets above the roof
- The building is classified as enclosed for wind design
- Seismic forces are less than the wind forces
- The building is rigid
- P-Delta Effects are ignored

- Live load reduction is not used
- Self weight of the wall stud is ignored
- The studs are braced by bridging at third points
- Spacing of the wall studs is 16"o.c.
- CFS Studs are concentrically loaded
- Serviceability of the CFS stud is accounted for in the selection of the stud as a curtain wall

5.3 Analysis Description

To begin the study, wall studs were initially sized for a uniform wind load with no axial compressive load. The wind load used for the selecting the stud size is for components and cladding (C&C) in accordance with AISI S211-07. Thus to size the stud the C&C wind load on the wall was calculated (See appendix A for wind load calculation). With the initial assumptions and the building configuration assumed, the C&C wind force was calculated to be 16.23 pounds per square foot (psf). With the wind load established and the heights of the walls known the stud size was selected. For this study wall studs were chosen from the SSMA standard catalog literature. Once the size was determined for the wall stud, the commercial software program CFS version 6.0.2 was used to establish the nominal strengths for bending and axial compression (See appendix B for CFS program calculations) (RGS, 2009).

With a stud selected for the curtain wall system, the maximum axial compression loads on the studs was then determined due to the applied loads on the spandrel beam. Knowing that the studs located at the mid-span of the spandrel beam are going to support the most axial compression load, the reaction at the mid-span of the beam must be obtained. The beam was modeled as a two span continuous beam, pinned at the ends. Depending on the particular bay configuration the reaction at the mid-span was calculated and used as the axial compressive load applied on the stud. With this 100% of the factored live load is applied to the spandrel beam to obtain the axial compression load of the CFS wall stud. This compressive force was compared to the nominal compressive force of the CFS stud obtained from the CFS Program; which always came out to be greater than it. Therefore the nominal axial compressive force

was used to determine the required reaction at the mid-span of the beam by simply rearranging the equation used. This required reaction is the maximum axial compressive force the CFS stud can support. This force is divided by 100% of the applied load on the spandrel beam to obtain the live load percentage the CFS wall stud can carry due to axial load alone. At this point the wall stud is only checked for the axial compressive force.

Next, the stud is checked for combined axial loading and bending. AISI S211-07 states that a wall stud subjected to combined bending and axial loads should be analyzed using the Main Wind Force Resisting System (MWFRS) wind loads as noted in section 3.1.1. Thus, with the proposed buildings parameters and assumptions the MWFRS wind load was calculated to be 7.97 psf (See appendix A). Following AISI S100-07, section C5.2.2 for combined compressive axial loads and bending, interaction checks are computed (See appendix C for calculation on each bay configuration). The checks are made 100% of the live load. In all cases the studs were not adequate for the load applied, so an iterative process was used to determine the maximum axial live load percentage that could be carried by the stud due to combined bending and axial compression. Once this percentage was obtained it was compared to the percentage due to axial load alone, and a correlation was observed between them. This relationship helped establish ratios, known as reduction factors, between the two percentages, which can determine the reduced deflection gap.

The building sections analyzed in the report are divided into two main cases: the second level studs of the building that are 12-ft high and the third level of the building that are 10-ft high. For each case the different bay conditions are analyzed twice. One for a 50 psf live load applied at the floor system supported by the spandrel beam and the other live load condition is with 80 psf. Table 5.1 summarizes the cases with the different bay systems.

Table 5.1: Case Studies

Case 1:	Bay Size:	25' X 30'	30' X 30'	30' X 30'	40' X 30'	40' X 30'
	Wall Height:	12 ft				
	Floor system:	2 joists	2 joists	3 joists	3 joists	Concrete slab
	Load Case 1:	50 psf				
	Load Case 2:	80 psf				
Case 2:	Bay Size:	25' X 30'	30' X 30'	30' X 30'	40' X 30'	40' X 30'
	Wall Height:	10 ft				
	Floor system:	2 joists	2 joists	3 joists	3 joists	Concrete slab
	Load Case 1:	50 psf				
	Load Case 2:	80 psf				

5.4 Case 1 – 12' Wall

Case one considers studs at the second level of the building and is analyzing the exterior studs at all five bays in question. For the 12ft high stud with an out of plane wind load of 16.23psf the stud selected from the SSMA specification catalogs in the curtain wall tables was a 600S162-33. See in appendix C for the analysis calculations.

5.4.1 25' X 30' Bay – 2 floor joists

In bay (a) at the second floor the spandrel beam is supporting two equally spaced floor joist that carry a 50psf live load and a 80psf live load. For both loads the reactions at the mid-span of the beam are calculated and the live load percentage capacity from just axial load and combined axial load and bending that the stud can carry are obtained.

5.4.1.1 Case with Live load equal to 50 psf

For the 50psf live load, the percentage due to just the axial load capacity is 25.16% and 48.35% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{ Due to combine loading}}{\% \text{ Due to axial load}} = \frac{48.35}{25.16} = 1.92$$

5.4.1.2 Case with Live load equal to 80 psf

For the 80psf live load, the percentage due to just the axial load capacity is 15.73% and 30.22% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{30.22}{15.73} = 1.92$$

5.4.2 30' X 30' Bay – 2 floor joists

In bay (b) at the second floor the spandrel beam is supporting two equally spaced floor joist that carry a 50psf live load and a 80psf live load. For both loads the reactions at the mid-span of the beam are calculated and the live load percentage capacity from just axial load and combined axial load and bending that the stud can carry are obtained.

5.4.2.1 Case with Live load equal to 50 psf

For the 50psf live load, the percentage due to just the axial load capacity is 20.97% and 40.29% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{40.29}{20.97} = 1.92$$

5.4.2.2 Case with Live load equal to 80 psf

For the 80psf live load, the percentage due to just the axial load capacity is 13.10% and 25.18% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{25.18}{13.10} = 1.92$$

5.4.3 30' X 30' Bay – 3 floor joists

In bay (c) at the second floor the spandrel beam is supporting two equally spaced floor joist that carry a 50psf live load and a 80psf live load. For both loads the reactions at the mid-

span of the beam are calculated and the live load percentage capacity from just axial load and combined axial load and bending that the stud can carry are obtained.

5.4.3.1 Case with Live load equal to 50 psf

For the 50psf live load, the percentage due to just the axial load capacity is 20.05% and 38.54% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{38.54}{20.05} = 1.92$$

5.4.3.2 Case with Live load equal to 80 psf

For the 80psf live load, the percentage due to just the axial load capacity is 12.53% and 24.09% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{24.09}{12.53} = 1.92$$

5.4.4 40' X 30' Bay – 3 floor joists

In bay (d) at the second floor the spandrel beam is supporting two equally spaced floor joist that carry a 50psf live load and a 80psf live load. For both loads the reactions at the mid-span of the beam are calculated and the live load percentage capacity from just axial load and combined axial load and bending that the stud can carry are obtained.

5.4.4.1 Case with Live load equal to 50 psf

For the 50psf live load, the percentage due to just the axial load capacity is 15.04% and 28.90% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{28.90}{15.04} = 1.92$$

5.4.4.2 Case with Live load equal to 80 psf

For the 80psf live load, the percentage due to just the axial load capacity is 9.40% and 18.07% for the axial capacity when looking at combined axial compression load and bending. The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{18.07}{9.40} = 1.92$$

5.4.5 40' X 30' Bay – Concrete Slab Floor System

In bay (e) at the second floor the spandrel beam is supporting two equally spaced floor joist that carry a 50psf live load and a 80psf live load. For both loads the reactions at the mid-span of the beam are calculated and the live load percentage capacity from just axial load and combined axial load and bending that the stud can carry are obtained.

5.4.5.1 Case with Live load equal to 50 psf

For the 50psf live load, the percentage due to just the axial load capacity is 14.29% and 27.46% for the axial capacity when looking at combined axial compression load and bending. The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{27.46}{14.29} = 1.92$$

5.4.5.2 Case with Live load equal to 80 psf

For the 80psf live load, the percentage due to just the axial load capacity is 8.93% and 17.16% for the axial capacity when looking at combined axial compression load and bending. The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{17.16}{8.93} = 1.92$$

5.5 Case 2 – 10' Wall

Case one considers at the second level of the building and is analyzing the exterior studs at all five bays in question. For the 10 ft high stud with an out of plane wind load of 16.23psf

the stud selected from the SSMA specification catalogs in the curtain wall tables; which was a 600S162-33. See in appendix C for the analysis calculations.

5.5.1 25' X 30' Bay – 2 floor joists

In bay (a) at the second floor the spandrel beam is supporting two equally spaced floor joist that carry a 50psf live load and a 80psf live load. For both loads the reactions at the mid-span of the beam are calculated and the live load percentage capacity from just axial load and combined axial load and bending that the stud can carry are obtained.

5.5.1.1 Case with Live load equal to 50 psf

For the 50psf live load, the percentage due to just the axial load capacity is 21.70% and 26.73% for the axial capacity when looking at combined axial compression load and bending. The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{26.73}{21.70} = 1.23$$

5.5.1.2 Case with Live load equal to 80 psf

For the 80psf live load, the percentage due to just the axial load capacity is 13.56% and 16.71% for the axial capacity when looking at combined axial compression load and bending. The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{16.71}{13.56} = 1.23$$

5.5.2 30' X 30' Bay – 2 floor joists

In bay (b) at the second floor the spandrel beam is supporting two equally spaced floor joist that carry a 50psf live load and a 80psf live load. For both loads the reactions at the mid-span of the beam are calculated and the live load percentage capacity from just axial load and combined axial load and bending that the stud can carry are obtained.

5.5.2.1 Case with Live load equal to 50 psf

For the 50psf live load, the percentage due to just the axial load capacity is 18.08% and 22.28% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{22.28}{18.08} = 1.23$$

5.5.2.2 Case with Live load equal to 80 psf

For the 80psf live load, the percentage due to just the axial load capacity is 11.30% and 13.92% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{13.92}{11.30} = 1.23$$

5.5.3 30' X 30' Bay – 3 floor joists

In bay (c) at the second floor the spandrel beam is supporting two equally spaced floor joist that carry a 50psf live load and a 80psf live load. For both loads the reactions at the mid-span of the beam are calculated and the live load percentage capacity from just axial load and combined axial load and bending that the stud can carry are obtained.

5.5.3.1 Case with Live load equal to 50 psf

For the 50psf live load, the percentage due to just the axial load capacity is 17.30% and 21.31% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{21.31}{17.30} = 1.23$$

5.5.3.2 Case with Live load equal to 80 psf

For the 80psf live load, the percentage due to just the axial load capacity is 10.81% and 13.32% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{13.32}{10.81} = 1.23$$

5.5.4 40' X 30' Bay – 3 floor joists

In bay (d) at the second floor the spandrel beam is supporting two equally spaced floor joist that carry a 50psf live load and a 80psf live load. For both loads the reactions at the mid-span of the beam are calculated and the live load percentage capacity from just axial load and combined axial load and bending that the stud can carry are obtained.

5.5.4.1 Case with Live load equal to 50 psf

For the 50psf live load, the percentage due to just the axial load capacity is 12.97% and 15.98% for the axial capacity when looking at combined axial compression load and bending. The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{15.98}{12.97} = 1.23$$

5.5.4.2 Case with Live load equal to 80 psf

For the 80psf live load, the percentage due to just the axial load capacity is 8.11% and 9.99% for the axial capacity when looking at combined axial compression load and bending. The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{9.99}{8.11} = 1.23$$

5.5.5 40' X 30' Bay – Concrete Slab Floor System

In bay (e) at the second floor the spandrel beam is supporting two equally spaced floor joist that carry a 50psf live load and a 80psf live load. For both loads the reactions at the mid-span of the beam are calculated and the live load percentage capacity from just axial load and combined axial load and bending that the stud can carry are obtained.

5.5.5.1 Case with Live load equal to 50 psf

For the 50psf live load, the percentage due to just the axial load capacity is 12.32% and 15.18% for the axial capacity when looking at combined axial compression load and bending.

The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{15.18}{12.32} = 1.23$$

5.5.5.2 Case with Live load equal to 80 psf

For the 80psf live load, the percentage due to just the axial load capacity is 7.70% and 9.49% for the axial capacity when looking at combined axial compression load and bending. The relationship between the two percentages is seen below:

$$\frac{\% \text{Due to combine loading}}{\% \text{Due to axial load}} = \frac{9.49}{7.70} = 1.23$$

6 RESULTS

Examining the correlation between the axial loading and the combined axial load and bending presented in Part 5, a common factor can be established for each stud size. The live load percentage due to axial load alone; which is the maximum load possible on the stud if there were no deflection gap (based on the nominal axial strength of the stud), is divided by the maximum allowable applied load on the spandrel beam. This is directly proportional to the combined axial load and bending percentages. The live load percent due to combined axial and bending is the maximum axial load allowed on the stud when bending forces are applied due to wind load on the stud if there were no deflection gap. This percentage is then divided by the percentage due to axial load alone allowable by the nominal compressive strength of the stud.

All of this established a common factor between the two percentages that is referred to as the reduction factor. The reduction factor, or multiplier, is used to determine the deflection gap reduction percentage. The factor came out to be the same each stud regardless of the loading condition and bay sizes. This relationship was then used as a set guideline for determining the percentage the deflection gap between the stud and the track can be reduced.

6.1 Case 1 – 12' Wall

For the 600S162-33 12' tall wall stud at the second level of the building the available strengths are summarized in the following tables. Table 6.1 is for the 50 psf floor live load and the Table 6.2 is for the 80 psf floor live load.

Table 6.1: Results for 600S162-33 with 50psf LL

		Stud: 600S162-33 LL : 50 pfs			
Cases:		25' X 30' - 2 Joists	30' X 30' - 2 Joists	30' X 30' - 3 Joists	
Axial Only	Applied load @ 100% LL	6.25 K	7.50 K	5.63 K	unfactored
	Max allow. load on Stud	1.57 K	1.57 K	1.13 K	unfactored
	% of P	25%	21%	20%	
	Factored force(P)	2.52 K	2.52 K	1.80 K	factored
Axial + Bending	R ₂ @ 100% LL	10.65 K	12.78 K	13.36 K	unfactored
	Max allow. R ₂ on stud	5.15 K	5.15 K	5.15 K	unfactored
	% of R ₂	48%	40%	39%	
	P _u	2.58 K	2.58 K	2.58 K	factored
	φP _n	4.29 K	4.29 K	4.29 K	
	M _u	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	
	φM _n	855.57 lb-ft	855.57 lb-ft	855.57 lb-ft	
		1.92	1.92	1.92	
Cases:		40' X 30' - 3 Joists	40' X 30' - Uniform		
Axial Only	Applied load @ 100% LL	7.50 K	0.75 Klif		unfactored
	Max allow. load on Stud	1.13 K	0.11 Klif		unfactored
	% of P	15%	14%		
	Factored force(P)	1.80 K	0.17 Klif		factored
Axial + Bending	R ₂ @ 100% LL	17.81 K	18.75 K		unfactored
	Max allow. R ₂ on stud	5.15 K	5.15 K		unfactored
	% of R ₂	29%	27%		
	P _u	2.58 K	2.58 K		factored
	φP _n	4.29 K	4.29 K		
	M _u	306.05 lb-ft	306.05 lb-ft		
	φM _n	855.57 lb-ft	855.57 lb-ft		
		1.92	1.92		

Table 6.2: Results for 600S162-33 with 80psf LL

		Stud: 600S162-33 LL : 80 pfs					
		Cases:	25' X 30' - 2 Joists	30' X 30' - 2 Joists	30' X 30' - 3 Joists		
Axial Only	Applied load @ 100% LL		10.00 K	12.00 K	9.00 K	unfactored	
	Max allow. load on Stud		1.57 K	1.57 K	1.13 K	unfactored	
	% of P		16%	13%	13%		
	Factored force(P)		2.52 K	2.52 K	1.80 K	factored	
Axial + Bending	R ₂ @ 100% LL		17.04 K	20.44 K	21.38 K	unfactored	
	Max allow. R ₂ on stud		5.15 K	5.15 K	5.15 K	unfactored	
	% of R ₂		30%	25%	24%		
	P _u		2.57 K	2.57 K	2.57 K	factored	
	φP _n		4.29 K	4.29 K	4.29 K		
	M _u		306.05 lb-ft	306.05 lb-ft	306.05 lb-ft		
	φM _n		855.57 lb-ft	855.57 lb-ft	855.57 lb-ft		
			1.92	1.92	1.92		
		Cases:	40' X 30' - 3 Joists	40' X 30' - Uniform			
Axial Only	Applied load @ 100% LL		12.00 K	1.20 Klif	unfactored		
	Max allow. load on Stud		1.13 K	0.11 Klif	unfactored		
	% of P		9%	9%			
	Factored force(P)		1.80 K	0.17 Klif	factored		
Axial + Bending	R ₂ @ 100% LL		28.50 K	30.00 K	unfactored		
	Max allow. R ₂ on stud		5.15 K	5.15 K	unfactored		
	% of R ₂		18%	17%			
	P _u		2.57 K	2.57 K	factored		
	φP _n		4.29 K	4.29 K			
	M _u		306.05 lb-ft	306.05 lb-ft			
	φM _n		855.57 lb-ft	855.57 lb-ft			
			1.92	1.92			

As shown in these tables, the constant ratios between the axial capacity percentage and the axial capacity percentage for compression plus bending for this stud size at 12' height is 1.92. This ratio is a reduction factor that can be multiplied by the “% of P” value for axial loading to arrive at the reduced percentage of the deflection gap. In other words, the “% of R₂”

value from the combined axial compressive force and bending is the percentage the deflection gap can be reduced. Since the process of actually determining the deflection gap percentage was an iterative process the reduction factors make it possible to eliminate the iteration by multiplying the reduction factor by the live load percentage due to the axial capacity assuming that is the governing load case for the stud. For each case analyzed a summarization can be made for each bay condition to compare the correlation of the analysis at hand.

6.2 Case 2 – 10' Wall

For the 362S162-33 10' tall wall stud at the third level of the building the capacities can be collaborated in to the following tables. The first table is for the 50psf floor live load applied and the second table is for the 80psf floor live load applied.

Table 6.3: Results for 362S162-33 50psf LL

		Stud: 362S162-33 LL : 50 pfs					
		Cases:	25' X 30' - 2 Joists	30' X 30' - 2 Joists	30' X 30' - 3 Joists		
Axial Only	Applied load @ 100% LL		6.25 K	7.50 K	5.63 K	unfactored	
	Max allow. load on Stud		1.36 K	1.36 K	0.97 K	unfactored	
	% of P		22%	18%	17%		
	Factored force(P)		2.17 K	2.17 K	1.56 K	factored	
Axial + Bending	R ₂ @ 100% LL		10.65 K	12.78 K	13.36 K	unfactored	
	Max allow. R ₂ on stud		2.85 K	2.85 K	2.85 K	unfactored	
	% of R ₂		27%	22%	21%		
	P _u		1.42 K	1.42 K	1.42 K	factored	
	φP _n		3.70 K	3.70 K	3.70 K		
	M _u		212.53 lb-ft	212.53 lb-ft	212.53 lb-ft		
	φM _n		396.77 lb-ft	396.77 lb-ft	396.77 lb-ft		
			1.23	1.23	1.23		
		Cases:	40' X 30' - 3 Joists	40' X 30' - Uniform			
Axial Only	Applied load @ 100% LL		7.50 K	0.75 Klif	unfactored		
	Max allow. load on Stud		0.97 K	0.09 Klif	unfactored		
	% of P		13%	12%			
	Factored force(P)		1.56 K	0.15 Klif	factored		
Axial + Bending	R ₂ @ 100% LL		17.81 K	18.75 K	unfactored		
	Max allow. R ₂ on stud		2.85 K	2.85 K	unfactored		
	% of R ₂		16%	15%			
	P _u		1.42 K	1.42 K	factored		
	φP _n		3.70 K	3.70 K			
	M _u		212.53 lb-ft	212.53 lb-ft			
	φM _n		396.77 lb-ft	396.77 lb-ft			
			1.23	1.23			

Table 6.4: Results for 362S162-33 80psf LL

		Stud: 362S162-33 LL : 80 pfs				
		Cases:	25' X 30' - 2 Joists	30' X 30' - 2 Joists	30' X 30' - 3 Joists	
Axial Only	Applied load @ 100% LL		10.00 K	12.00 K	9.00 K	unfactored
	Max allow. load on Stud		1.36 K	1.36 K	0.97 K	unfactored
	% of P		14%	11%	11%	
	Factored force(P)		2.17 K	2.17 K	1.56 K	factored
Axial + Bending	R ₂ @ 100% LL		17.04 K	20.44 K	21.38 K	unfactored
	Max allow. R ₂ on stud		2.85 K	2.85 K	2.85 K	unfactored
	% of R ₂		17%	14%	13%	
	P _u		1.42 K	1.42 K	1.42 K	factored
	φP _n		3.70 K	3.70 K	3.70 K	
	M _u		212.53 lb-ft	212.53 lb-ft	212.53 lb-ft	
	φM _n		396.77 lb-ft	396.77 lb-ft	396.77 lb-ft	
			1.23	1.23	1.23	
		Cases:	40' X 30' - 3 Joists	40' X 30' - Uniform		
Axial Only	Applied load @ 100% LL		12.00 K	1.20 KlF		unfactored
	Max allow. load on Stud		0.97 K	0.09 KlF		unfactored
	% of P		8%	8%		
	Factored force(P)		1.56 K	0.15 KlF		factored
Axial + Bending	R ₂ @ 100% LL		28.50 K	30.00 K		unfactored
	Max allow. R ₂ on stud		2.85 K	2.85 K		unfactored
	% of R ₂		10%	9%		
	P _u		1.42 K	1.42 K		factored
	φP _n		3.70 K	3.70 K		
	M _u		212.53 lb-ft	212.53 lb-ft		
	φM _n		396.77 lb-ft	396.77 lb-ft		
			1.23	1.23		

For this case, the reduction factor is 1.23 and is constant for this stud size at this height.

The same observations made in the previous case apply also to this case.

6.3 Analysis of Results

An analysis of the results shows that the percentage of the total axial load that can be supported by the stud due to combine axial compression and bending is the actual percentage the deflection gap can be reduced. From the above tables, a direct correlation between the axial capacity of the stud and the combined axial and bending capacity of the stud can be seen. It can also be seen that the manner in which the load is applied and the amount of load applied has no direct impact in the reduction factors. It should also be noted that if the height of the stud or thickness of the steel changes the reduction factor will also change. This was obtained by changing the wall stud heights for each condition. Table 8.1 shows this correlation. The taller the stud is, the smaller the reduction factor and thus the smaller the percentage of the reduction to the deflection gap.

7 CONCLUSION

It has been shown that curtain wall studs originally sized only for out of plane wind loads have an available axial strength that can be taken into account to reduce the size of the deflection gap below a spandrel beam that would normally be required. It has also been shown that for the load case of axial compression only, the percentage of the total axial load the stud can support provides the percentage reduction for the deflection gap for this load case. Additionally, for the load case of bending and axial compression, the percentage of the total axial load the stud can support when combined with bending can also be determined. The latter percentage represents the governing percent reduction for the deflection gap. The relationship of the percentage of the total load the stud can support between the two load cases is a constant and thus eliminating the need to determine the available axial strength of the stud for combined axial compression and bending and merely apply the reduction factor to the percentage of the total axial load the stud can support for axial compression only to determine the final percent reduction of the deflection gap. This eliminates the need for an iterative approach to determine the reduction in the deflection gap.

This study shows that the manner in which the live load is applied to the spandrel beam does not affect the correlation between the two load cases nor does the magnitude of the live load applied. The reason for this is because the capacity of the stud is the same no matter what the load is or how it is applied. The material and member properties of the stud are the governing elements of the analysis.

Apart from the study and the analysis conducted, the probability of the having the full live load applied near the exterior of the building should be considered. As stated in section 2, the probability that even 75% of the live load is accumulated in the perimeter of the building is small (AISC, 2003). Noting that in any type of building, the areas close to the walls, especially if there are window, will not have a lot of activity around them. With this the feasibility of reducing the deflection gap due to live load is even more possible.

8 RECOMMENDATIONS

This study shows that a prescriptive method to determine a reduced deflection gap can be established. Tables can be generated that list the reduction factors for a particular stud at a particular height. In this study since the type of building considered is an office building a Table 8.1 has been created that lists common stud sizes used and their corresponding reduction factors for the deflection gap due to live loads for an office building. Limitations are also listed for the use of the table. The table shown is for uniform wind loads of 5, 10, 15, 20, 25 and 30 psf. The wind forces are based on MWFRS wind loads since the combined axial load and bending load case governs the overall analysis. Additional tables can be derived for different wind loads and stud sizes that may be applied to a structure. To make the tables easy to follow they are provided in increments of 5psf for the varying wind loads similar to the load and span tables typically provided by manufacturers for CFS curtain wall studs. Interpolation between tables can then be done to determine the exact reduction factor that is desired for a wind force; since there is a semi-linear relationship between the two percentages.

Section 8.1 provides a step by step procedure to determine the reduced deflection gap on an exterior wall. Table 8.1 is used to obtain the reduction factors needed to design the reduced gap. The method is intended for office buildings and that support live loads of 100 psf or less. Modifications can also be made to the procedure as long as all assumptions are stated clearly and the tables are recreated for the situation specific to the structure.

Table 8.1: Reduction Factors

Stud	Max Height @ (L/360)	Wall Height (ft)	Reduction Factor					
			Wind Force (MWFRS)					
			5 psf	10 psf	15 psf	20 psf	25 psf	30 psf
362S162-33	11' - 11"	10	1.86	0.85	--	--	--	--
362S200-33	12' - 7"	10	1.97	1.01	0.2	--	--	--
		12	1.47	0.28	--	--	--	--
362S162-43	13' - 0"	10	2.17	1.37	0.68	0.07	--	--
		12	1.74	0.73	--	--	--	--
362S200-43	13' - 9"	10	2.29	1.56	0.94	0.38	--	--
		12	1.89	0.96	0.2	--	--	--
362S162-54	13' - 11"	10	2.34	1.65	1.07	0.55	0.08	--
		12	1.96	1.09	0.38	--	--	--
362S200-54	14' - 9"	10	2.46	1.86	1.34	0.87	0.45	0.05
		12	2.12	1.34	0.7	0.14	--	--
		14	1.78	0.84	0.09	--	--	--
600S162-33	17' - 11"	10	2.64	2.11	1.6	1.11	0.64	0.18
		12	2.37	1.62	0.93	0.28	--	--
		14	2.07	1.09	0.22	--	--	--
		16	1.73	0.54	--	--	--	--
600S200-33	18' - 7"	10	2.68	2.18	1.71	1.25	0.8	0.38
		12	2.43	1.73	1.08	0.46	--	--
		14	2.14	1.22	0.4	--	--	--
		16	1.81	0.69	--	--	--	--
		18	1.46	0.15	--	--	--	--
600S162-43	19' - 4"	10	2.81	2.43	2.07	1.72	1.38	1.05
		12	2.62	2.08	1.58	1.1	0.65	0.21
		14	2.39	1.68	1.03	0.44	--	--
		16	2.14	1.25	0.47	--	--	--
		18	1.86	0.81	--	--	--	--
600S200-43	20' - 3"	10	2.82	2.45	2.1	1.76	1.43	1.11
		12	2.63	2.11	1.62	1.16	0.71	0.3
		14	2.41	1.71	1.09	0.51	--	--
		16	2.15	1.29	0.54	--	--	--
		18	1.88	0.86	--	--	--	--
		20	1.6	0.43	--	--	--	--
600S162-54	20' - 8"	10	2.89	2.58	2.29	2.01	1.73	1.46
		12	2.73	2.29	1.88	1.49	1.11	0.76
		14	2.54	1.95	1.42	0.93	0.46	0.03
		16	2.32	1.59	0.94	0.35	--	--
		18	2.09	1.21	0.46	--	--	--
		20	1.85	0.83	--	--	--	--
600S200-54	21' - 9"	10	2.92	2.65	2.39	2.14	1.89	1.65
		12	2.78	2.39	2.02	1.67	1.33	1.01
		14	2.61	2.08	1.6	1.15	0.73	0.34
		16	2.41	1.74	1.16	0.62	0.13	--
		18	2.19	1.39	0.71	0.1	--	--
		20	1.97	1.04	0.27	--	--	--

**Table is based on 90 MPH Wind and Exposure B for both C&C and MWFRS

**Table lists the most common used studs for steel frame infill studs

**Spacing of stud is 16" o.c. And stays constant

** Governing stud deflection Deflection is L/360

** Max Height is based on the use of C&C wind force

** Floor and roof liveloads should not exceed 100 psf

8.1 Step by Step Procedure

With the results and analysis of this report; it is recommend to use the following procedure and assumptions to determine the percentage the deflection gap can be reduced due to the live load induced on the spandrel beam:

Assumptions:

- Only the stud at the mid-span of the spandrel beam is loaded
- The wall stud is loaded concentrically
- P-Delta Effects are neglected
- Axial load governs initially
- Wind force is uniform throughout the length of the CFS stud
- Worst case scenario is at the mid-span of the beam
- Use LRFD for design
- Dead load gap is assumed to stay constant and will not be reduced
- Height of the wall is constant
- Spacing of studs is constant throughout the span of the beam
- The live load applied is to be less than 100psf
- The analysis only looks at only a single long leg track construction assembly with bridging at third points on the stud

Procedure:

Step 1: Preliminary stud selection

- Size the CFS stud for only the out of plane wind load (Components & Cladding)
 - *From section A3.1 of AISI S211-07*
- Use the standard SSMA catalogs
- Obtain the design axial compressive strength of the selected stud (ϕP_n)

Step 2: Determine Allowable Load on Beam

- Depending on the static loading condition of the spandrel beam, obtain the interior beam reaction on the stud at the mid-span of the beam (worst case)
- Substitute the available axial strength from step 1 in compression for the reaction.
- Rearrange equation to find the allowable load that the beam can carry.
- Note that the load obtained is factored.

Step 3: Determine Percentage of available strength to the applied load

- Calculate the factored design live load supported by the beam (1.6L)
- Divide the available strength by the design load and multiply by 100 to get percentage.

Step 4: Look up Factor from Table 8.1

- Look up factor
- Factor is based on the material properties of the stud and the height of the wall

Step 5: Obtain percentage the live load gap can be reduced by

- Multiply factor by the percentage calculated in step 3

Step 6: Check Combine Axial and Bending load

- Obtain governing load combination (1.2D + 1.6W + L)
- For the bending load on the stud use the in-plane wind load (MWFRS)
 - See Section A3.1 of AISI S211-07
- Use the percentage that the live load can be reduced by in step 5 to obtain the axial factored load for the load combination.
- Follow the procedure in AISI S100-07 for Combined Axial and Bending
 - Section C5.2.2
- Check interaction equation result to be less than or equal to 1.0
 - Equations C5.2.2-1, C5.2.2-2, C5.2.2-3
- If check is met proceed to step 7

- If not, change stud thickness

Step 7: Determine reduced deflection gap

- Calculate the 100% live load deflection gap for the beam ($L/240$, $L/360$, etc.)
- Multiply it by 1 minus the reduction amount obtained in step 5

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APPENDIX A - WIND LOAD CALCULATIONS

Wind Load

IBC 2006 code & ASCE7-05 manual are used for the design

6.5 METHOD 2- ANALYTICAL PROCEDURE

Basic wind speed (3 sec gust) = 90 MPH
 Length of Building = 95 ft
 Width of Building = 60 ft
 Exposure = B
 Roof Pitch = 0.00 :12
 Mean Roof Height h = 44 ft
 Importance factor I_w = 1.00 T-6-1

6.5.12.2.2 Low-Rise Building

Velocity pressure $q_z = .00256 K_z K_{zt} K_d V^2$ (6-15)

$K_{zt} = 1.00$ $K_d = 0.85$ $V = 90$ $I_w = 1.00$

at mean roof high h = 44

$K_h = 0.78$ Exp = B T-6-3

Roof angle $\theta = 0.0$

$p = q_h[(GC_{pf}) - (GC_{pi})]$ (6-18)

$q_h = 13.75$

Internal Pressure Coef (GC_{pi}) = ± 0.18 Enclosed Building Figure 6-5

2a = 12 ft

Figure 6-10 MWFRS Method 2

Load Direction	Equations	Building Surface									
		1	2	3	4	5	6	1E	2E	3E	4E
Trans	GC_{pf}	0.40	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43
Long	GC_{pf}	0.40	-0.69	-0.37	-0.29	-0.45	-0.45	0.61	-1.07	-0.53	-0.43
Trans	$q_h(GC_{pf} + GC_{pi})$	7.97	-7.01	-2.61	-1.51	-3.71	-3.71	10.86	-12.24	-4.81	-3.44
	$q_h(GC_{pf} - GC_{pi})$	3.02	-11.96	-7.56	-6.46	-8.66	-8.66	5.91	-17.18	-9.76	-8.39
Long	$q_h(GC_{pf} + GC_{pi})$	7.97	-7.01	-2.61	-1.51	-3.71	-3.71	10.86	-12.24	-4.81	-3.44
	$q_h(GC_{pf} - GC_{pi})$	3.02	-11.96	-7.56	-6.46	-8.66	-8.66	5.91	-17.18	-9.76	-8.39

6.5.12.4 COMPONENTS AND CLADDING

6.5.12.4.1 Low-rise building and building with $h \leq 60$ ft

Velocity pressure $q_z = .00256 K_z K_{zt} K_d I_w V^2$ (6-15)

$K_{zt} = 1.00$ $K_d = 0.85$ $V = 90$ $I_w = 1.00$

$q_z = 17.63 K_z$

at mean roof high h = 44.0 ft OK

$K_h = 0.78$ Exp = B

velocity at mean roof height, $q_h = 13.75$

$p = q_h(GC_p - GC_{pi})$ (6-22)

External pressure, $+GC_p = 0.9$ $-GC_p = -1$ Figure 6.11 through 6-16

Internal pressure, $G_{pi} = \pm 0.18$ Enclosed Building Figure 6-5

$p = 13.75[0.9 - (\pm 0.18)]$ psf $13.75[-1 - (\pm 0.18)]$ psf

$p = 14.85$ psf -16.23 psf

APPENDIX B – CFS PROGRAM CALCULATIONS

Section Inputs 600S162-33

Material: A653 SS Grade 33
 No strength increase from cold work of forming.
 Modulus of Elasticity, E 29500 ksi
 Yield Strength, Fy 33 ksi
 Tensile Strength, Fu 45 ksi
 Warping Constant Override, Cw 0 in⁶
 Torsion Constant Override, J 0 in⁴

Stiffened Channel, Thickness 0.0346 in (20 Gage)
 Placement of Part from Origin:
 X to center of gravity 0 in
 Y to center of gravity 0 in

Outside dimensions, Open shape

	Length (in)	Angle (deg)	Radius (in)	Web	k Coef.	Hole Size (in)	Distance (in)
1	0.5000	270.000	0.076500	None	0.000	0.0000	0.2500
2	1.6250	180.000	0.076500	Single	0.000	0.0000	0.8125
3	6.0000	90.000	0.076500	Cee	0.000	0.0000	3.0000
4	1.6250	0.000	0.076500	Single	0.000	0.0000	0.8125
5	0.5000	-90.000	0.076500	None	0.000	0.0000	0.2500

Member Check - 2007 North American Specification - US (LRFD)

Material Type: A653 SS Grade 33, Fy=33 ksi
 Design Parameters:
 Lx 12.000 ft Ly 4.000 ft Lt 4.000 ft
 Kx 1.0000 Ky 1.0000 Kt 1.0000
 Cbx 1.0000 Cby 1.0000 ex 0.0000 in
 Cmx 1.0000 Cmy 1.0000 ey 0.0000 in
 Braced Flange: None Red. Factor, R: 0 Stiffness, kφ: 0 k

Loads:

	P (k)	Mx (k-in)	Vy (k)	My (k-in)	Vx (k)
Entered	2.5700	0.000	0.0000	0.000	0.0000
Applied	2.5700	0.000	0.0000	-0.305	0.0000
Strength	4.2866	14.239	0.9697	2.590	1.8260

Effective section properties at applied loads:
 Ae 0.24158 in² Ixe 1.7175 in⁴ Iye 0.0933 in⁴
 Sxe (t) 0.57249 in³ Sye (l) 0.16061 in³
 Sxe (b) 0.57249 in³ Sye (r) 0.08941 in³

Interaction Equations
 NAS Eq. C5.2.2-1 (P, Mx, My) 0.600 + 0.000 + 0.143 = 0.742 ≤ 1.0
 NAS Eq. C5.2.2-2 (P, Mx, My) 0.462 + 0.000 + 0.118 = 0.580 ≤ 1.0
 NAS Eq. C3.3.2-1 (Mx, Vy) Sqrt(0.000 + 0.000) = 0.000 ≤ 1.0
 NAS Eq. C3.3.2-1 (My, Vx) Sqrt(0.014 + 0.000) = 0.118 ≤ 1.0

Fully Braced Strength - 2007 North American Specification - US (LRFD)

Material Type: A653 SS Grade 33, Fy=33 ksi

	Compression	Positive Moment	Positive Moment
φPno	5.561 k	φMnxo 18.115 k-in	φMnyo 2.849 k-in
Ae	0.19824 in ²	Ixe 1.7553 in ⁴	Iye 0.1163 in ⁴
		Sxe (t) 0.57784 in ³	Sye (l) 0.28154 in ³
		Sxe (b) 0.59252 in ³	Sye (r) 0.09591 in ³
Tension		Negative Moment	Negative Moment
φTn	10.225 k	φMnxo 18.115 k-in	φMnyo 2.590 k-in
		Ixe 1.7553 in ⁴	Iye 0.0870 in ⁴
Shear		Sxe (t) 0.59252 in ³	Sye (l) 0.13851 in ³
φVny	0.970 k	Sxe (b) 0.57784 in ³	Sye (r) 0.08722 in ³
φVnx	1.826 k		

Section Inputs 362S162-33

Material: A653 SS Grade 33

No strength increase from cold work of forming.

Modulus of Elasticity, E 29500 ksi
 Yield Strength, Fy 33 ksi
 Tensile Strength, Fu 45 ksi
 Warping Constant Override, Cw 0 in⁶
 Torsion Constant Override, J 0 in⁴

Stiffened Channel, Thickness 0.0346 in (20 Gage)

Placement of Part from Origin:

X to center of gravity 0 in
 Y to center of gravity 0 in

Outside dimensions, Open shape

	Length (in)	Angle (deg)	Radius (in)	Web	k Coef.	Hole Size (in)	Distance (in)
1	0.5000	270.000	0.076500	None	0.000	0.0000	0.2500
2	1.6250	180.000	0.076500	Single	0.000	0.0000	0.8125
3	3.6250	90.000	0.076500	Cee	0.000	0.0000	1.8125
4	1.6250	0.000	0.076500	Single	0.000	0.0000	0.8125
5	0.5000	-90.000	0.076500	None	0.000	0.0000	0.2500

Member Check - 2007 North American Specification - US (LRFD)

Material Type: A653 SS Grade 33, Fy=33 ksi

Design Parameters:

Lx 10.0000 ft Ly 3.3300 ft Lt 3.3300 ft
 Kx 1.0000 Ky 1.0000 Kt 1.0000
 Cbx 1.0000 Cby 1.0000 ex 0.0000 in
 Cmx 1.0000 Cmy 1.0000 ey 0.0000 in
 Braced Flange: None Red. Factor, R: 0 Stiffness, kφ: 0 k

Loads: P Mx Vy My Vx
 (k) (k-in) (k) (k-in) (k)
 Entered 1.4200 0.0000 0.0000 0.0000 0.0000
 Applied 1.4200 0.0000 0.0000 -0.0071 0.0000
 Strength 3.6969 8.1781 1.5558 2.5590 1.8260

Effective section properties at applied loads:

Ae 0.25904 in² Ixe 0.55122 in⁴ Iye 0.09851 in⁴
 Sxe(t) 0.30412 in³ Sye(l) 0.18138 in³
 Sxe(b) 0.30412 in³ Sye(r) 0.09106 in³

Interaction Equations

NAS Eq. C5.2.2-1 (P, Mx, My) 0.384 + 0.000 + 0.003 = 0.387 <= 1.0
 NAS Eq. C5.2.2-2 (P, Mx, My) 0.260 + 0.000 + 0.003 = 0.263 <= 1.0
 NAS Eq. C3.3.2-1 (Mx, Vy) Sqrt(0.000 + 0.000) = 0.000 <= 1.0
 NAS Eq. C3.3.2-1 (My, Vx) Sqrt(0.000 + 0.000) = 0.003 <= 1.0

Fully Braced Strength - 2007 North American Specification - US (LRFD)

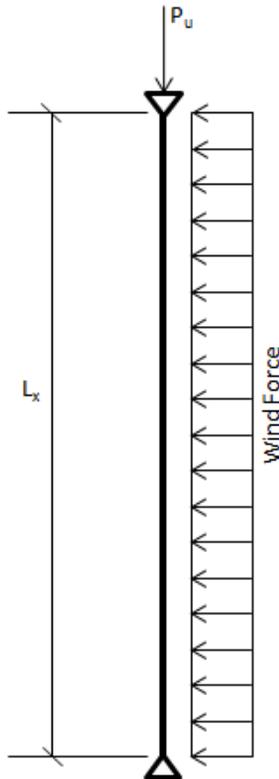
Material Type: A653 SS Grade 33, Fy=33 ksi

Compression Positive Moment Positive Moment
 φPno 5.4607 k φMnxo 9.1647 k-in φMnyo 2.7120 k-in
 Ae 0.19468 in² Ixe 0.53825 in⁴ Iye 0.09935 in⁴
 Sxe(t) 0.29233 in³ Sye(l) 0.18503 in³
 Tension Sxe(b) 0.30175 in³ Sye(r) 0.09131 in³
 φTn 7.7848 k
 Negative Moment Negative Moment
 φMnxo 9.1647 k-in φMnyo 2.5590 k-in
 Shear Ixe 0.53825 in⁴ Iye 0.08411 in⁴
 φVny 1.5558 k Sxe(t) 0.30175 in³ Sye(l) 0.12963 in³
 φVnx 1.8260 k Sxe(b) 0.29233 in³ Sye(r) 0.08616 in³

APPENDIX C – EXCEL SPREADSHEETS

For case 1, the 12' height wall stud at the second level is analyzed and the following are the calculations made in determining the size and capacities of the stud for both load conditions.

CFS Stud Sizing Due to Wind Load Only:



Note: From section A3.1 of the 2007 NASCFSF for wall studs, Component and cladding wind loads are to be used when sizing a wall stud due only to

Stud Spacing:	16 in o.c.	<u>Bridging Spacing</u>
L_x :	12 ft	L_t : 4.00 ft
Wind Load:	16.23 psf (C&C)	L_y : 4.00 ft
Load Factor:	1.6	

Moment in Stud:

$$M_x = \frac{wL^2}{8}$$

$$= \frac{623.23 \text{ lb*ft}}{8} \text{ ----> } 7.48 \text{ k*in}$$

Stud Selection:

From General Specification Catalogs: **600S162-33**
 At 15psf Lateral load & L/360 horizontal
 deflection limit the max span: 17.34 ft
 > Lx Good!

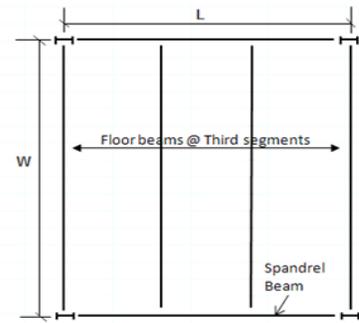
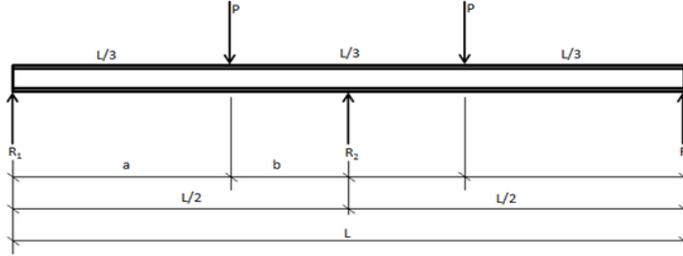
***See attached specification sheet for the studs properties*

Track Selection:

From General Specification Catalogs: **600T200-33**

Stud Capacity: (25' X 30' Bay)

Due to 50 psf live load only



Given: L = 25 ft LL = 50 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 1.793 in⁴
 a = 8.33 ft L_x = 12 ft K_x = 1
 b = 4.17 ft Stud spacing: 16 in K_x*L_x = 144 in
 l = L/2 = 12.5 ft

Live Load: P @ 50% = 3125 lb -----> 5 Kips factored
 P @ 100% = 6250 lb -----> 10 Kips factored

Reactions:

$$R_2 @ 50\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2 \quad R_1 @ 50\% = \sum M_{R1} = 0.741 \text{ kips} \quad R_3 = R_1 = 0.741 \text{ Kips}$$

$$= 8.52 \text{ Kips}$$

$$R_2 @ 100\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2 \quad R_1 @ 100\% = \sum M_{R1} = 5.741 \text{ kips} \quad R_3 = R_1 = 5.741 \text{ Kips}$$

$$= 17.04 \text{ Kips}$$

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 600S162-33

φP_n = 4.2866 K From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

$$P_u = \frac{\phi P_n \cdot (2l^3)}{2(2l^2 + b(l+a))a}$$

$$= 2.52 \text{ kips}$$

Percentages: $\frac{P_u}{P_u @ 100\%} = 40.26\%$ of LL For one stud
 = 25.16% of 1.6LL for one stud Two studs: 50.32% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination: 1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFWF for wall studs.

Loads on stud:	50% LL	100% LL	48.35% LL	
W =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	5.32 K	10.65 K	5.15 K	
P _u (Factored R ₂) =	2.66 K	5.32 K	2.57 K	**check with MathCAD calculations
M _u =	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	**check with MathCAD calculations
α _x =	0.894	0.789	0.898	**check with MathCAD calculations

Stud strength properties:

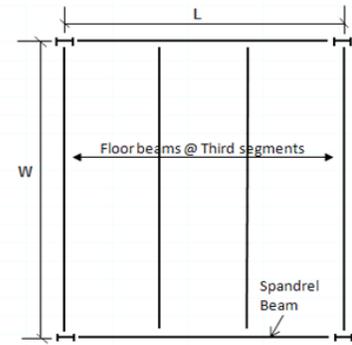
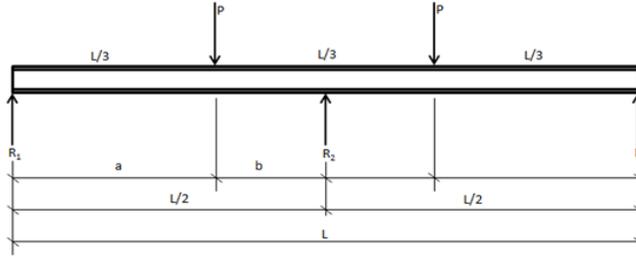
φP_n = 4.2866 K **From CFS program Calculations P_{Ex} = 25.1754 Kips **check with MathCAD calculations
 φM_n = 855.57 lb-ft **From MathCAD Calculation C_m = 1 **From MathCAD Calculation
 φP_{no} = 5.561 K **From CFS program Calculations Ends of beam are pinned-pinned

Interaction Check:

		100% LL	50% LL	48.35% LL	Controlling %
	$\frac{P_u}{\phi P_n}$	1.2420	0.6210	0.6005	
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_n}$	1.600	0.979	0.958	
C5.2.2-3			2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN
2	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_{no}}$	1.315	0.836	0.821	
C5.2.2-2					
3	$\frac{P_u}{\phi P_n} + \frac{C_m \cdot M_u}{\phi M_n \cdot \alpha_x}$	1.696	1.021	0.999	
C5.2.2-1		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK	Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate	

Stud Capacity: (30' X 30' Bay)

Due to 50 psf live load only



Given: L = 30 ft LL = 50 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 1.793 in⁴
 a = 10.00 ft L_x = 12 ft K_x = 1
 b = 5.00 ft Stud spacing: 16 in K_x*L_x = 144 in
 I = L/2 = 15 ft

Live Load: P @ 50% = 3750 lb -----> 6 Kips factored
 P @ 100% = 7500 lb -----> 12 Kips factored

Reactions:

$$R_2 @ 50\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2 = 10.22 \text{ Kips}$$

$$R_1 @ 50\% = \sum M_{R1} = 0.889 \text{ kips} \quad R_3 = R_1 = 0.889 \text{ Kips}$$

$$R_2 @ 100\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2 = 20.44 \text{ Kips}$$

$$R_1 @ 100\% = \sum M_{R1} = 6.889 \text{ kips} \quad R_3 = R_1 = 6.889 \text{ Kips}$$

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 600S162-33
 $\phi P_n = 4.2866 \text{ K}$ From CFS program calculation (See attached sheets)

Note: $\phi P_n < P_u @ 50\% \text{ LL}$ **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting ϕP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

$$P_u = \frac{\phi P_n \cdot (2l^3)}{2(2l^2 + b(l+a))a} = 2.52 \text{ kips}$$

Percentages: $\frac{P_u}{P_u @ 100\%} = 33.55\%$ of LL For one stud
 = 20.97% of 1.6LL for one stud Two studs: 41.93% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination:

1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSE for wall studs.

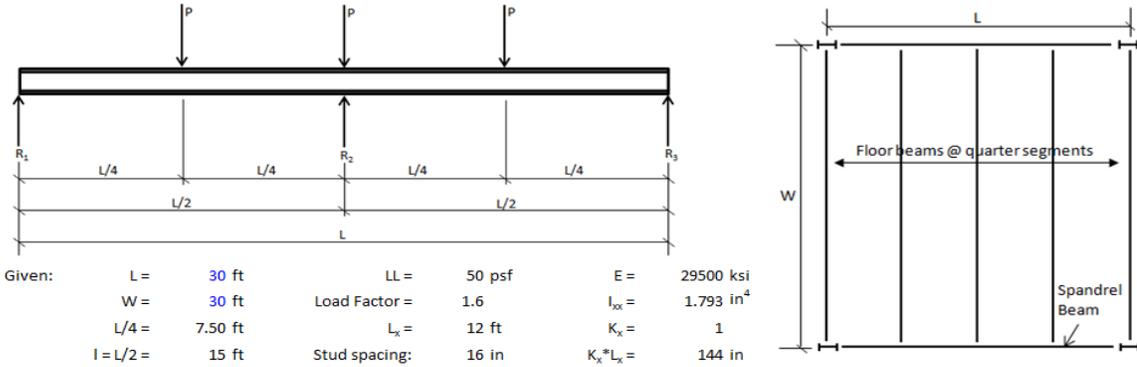
Loads on stud:	50% LL	100% LL	40.29% LL	
W =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	6.39 K	12.78 K	5.15 K	
P _u (Factored R ₂) =	3.19 K	6.39 K	2.57 K	**check with MathCAD calculations
M _u =	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	**check with MathCAD calculations
α _x =	0.873	0.746	0.898	**check with MathCAD calculations

Stud strength properties:			
φP _n =	4.2866 K	**From CFS program Calculations	P _{ex} = 25.17544 Kips **check with MathCAD calculations
φM _s =	855.57 lb-ft	**From MathCAD Calculation	C _m = 1 **From MathCAD Calculation
φP _{no} =	5.561 K	**From CFS program Calculations	Ends of beam are pinned-pinned

Interaction Check:		100% LL	50% LL	40.29% Controlling %
	$\frac{P_u}{\phi P_n}$	1.4904	0.7452	0.6005
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)
1 C5.2.2-3	$\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_n}$	1.848	1.103	0.958
		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN
2 C5.2.2-2	$\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_{no}}$	1.507	0.932	0.821
3 C5.2.2-1	$\frac{P_u}{\phi P_n} + \frac{C_m \cdot M_u}{\phi M_s \cdot \alpha_x}$	1.970	1.155	0.999
		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate

Stud Capacity: (30' X 30' Bay)

Due to 50 psf live load only



Given: L = 30 ft LL = 50 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 1.793 in⁴
 L/4 = 7.50 ft L_x = 12 ft K_x = 1
 I = L/2 = 15 ft Stud spacing: 16 in K_x*L_x = 144 in

Live Load: P @ 50% = 2812.5 lb -----> 4.5 Kips factored
 P @ 100% = 5625 lb -----> 9 Kips factored

Reactions:

$$R_1 @ 50\% = \frac{5P}{16} = 1.41 \text{ Kips}$$

$$R_3 = R_1 = 1.406 \text{ Kips}$$

$$R_2 @ 50\% = (P - R_1)2 + P = 10.69 \text{ kips}$$

$$R_1 @ 100\% = \frac{5P}{16} = 2.81 \text{ Kips}$$

$$R_3 = R_1 = 2.813 \text{ Kips}$$

$$R_2 @ 100\% = (P - R_1)2 + P = 21.38 \text{ kips}$$

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 600S162-33

φP_n = 4.2866 K From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

$$\phi P_u = \frac{\phi P_n}{2.375} = 1.80 \text{ kips}$$

Percentages: $\frac{P_u}{P_u @ 100\%} = 32.09\%$ of LL For one stud
 = 20.05% of 1.6LL for one stud Two studs: 40.11% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination: 1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCF SF for wall studs.

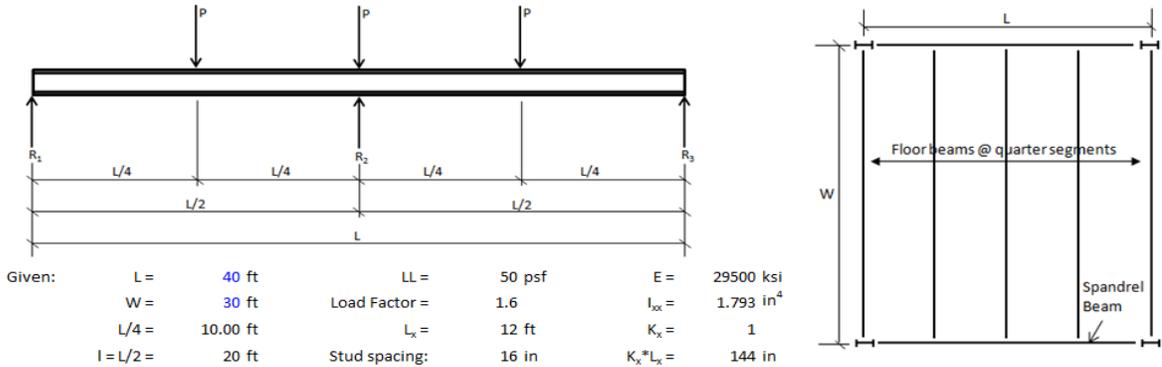
Loads on stud:	50% LL	100% LL	38.54% LL	
W	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂)	6.68 K	13.36 K	5.15 K	
P _u (Factored R ₂)	3.34 K	6.68 K	2.57 K	**check with MathCAD calculations
M _u	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	**check with MathCAD calculations
α _x	0.867	0.735	0.898	**check with MathCAD calculations
Stud strength properties:				
φP _n	4.2866 K	**From CFS program Calculations	P _{ex} = 25.17544 Kips	**check with MathCAD calculations
φM _s	855.57 lb-ft	**From MathCAD Calculation	C _m = 1	**From MathCAD Calculation
φP _{no}	5.561 K	**From CFS program Calculations	Ends of beam are pinned-pinned	

Interaction Check:

		100% LL	50% LL	38.54% LL	Controlling %
	$\frac{P_u}{\phi P_n}$	1.5583	0.7791	0.6005	
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1	$\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_n}$	1.916	1.137	0.958	
C5.2.2-3		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN	
2	$\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_{no}}$	1.559	0.958	0.821	
C5.2.2-2					
3	$\frac{P_u}{\phi P_n} + \frac{C_m * M_u}{\phi M_s * \alpha_x}$	2.045	1.192	0.999	
C5.2.2-1		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK	Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate	

Stud Capacity: (40' X 30' Bay)

Due to 50 psf live load only



Given: L = 40 ft LL = 50 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 1.793 in⁴
 L/4 = 10.00 ft L_x = 12 ft K_x = 1
 l = L/2 = 20 ft Stud spacing: 16 in K_x*L_x = 144 in

Live Load: P @ 50% = 3750 lb -----> 6 Kips factored
 P @ 100% = 7500 lb -----> 12 Kips factored

Reactions: R₁ @ 50% = $\frac{5P}{16} = 1.88$ Kips
 R₃ = R₁ = 1.875 Kips
 R₂ @ 50% = (P - R₁)2 + P = 14.25 kips

R₁ @ 100% = $\frac{5P}{16} = 3.75$ Kips
 R₃ = R₁ = 3.750 Kips
 R₂ @ 100% = (P - R₁)2 + P = 28.50 kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 600S162-33

φP_n = 4.2866 K

From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

Percentages: $\frac{\phi P_n}{P_u @ 100\%} = 24.07\%$ of LL For one stud
 = 1.80 kips
 = 15.04% of 1.6LL for one stud Two studs: 30.08% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination:

1.2D + 1.6W + LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSE for wall studs.

Loads on stud:	50% LL	100% LL	28.90% LL	
W =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	8.91 K	17.81 K	5.15 K	
P _u (Factored R ₂) =	4.45 K	8.91 K	2.57 K	**check with MathCAD calculations
M _u =	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	**check with MathCAD calculations
α _x =	0.823	0.646	0.898	**check with MathCAD calculations

Stud strength properties:

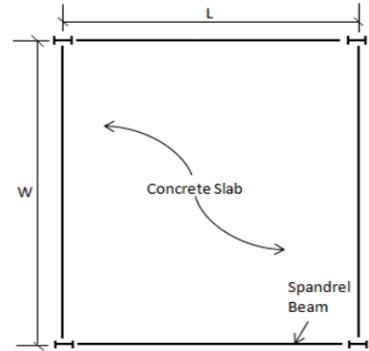
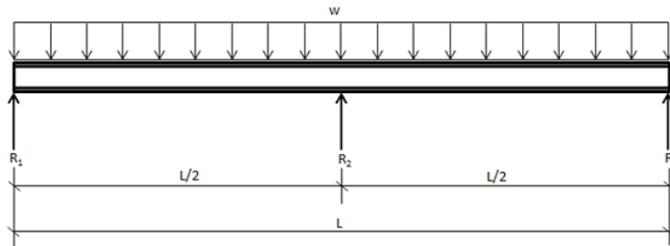
φP_n = 4.2866 K **From CFS program Calculations P_{ex} = 25.17544 Kips **check with MathCAD calculations
 φM_s = 855.57 lb-ft **From MathCAD Calculation C_m = 1 **From MathCAD Calculation
 φP_{no} = 5.561 K **From CFS program Calculations Ends of beam are pinned-pinned

Interaction Check:

		100% LL	50% LL	28.90% Controlling %
	$\frac{P_u}{\phi P_n}$ →→→	2.0777	1.0388	0.6005
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)
1	C5.2.2-3 $\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_n}$ →→→	2.435	1.397	0.958
		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN
2	C5.2.2-2 $\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_{no}}$ →→→	1.959	1.158	0.821
3	C5.2.2-1 $\frac{P_u}{\phi P_n} + \frac{C_m * M_u}{\phi M_s * \alpha_x}$ →→→	2.631	1.473	0.999
		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate

Stud Capacity: (40' X 30' Bay)

Due to 50 psf live load only



Given: L = 40 ft LL = 50 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 1.793 in⁴
 l = L/2 = 20 ft L_x = 12 ft K_x = 1
 Stud spacing: 16 in K_x*L_x = 144 in

Live Load: w @ 50% = 375 plf -----> w_u @ 50% = 0.6 klf *factored*
 w @ 100% = 750 plf -----> w_u @ 100% = 1.2 klf *factored*

Reactions: R₁ @ 50% = $\frac{3wl}{8}$ = 4.50 Kips R₃ = R₁ = 4.500 Kips R₂ @ 50% = $\frac{10wl}{8}$ = 15.00 Kips
 R₁ @ 100% = $\frac{3wl}{8}$ = 9.00 Kips R₃ = R₁ = 9.000 Kips R₂ @ 100% = $\frac{10wl}{8}$ = 30.00 Kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 600S162-33

φP_n = 4.2866 K

From CFS program calculation (See attached sheets)

Note: φP_n < R₂ @ 50% LL ****Stud is not adequate to take 50% of the LL**

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

w_u = $\frac{8*\phi P_n}{10*I}$ = 0.17 kips
 Percentages: $\frac{w_u}{w_u @ 100\%}$ = 22.86% of LL For one stud
 = 14.29% of 1.6LL for one stud Two studs: 28.58% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination: 1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSF for wall studs.

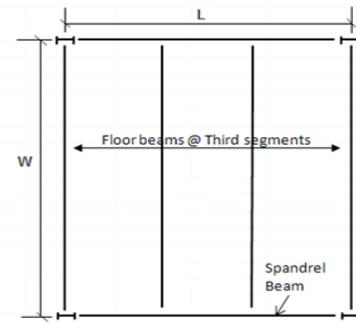
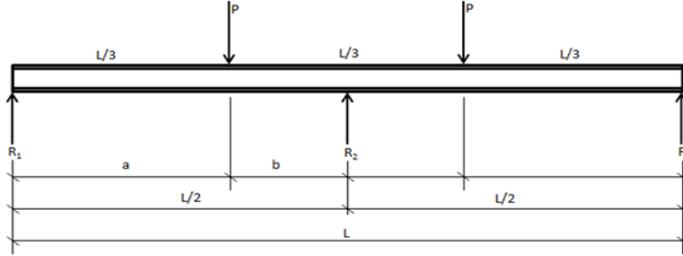
Loads on stud:	50% LL	100% LL	27.46% LL	
Wind =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	9.38 K	18.75 K	5.15 K	
P _u (Factored R ₂) =	4.69 K	9.38 K	2.57 K	**check with MathCAD calculations
M _u =	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	**check with MathCAD calculations
α _x =	0.814	0.628	0.898	**check with MathCAD calculations

Stud strength properties:
 φP_n = 4.2866 K ***From CFS program Calculations* P_{ex} = 25.17544 Kips ***check with MathCAD calculations*
 φM_n = 855.57 lb-ft ***From MathCAD Calculation* C_m = 1 ***From MathCAD Calculation*
 φP_{no} = 5.561 K ***From CFS program Calculations* Ends of beam are pinned-pinned

Interaction Check:		100% LL	50% LL	27.46% LL	Controlling %
1	$\frac{P_u}{\phi P_n}$	2.1870	1.0935	0.6005	
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
2	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_n}$	2.545	1.451	0.958	2 & 3 GOVERN
3	$\frac{P_u}{\phi P_n} + \frac{C_m * M_u}{\phi M_n * \alpha_x}$	2.757	1.533	0.999	2 & 3 GOVERN
		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK	Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate	

Stud Capacity: (25' X 30' Bay)

Due to 80 psf live load only



Given: L = 25 ft LL = 80 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 1.793 in⁴
 a = 8.33 ft L_x = 12 ft K_x = 1
 b = 4.17 ft Stud spacing: 16 in K_x*L_x = 144 in
 l = L/2 = 12.5 ft

Live Load: P @ 50% = 5000 lb -----> 8 Kips factored
 P @ 100% = 10000 lb -----> 16 Kips factored

Reactions:

$$R_2 @ 50\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2 \quad R_1 @ 50\% = \sum M_{R1} = 1.185 \text{ kips} \quad R_3 = R_1 = 1.185 \text{ Kips}$$

$$= 13.63 \text{ Kips}$$

$$R_2 @ 100\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2 \quad R_1 @ 100\% = \sum M_{R1} = 9.185 \text{ kips} \quad R_3 = R_1 = 9.185 \text{ Kips}$$

$$= 27.26 \text{ Kips}$$

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 600S162-33

φP_n = 4.2866 K From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

$$P_u = \frac{\phi P_n \cdot (2l^3)}{2(2l^2 + b(l+a))a}$$

$$= 2.52 \text{ kips}$$

Percentages: $\frac{P_u}{P_u @ 100\%} = 25.16\%$ of LL For one stud

= 15.73% of 1.6LL for one stud Two studs: 31.45% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination: 1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCF SF for wall studs.

Loads on stud:	50% LL	100% LL	30.22% LL	
W =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	8.52 K	17.04 K	5.15 K	
P _u (Factored R ₂) =	4.26 K	8.52 K	2.57 K	**check with MathCAD calculations
M _u =	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	**check with MathCAD calculations
α _x =	0.831	0.662	0.898	**check with MathCAD calculations

Stud strength properties:

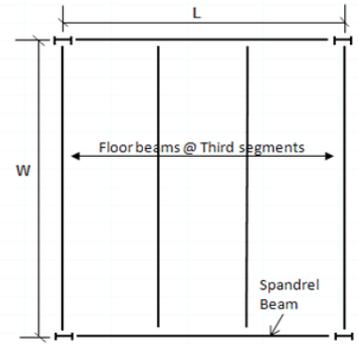
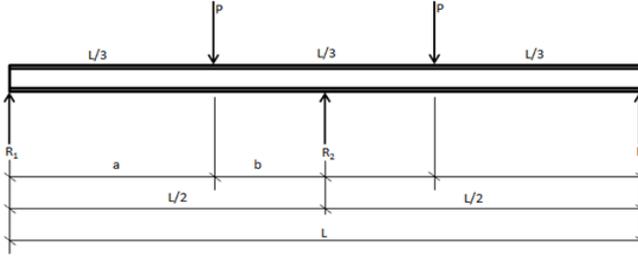
φP _n =	4.2866 K	**From CFS program Calculations	P _{ex} =	25.1754 Kips	**check with MathCAD calculations
φM _n =	855.57 lb-ft	**From MathCAD Calculation	C _m =	1	**From MathCAD Calculation
φP _{no} =	5.561 K	**From CFS program Calculations	Ends of beam are pinned-pinned		

Interaction Check:

		100% LL	50% LL	30.22% LL	Controlling %
	$\frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	1.9872	0.9936	0.6005	
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	2.345	1.351	0.958	
C5.2.2-3		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN	
2	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_{no}} \rightarrow \rightarrow \rightarrow$	1.890	1.124	0.821	
C5.2.2-2					
3	$\frac{P_u}{\phi P_n} + \frac{C_m \cdot M_u}{\phi M_n \cdot \alpha_x} \rightarrow \rightarrow \rightarrow$	2.528	1.424	0.999	
C5.2.2-1		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK	Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate	

Stud Capacity: (30' X 30' Bay)

Due to 80 psf live load only



Given: L = 30 ft LL = 80 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 1.793 in⁴
 a = 10.00 ft L_x = 12 ft K_x = 1
 b = 5.00 ft Stud spacing: 16 in K_x*L_x = 144 in
 l = L/2 = 15 ft

Live Load: P @ 50% = 6000 lb -----> 9.6 Kips factored
 P @ 100% = 12000 lb -----> 19.2 Kips factored

Reactions: $R_2 @ 50\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2$ $R_1 @ 50\% = \sum M_{R1} = 1.422$ kips $R_3 = R_1 = 1.422$ Kips
 = 16.36 Kips

$R_2 @ 100\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2$ $R_1 @ 100\% = \sum M_{R1} = 11.022$ kips $R_3 = R_1 = 11.022$ Kips
 = 32.71 Kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 600S162-33

$\phi P_n = 4.2866$ K From CFS program calculation (See attached sheets)

Note: $\phi P_n < P_u @ 50\% LL$ **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting ϕP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

$P_u = \frac{\phi P_n \cdot (2l^2)}{2(2l^2 + b(l+a))a}$ Percentages: $\frac{P_u}{P_u @ 100\%} = 20.97\%$ of LL For one stud
 = 2.52 kips = 13.10% of 1.6LL for one stud Two studs: 26.21% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination:

1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSF for wall studs.

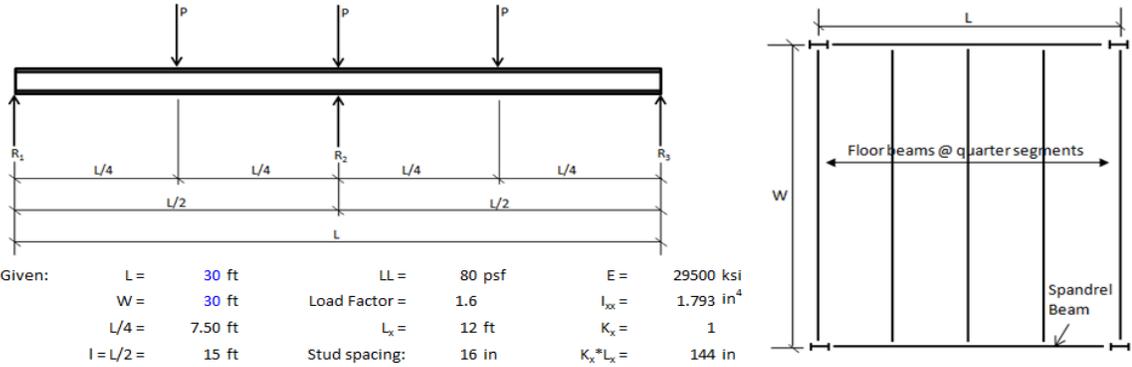
Loads on stud:	50% LL	100% LL	25.18% LL	
W =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	10.22 K	20.44 K	5.15 K	
P _u (Factored R ₂) =	5.11 K	10.22 K	2.57 K	**check with MathCAD calculations
M _u =	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	**check with MathCAD calculations
α_x =	0.797	0.594	0.898	**check with MathCAD calculations
Stud strength properties:				
ϕP_n =	4.2866 K		P _{Ex} = 25.17544 Kips	**check with MathCAD calculations
ϕM_n =	855.57 lb-ft		C _m = 1	**From MathCAD Calculation
ϕP_{no} =	5.561 K			Ends of beam are pinned-pinned

Interaction Check:

	100% LL	50% LL	25.18% LL	Controlling %
$\frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	2.3847	1.1923	0.6005	
> 0.15 (check 2 & 3)				
1 C5.2.2-3 $\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	2.742	1.550	0.958	
> 0.15 (check 2 & 3)				
2 C5.2.2-2 $\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_{no}} \rightarrow \rightarrow \rightarrow$	2.196	1.277	0.821	
> 0.15 (check 2 & 3)				
3 C5.2.2-1 $\frac{P_u}{\phi P_n} + \frac{C_m \cdot M_u}{\phi M_n \cdot \alpha_x} \rightarrow \rightarrow \rightarrow$	2.987	1.641	0.999	
$> 1.0 \rightarrow \rightarrow$ N.G.				
Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate		Calculate

Stud Capacity: (30' X 30' Bay)

Due to 80 psf live load only



Given: L = 30 ft LL = 80 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 1.793 in⁴
 L/4 = 7.50 ft L_x = 12 ft K_x = 1
 I = L/2 = 15 ft Stud spacing: 16 in K_x*L_x = 144 in

Live Load: P @ 50% = 4500 lb -----> 7.2 Kips factored
 P @ 100% = 9000 lb -----> 14.4 Kips factored

Reactions: R₁ @ 50% = $\frac{5P}{16} = 2.25$ Kips
 R₃ = R₁ = 2.250 Kips
 R₂ @ 50% = $(P - R_1)2 + P = 17.10$ kips
 R₁ @ 100% = $\frac{5P}{16} = 4.50$ Kips
 R₃ = R₁ = 4.500 Kips
 R₂ @ 100% = $(P - R_1)2 + P = 34.20$ kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 600S162-33

φP_n = 4.2866 K

From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

φP_n = $\frac{\phi P_n}{2.375} = 1.80$ kips
 Percentages: $\frac{P_u}{P_u @ 100\%} = 20.05\%$ of LL For one stud
 = 12.53% of 1.6LL for one stud Two studs: 25.07% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination: 1.6W + 0.5LL

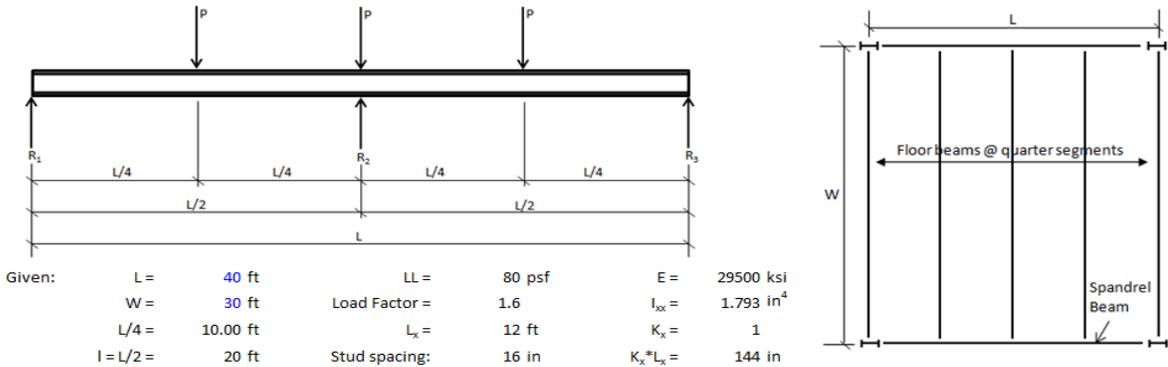
Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSS for wall studs.

Loads on stud:	50% LL	100% LL	24.09% LL	
W	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂)	10.69 K	21.38 K	5.15 K	
P _u (Factored R ₂)	5.34 K	10.69 K	2.57 K	**check with MathCAD calculations
M _u	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	**check with MathCAD calculations
α _x	0.788	0.575	0.898	**check with MathCAD calculations
Stud strength properties:				
φP _n	4.2866 K	**From CFS program Calculations	P _{ex} = 25.17544 Kips	**check with MathCAD calculations
φM _s	855.57 lb-ft	**From MathCAD Calculation	C _m = 1	**From MathCAD Calculation
φP _{no}	5.561 K	**From CFS program Calculations	Ends of beam are pinned-pinned	

Interaction Check:	100% LL	50% LL	24.09% LL	Controlling %
$\frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	2.4932	1.2466	0.6005	
	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1 C5.2.2-3 $\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	2.851	1.604	0.958	
	2 & 3 GOVERN			
2 C5.2.2-2 $\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_{no}} \rightarrow \rightarrow \rightarrow$	2.280	1.319	0.821	
	2 & 3 GOVERN			
3 C5.2.2-1 $\frac{P_u}{\phi P_n} + \frac{C_m M_u}{\phi M_s \alpha_x} \rightarrow \rightarrow \rightarrow$	3.115	1.701	0.999	< 1.0 OK Calculate
	> 1.0 --> N.G.	> 1.0 --> N.G.		
	Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate	

Stud Capacity: (40' X 30' Bay)

Due to 80 psf live load only



Given: L = 40 ft LL = 80 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 1.793 in⁴
 L/4 = 10.00 ft L_x = 12 ft K_x = 1
 l = L/2 = 20 ft Stud spacing: 16 in K_x*L_x = 144 in

Live Load: P @ 50% = 6000 lb -----> 9.6 Kips factored
 P @ 100% = 12000 lb -----> 19.2 Kips factored

Reactions: R₁ @ 50% = $\frac{5P}{16} = 3.00$ Kips
 R₃ = R₁ = 3.000 Kips
 R₂ @ 50% = (P - R₁)2 + P = 22.80 kips

R₁ @ 100% = $\frac{5P}{16} = 6.00$ Kips
 R₃ = R₁ = 6.000 Kips
 R₂ @ 100% = (P - R₁)2 + P = 45.60 kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 600S162-33

φP_n = 4.2866 K

From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

φP_u = $\frac{\phi P_n}{2.375} = 1.80$ kips
 Percentages: $\frac{P_u}{P_u @ 100\%} = 15.04\%$ of LL For one stud
 = 9.40% of 1.6LL for one stud Two studs: 18.80% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination:

1.2D + 1.6W + LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSF for wall studs.

Loads on stud:	50% LL	100% LL	18.07% LL	
W =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	14.25 K	28.50 K	5.15 K	
P _u (Factored R ₂) =	7.13 K	14.25 K	2.57 K	**check with MathCAD calculations
M _u =	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	**check with MathCAD calculations
α _x =	0.717	0.434	0.898	**check with MathCAD calculations

Stud strength properties:

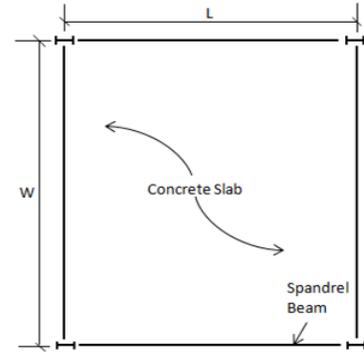
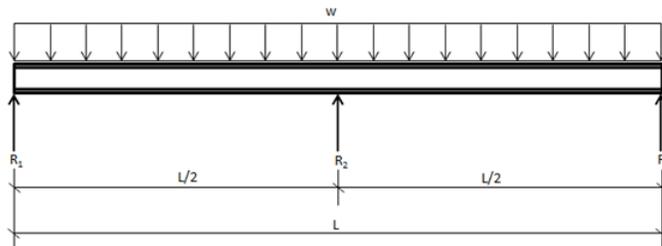
φP _n =	4.2866 K	**From CFS program Calculations	P _{Ex} =	25.17544 Kips	**check with MathCAD calculations
φM _s =	855.57 lb-ft	**From MathCAD Calculation	C _m =	1	**From MathCAD Calculation
φP _{no} =	5.561 K	**From CFS program Calculations	Ends of beam are pinned-pinned		

Interaction Check:

		100% LL	50% LL	18.07% Controlling %
	$\frac{P_u}{\phi P_n}$	3.3243	1.6622	0.6005
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)
1	$\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_n}$	3.682	2.020	0.958
C5.2.2-3		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN
2	$\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_{no}}$	2.920	1.639	0.821
C5.2.2-2				
3	$\frac{P_u}{\phi P_n} + \frac{C_m M_u}{\phi M_s \alpha_x}$	4.149	2.161	0.999
C5.2.2-1		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate

Stud Capacity: (40' X 30' Bay)

Due to **80 psf** live load only



Given: L = 40 ft LL = 80 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 1.793 in⁴
 l = L/2 = 20 ft L_x = 12 ft K_x = 1
 Stud spacing: 16 in K_x*L_x = 144 in

Live Load: w @ 50% = 600 plf -----> w_u @ 50% = 0.96 klf *factored*
 w @ 100% = 1200 plf -----> w_u @ 100% = 1.92 klf *factored*

Reactions: R₁ @ 50% = $\frac{3wl}{8} = 7.20$ Kips
 R₃ = R₁ = 7.200 Kips
 R₂ @ 50% = $\frac{10wl}{8} = 24.00$ Kips
 R₁ @ 100% = $\frac{3wl}{8} = 14.40$ Kips
 R₃ = R₁ = 14.400 Kips
 R₂ @ 100% = $\frac{10wl}{8} = 48.00$ Kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 600S162-33

φP_n = 4.2866 K

From CFS program calculation (See attached sheets)

Note: φP_n < R₂ @ 50% LL ****Stud is not adequate to take 50% of the LL**

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

w_u = $\frac{8*\phi P_n}{10*I} = 0.17$ kips
 Percentages: $\frac{w_u}{w_u @ 100\%} = 14.29\%$ of LL For one stud
 = 8.93% of 1.6LL for one stud Two studs: 17.86% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination: 1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCF SF for wall studs.

Loads on stud:	50% LL	100% LL	17.16% LL	
Wind =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	15.00 K	30.00 K	5.15 K	
P _u (Factored R ₂) =	7.50 K	15.00 K	2.57 K	**check with MathCAD calculations
M _u =	306.05 lb-ft	306.05 lb-ft	306.05 lb-ft	**check with MathCAD calculations
α _x =	0.702	0.404	0.898	**check with MathCAD calculations

Stud strength properties:

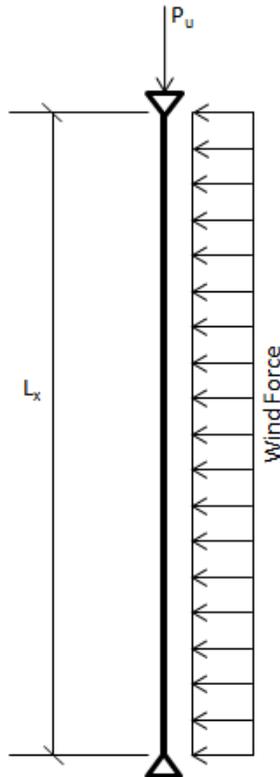
φP_n = 4.2866 K ****From CFS program Calculations**
 φM_n = 855.57 lb-ft ****From MathCAD Calculation**
 φP_{no} = 5.561 K ****From CFS program Calculations**
 P_{ex} = 25.17544 Kips ****check with MathCAD calculations**
 C_m = 1 ****From MathCAD Calculation**
 Ends of beam are pinned-pinned

Interaction Check:

		100% LL	50% LL	17.16% Controlling %
	$\frac{P_u}{\phi P_n}$	3.4993	1.7496	0.6005
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)
1	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_n}$	3.857	2.107	0.958
C5.2.2-3		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN
2	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_{no}}$	3.055	1.706	0.821
C5.2.2-2				
3	$\frac{P_u}{\phi P_n} + \frac{C_m * M_u}{\phi M_n * \alpha_x}$	4.384	2.259	0.999
C5.2.2-1		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate

For case 1, the 12' height wall stud at the second level is analyzed and the following are the calculations made in determining the size and capacities of the stud for both load conditions.

CFS Stud Sizing Due to Wind Load Only:



Note: From section A3.1 of the 2007 NASCFSF for wall studs, Component and cladding wind loads are to be used when sizing a wall stud due only to

Stud Spacing:	16 in o.c.	<u>Bridging Spacing</u>
L_x :	10 ft	L_t : 3.33 ft
Wind Load:	16.23 psf (C&C)	L_y : 3.33 ft
Load Factor :	1.6	

Moment in Stud:

$$M_x = \frac{wL^2}{8}$$

$$= \frac{432.80 \text{ lb*ft}}{8} \text{ ----> } 5.19 \text{ k*in}$$

Stud Selection:

From General Specification Catalogs: **362S162-33**
 At 15psf Lateral load & L/360 horizontal
 deflection limit the max span: **11.9167 ft**
> Lx Good!

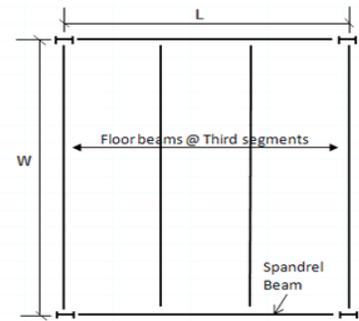
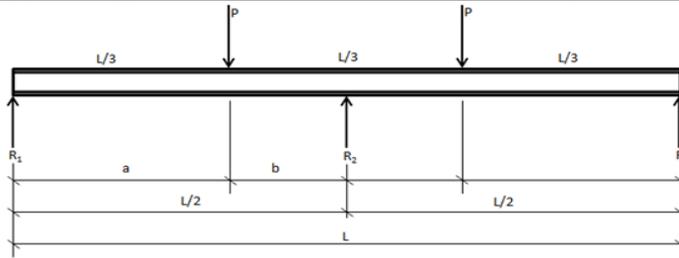
***See attached specification sheet for the studs properties*

Track Selection:

From General Specification Catalogs: **362T200-33**

Stud Capacity: (25' X 30' Bay)

Due to 50 psf live load only



Given: L = 25 ft LL = 50 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 0.5512 in⁴
 a = 8.33 ft L_x = 10 ft K_x = 1
 b = 4.17 ft Stud spacing: 16 in K_x*L_x = 120 in
 l = L/2 = 12.5 ft

Live Load: P @ 50% = 3125 lb -----> 5 Kips factored
 P @ 100% = 6250 lb -----> 10 Kips factored

Reactions:

$$R_2 @ 50\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2 \quad R_1 @ 50\% = \sum M_{R1} = 0.741 \text{ kips} \quad R_3 = R_1 = 0.741 \text{ Kips}$$

$$= 8.52 \text{ Kips}$$

$$R_2 @ 100\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2 \quad R_1 @ 100\% = \sum M_{R1} = 5.741 \text{ kips} \quad R_3 = R_1 = 5.741 \text{ Kips}$$

$$= 17.04 \text{ Kips}$$

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 362S162-33

φP_n = 3.6969 K From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL ****Stud is not adequate to take 50% of the LL**

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

$$P_u = \frac{\phi P_n \cdot (2l^3)}{2(2l^2 + b(l+a))a} \quad \text{Percentages: } \frac{P_u}{P_u @ 100\%} = 34.72\% \text{ of LL For one stud}$$

$$= 2.17 \text{ kips} \quad = 21.70\% \text{ of 1.6LL for one stud} \quad \text{Two studs: } 43.40\% \text{ of 1.6 LL}$$

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination:
 1.6W + 0.5LL

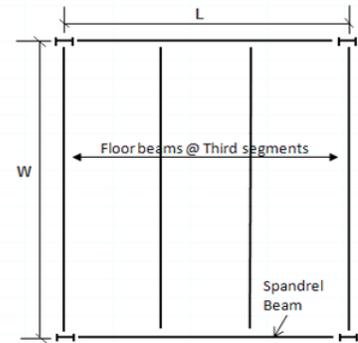
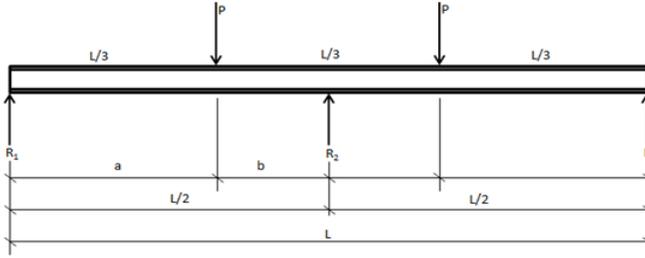
Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCF SF for wall studs.

Loads on stud:	50% LL	100% LL	26.73% LL	
W =	7.97 psf	7.96 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	5.32 K	10.65 K	2.85 K	
P _u (Factored R ₂) =	2.66 K	5.32 K	1.42 K	**check with MathCAD calculations
M _u =	212.53 lb-ft	212.27 lb-ft	212.53 lb-ft	**check with MathCAD calculations
α _x =	0.761	0.522	0.872	**check with MathCAD calculations
Stud strength properties:				
φP _n =	3.6969 K		P _{Ex} = 11.1447 Kips	**check with MathCAD calculations
φM _n =	396.77 lb-ft		C _m = 1	**From MathCAD Calculation
φP _{no} =	5.4061 K			Ends of beam are pinned-pinned

Interaction Check:	100% LL	50% LL	26.73% LL	Controlling %
$\frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	1.4401	0.7201	0.3850	
$\frac{1}{C5.2.2-3} \frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	1.975	1.256	0.921	
$\frac{2}{C5.2.2-2} \frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_{no}} \rightarrow \rightarrow \rightarrow$	1.520	1.028	0.799	
$\frac{3}{C5.2.2-1} \frac{P_u}{\phi P_n} + \frac{C_m \cdot M_u}{\phi M_n \cdot \alpha_x} \rightarrow \rightarrow \rightarrow$	2.464	1.424	0.999	
	> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK	Calculate

Stud Capacity: (30' X 30' Bay)

Due to 50 psf live load only



Given: L = 30 ft LL = 50 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 0.5512 in⁴
 a = 10.00 ft L_x = 10 ft K_x = 1
 b = 5.00 ft Stud spacing: 16 in K_x*L_x = 120 in
 l = L/2 = 15 ft

Live Load: P @ 50% = 3750 lb -----> 6 Kips factored
 P @ 100% = 7500 lb -----> 12 Kips factored

Reactions: $R_2 @ 50\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2$ $R_1 @ 50\% = \sum M_{R1} = 0.889$ kips $R_3 = R_1 = 0.889$ Kips
 = 10.22 Kips

$R_2 @ 100\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2$ $R_1 @ 100\% = \sum M_{R1} = 6.889$ kips $R_3 = R_1 = 6.889$ Kips
 = 20.44 Kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 362S162-33

φP_n = 3.6969 K

From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL ****Stud is not adequate to take 50% of the LL**

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

$P_u = \frac{\phi P_n \cdot (2l^3)}{2(2l^2 + b(l+a))a}$ Percentages: $\frac{P_u}{P_u @ 100\%} = 28.93\%$ of LL For one stud
 = 2.17 kips
 = 18.08% of 1.6LL for one stud Two studs: 36.17% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination:

1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSF for wall studs.

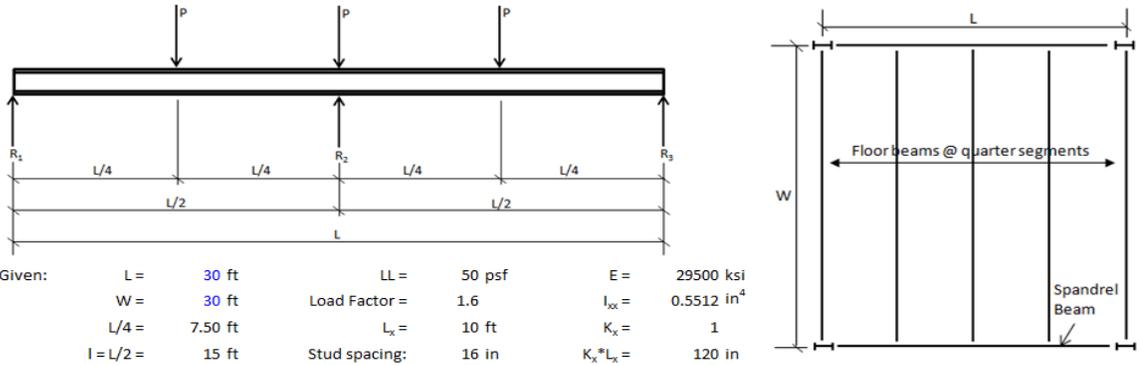
Loads on stud:	50% LL	100% LL	22.28% LL	
W =	7.97 psf	7.96 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	6.39 K	12.78 K	2.85 K	
P _u (Factored R ₂) =	3.19 K	6.39 K	1.42 K	**check with MathCAD calculations
M _u =	212.53 lb-ft	212.27 lb-ft	212.53 lb-ft	**check with MathCAD calculations
α _x =	0.713	0.427	0.872	**check with MathCAD calculations
Stud strength properties:				
φP _n =	3.6969 K		P _{ex} = 11.1447 Kips	**From CFS program Calculations **check with MathCAD calculations
φM _s =	396.77 lb-ft		C _m = 1	**From MathCAD Calculation **From MathCAD Calculation
φP _{no} =	5.4061 K			Ends of beam are pinned-pinned

Interaction Check:

		100% LL	50% LL	22.28% LL	Controlling %
	$\frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	1.7282	0.8641	0.3850	
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1	$\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	2.263	1.400	0.921	
C5.2.2-3		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN	
2	$\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_{no}} \rightarrow \rightarrow \rightarrow$	1.717	1.127	0.799	
C5.2.2-2					
3	$\frac{P_u}{\phi P_n} + \frac{C_m \cdot M_u}{\phi M_s \cdot \alpha_x} \rightarrow \rightarrow \rightarrow$	2.982	1.615	0.999	
C5.2.2-1		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK	Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate	

Stud Capacity: (30' X 30' Bay)

Due to 50 psf live load only



Live Load: P @ 50% = 2812.5 lb -----> 4.5 Kips factored
 P @ 100% = 5625 lb -----> 9 Kips factored

Reactions:

R ₁ @ 50% = $\frac{5P}{16}$	R ₃ = R ₁ = 1.406 Kips	R ₂ @ 50% = (P - R ₁)2 + P
= 1.41 Kips		= 10.69 kips
R ₁ @ 100% = $\frac{5P}{16}$	R ₃ = R ₁ = 2.813 Kips	R ₂ @ 100% = (P - R ₁)2 + P
= 2.81 Kips		= 21.38 kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 362S162-33

φP_n = 3.6969 K

From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

Percentages: $\frac{\phi P_n}{P_u @ 100\%} = \frac{2.375}{9} = 27.67\%$ of LL For one stud
 = 1.56 kips
 = 17.30% of 1.6LL for one stud Two studs: 34.59% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination:

1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFWF for wall studs.

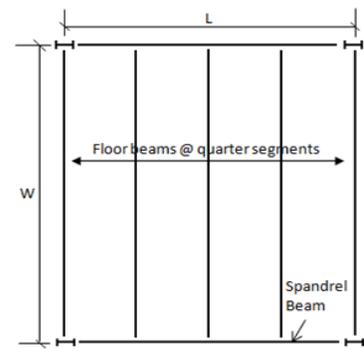
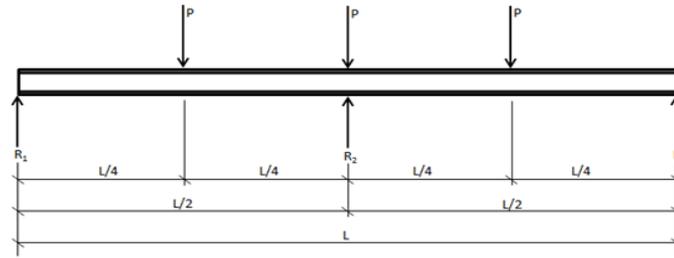
Loads on stud:	50% LL	100% LL	21.31% LL	
W =	7.97 psf	7.96 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	6.68 K	13.36 K	2.85 K	
P _u (Factored R ₂) =	3.34 K	6.68 K	1.42 K	**check with MathCAD calculations
M _u =	212.53 lb-ft	212.27 lb-ft	212.53 lb-ft	**check with MathCAD calculations
α _x =	0.700	0.401	0.872	**check with MathCAD calculations
Stud strength properties:				
φP _n =	3.6969 K	**From CFS program Calculations	P _{ex} = 11.144702 Kips	**check with MathCAD calculations
φM _a =	396.77 lb-ft	**From MathCAD Calculation	C _m = 1	**From MathCAD Calculation
φP _{no} =	5.4061 K	**From CFS program Calculations		Ends of beam are pinned-pinned

Interaction Check:

		100% LL	50% LL	21.31% LL	Controlling %
	$\frac{P_u}{\phi P_n}$	1.8068	0.9034	0.3850	
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1	$\frac{M_u}{\phi M_a} + \frac{P_u}{\phi P_n}$	2.342	1.439	0.921	
C5.2.2-3		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN	
2	$\frac{M_u}{\phi M_a} + \frac{P_u}{\phi P_{no}}$	1.771	1.153	0.799	
C5.2.2-2					
3	$\frac{P_u}{\phi P_n} + \frac{C_m * M_u}{\phi M_a * \alpha_x}$	3.142	1.668	0.999	
C5.2.2-1		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK	Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate	

Stud Capacity: (40' X 30' Bay)

Due to 50 psf live load only



Given: L = 40 ft LL = 50 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 0.5512 in⁴
 L/4 = 10.00 ft L_x = 10 ft K_x = 1
 I = L/2 = 20 ft Stud spacing: 16 in K_x*L_x = 120 in

Live Load: P @ 50% = 3750 lb -----> 6 Kips factored
 P @ 100% = 7500 lb -----> 12 Kips factored

Reactions: R₁ @ 50% = $\frac{5P}{16} = 1.88$ Kips
 R₃ = R₁ = 1.875 Kips
 R₂ @ 50% = (P - R₁)2 + P = 14.25 kips
 R₁ @ 100% = $\frac{5P}{16} = 3.75$ Kips
 R₃ = R₁ = 3.750 Kips
 R₂ @ 100% = (P - R₁)2 + P = 28.50 kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 362S162-33

φP_n = 3.6969 K

From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

φP_u = $\frac{\phi P_n}{2.375} = 1.56$ kips
 Percentages: $\frac{P_u}{P_u @ 100\%} = 20.75\%$ of LL For one stud
 = 12.97% of 1.6LL for one stud Two studs: 25.94% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination:

1.2D + 1.6W + LL

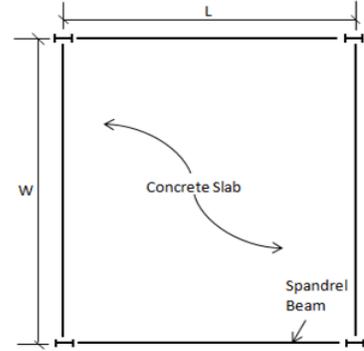
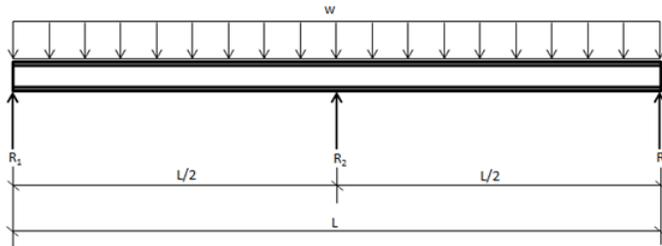
Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSF for wall studs.

Loads on stud:	50% LL	100% LL	15.98% LL	
W =	7.97 psf	7.96 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	8.91 K	17.81 K	2.85 K	
P _u (Factored R ₂) =	4.45 K	8.91 K	1.42 K	**check with MathCAD calculations
M _u =	212.53 lb-ft	212.27 lb-ft	212.53 lb-ft	**check with MathCAD calculations
α _x =	0.600	0.201	0.872	**check with MathCAD calculations
Stud strength properties:				
φP _n =	3.6969 K		P _{Ex} = 11.1447 Kips	**check with MathCAD calculations
φM _s =	396.77 lb-ft		C _m = 1	**From MathCAD Calculation
φP _{no} =	5.4061 K			Ends of beam are pinned-pinned

Interaction Check:		100% LL	50% LL	15.98% LL	Controlling %
	$\frac{P_u}{\phi P_n}$	2.4091	1.2046	0.3850	
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1 C5.2.2-3	$\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_n}$	2.944	1.740	0.921	
		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN	
2 C5.2.2-2	$\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_{no}}$	2.182	1.359	0.799	
3 C5.2.2-1	$\frac{P_u}{\phi P_n} + \frac{C_m * M_u}{\phi M_s * \alpha_x}$	5.073	2.097	0.999	
		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK	Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate	

Stud Capacity: (40' X 30' Bay)

Due to 50 psf live load only



Given: L = 40 ft LL = 50 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{ox} = 0.5512 in⁴
 l = L/2 = 20 ft L_x = 10 ft K_x = 1
 Stud spacing: 16 in K_x*L_x = 120 in

Live Load: w @ 50% = 375 plf ----> w_u @ 50% = 0.6 klf *factored*
 w @ 100% = 750 plf ----> w_u @ 100% = 1.2 klf *factored*

Reactions: R₁ @ 50% = $\frac{3wl}{8} = 4.50$ Kips R₃ = R₁ = 4.500 Kips R₂ @ 50% = $\frac{10wl}{8} = 15.00$ Kips
 R₁ @ 100% = $\frac{3wl}{8} = 9.00$ Kips R₃ = R₁ = 9.000 Kips R₂ @ 100% = $\frac{10wl}{8} = 30.00$ Kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 362S162-33

φP_n = 3.6969 K

From CFS program calculation (See attached sheets)

Note: φP_n < R₂ @ 50% LL ****Stud is not adequate to take 50% of the LL**

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

w_u = $\frac{8*\phi P_n}{10*I}$ = 0.15 kips Percentages: $\frac{w_u}{w_u @ 100\%} = 19.72\%$ of LL For one stud
 = 12.32% of 1.6LL for one stud Two studs: 24.65% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination: 1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSF for wall studs.

Loads on stud:	50% LL	100% LL	15.18% LL
Wind =	7.97 psf	7.96 psf	7.97 psf
LL (Unfactored R ₂) =	9.38 K	18.75 K	2.85 K
P _u (Factored R ₂) =	4.69 K	9.38 K	1.42 K
M _u =	212.53 lb-ft	212.27 lb-ft	212.53 lb-ft
α _x =	0.579	0.159	0.872

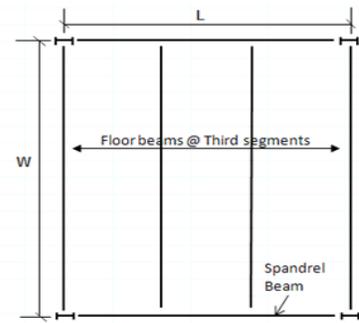
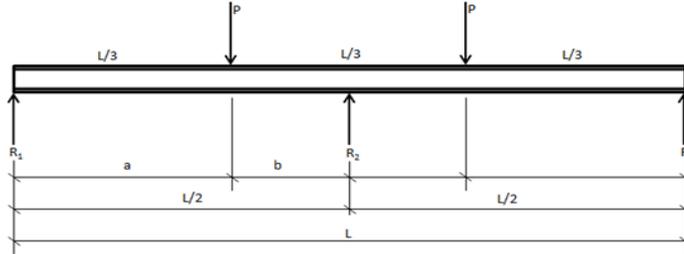
***MWFRS See wind load Calculations*
***check with MathCAD calculations*
***check with MathCAD calculations*

Stud strength properties:
 φP_n = 3.6969 K ***From CFS program Calculations* P_{ex} = 11.1447 Kips ***check with MathCAD calculations*
 φM_n = 396.77 lb-ft ***From MathCAD Calculation* C_m = 1 ***From MathCAD Calculation*
 φP_{no} = 5.4061 K ***From CFS program Calculations* *Ends of beam are pinned-pinned*

Interaction Check:	100% LL	50% LL	15.18% LL	Controlling %
$\frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	2.5359	1.2680	0.3850	
	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1 C5.2.2-3 $\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	3.071	1.804	0.921	
	2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN	
2 C5.2.2-2 $\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_{no}} \rightarrow \rightarrow \rightarrow$	2.269	1.403	0.799	
3 C5.2.2-1 $\frac{P_u}{\phi P_n} + \frac{C_m * M_u}{\phi M_n * \alpha_x} \rightarrow \rightarrow \rightarrow$	5.905	2.192	0.999	
	> 1.0 --> N.G. <i>Stud is NOT adequate</i>	> 1.0 --> N.G. <i>Stud is NOT adequate</i>	< 1.0 OK <i>Stud IS adequate</i>	Calculate

Stud Capacity: (25' X 30' Bay)

Due to 80 psf live load only



Given: L = 25 ft LL = 80 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 0.5512 in⁴
 a = 8.33 ft L_x = 10 ft K_x = 1
 b = 4.17 ft Stud spacing: 16 in K_x*L_x = 120 in
 I = L/2 = 12.5 ft

Live Load: P @ 50% = 5000 lb -----> 8 Kips factored
 P @ 100% = 10000 lb -----> 16 Kips factored

Reactions:

$$R_2 @ 50\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2 \quad R_1 @ 50\% = \sum M_{R1} = 1.185 \text{ kips} \quad R_3 = R_1 = 1.185 \text{ Kips}$$

$$= 13.63 \text{ Kips}$$

$$R_2 @ 100\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2 \quad R_1 @ 100\% = \sum M_{R1} = 9.185 \text{ kips} \quad R_3 = R_1 = 9.185 \text{ Kips}$$

$$= 27.26 \text{ Kips}$$

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 362S162-33

φP_n = 3.6969 K From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

$$P_u = \frac{\phi P_n \cdot (2l^3)}{2(2l^2 + b(l+a))a} \quad \text{Percentages: } \frac{P_u}{P_u @ 100\%} = 21.70\% \text{ of LL For one stud}$$

$$= 2.17 \text{ kips} \quad = 13.56\% \text{ of 1.6LL for one stud} \quad \text{Two studs: } 27.12\% \text{ of 1.6 LL}$$

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination: 1.6W + 0.5LL

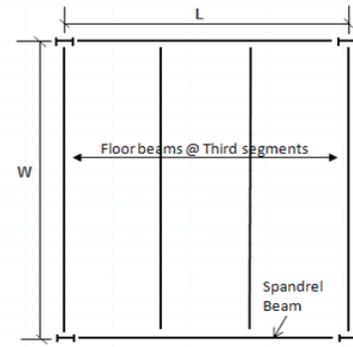
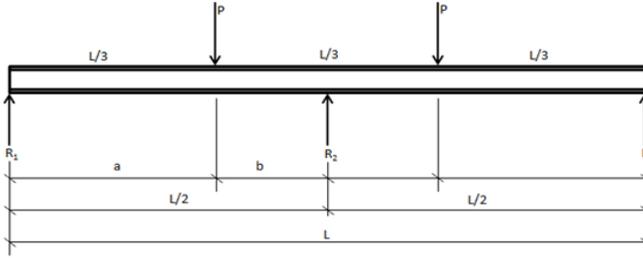
Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCF SF for wall studs.

Loads on stud:	50% LL	100% LL	16.71% LL	
W =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	8.52 K	17.04 K	2.85 K	
P _u (Factored R ₂) =	4.26 K	8.52 K	1.42 K	**check with MathCAD calculations
M _u =	212.53 lb-ft	212.53 lb-ft	212.53 lb-ft	**check with MathCAD calculations
α _x =	0.618	0.236	0.872	**check with MathCAD calculations
Stud strength properties:				
φP _n =	3.6969 K		P _{Ex} = 11.1447 Kips	**check with MathCAD calculations
φM _n =	396.77 lb-ft		C _m = 1	**From MathCAD Calculation
φP _{no} =	5.4061 K			Ends of beam are pinned-pinned

Interaction Check:		100% LL	50% LL	16.71% Controlling %
	$\frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	2.3042	1.1521	0.3850
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)
1 C5.2.2-3	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	2.840	1.688	0.921
		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN
2 C5.2.2-2	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_{no}} \rightarrow \rightarrow \rightarrow$	2.111	1.324	0.799
3 C5.2.2-1	$\frac{P_u}{\phi P_n} + \frac{C_m \cdot M_u}{\phi M_n \cdot \alpha_x} \rightarrow \rightarrow \rightarrow$	4.577	2.019	0.999
		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate

Stud Capacity: (30' X 30' Bay)

Due to **80 psf** live load only



Given: L = 30 ft LL = 80 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 0.5512 in⁴
 a = 10.00 ft L_x = 10 ft K_y = 1
 b = 5.00 ft Stud spacing: 16 in K_x*L_x = 120 in
 l = L/2 = 15 ft

Live Load: P @ 50% = 6000 lb -----> 9.6 Kips factored
 P @ 100% = 12000 lb -----> 19.2 Kips factored

Reactions: $R_2 @ 50\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2$ $R_1 @ 50\% = \sum R_{R1} = 1.422$ kips $R_3 = R_1 = 1.422$ Kips
 = 16.36 Kips

$R_2 @ 100\% = \frac{P_u \cdot a}{2 \cdot l^3} [2 \cdot l^2 + b \cdot (l+a)] \cdot 2$ $R_1 @ 100\% = \sum R_{R1} = 11.022$ kips $R_3 = R_1 = 11.022$ Kips
 = 32.71 Kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 362S162-33

$\phi P_n = 3.6969$ K From CFS program calculation (See attached sheets)

Note: $\phi P_n < P_u @ 50\% LL$ ****Stud is not adequate to take 50% of the LL**

Rearranging the equation for R₂ and substituting ϕP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

$P_u = \frac{\phi P_n \cdot (2l^3)}{2(2l^2 + b(l+a))a}$ Percentages: $\frac{P_u}{P_u @ 100\%} = 18.08\%$ of LL For one stud
 = 2.17 kips
 = 11.30% of 1.6LL for one stud Two studs: 22.60% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination:

1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCF for wall studs.

Loads on stud:	50% LL	100% LL	13.92% LL	
W	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂)	10.22 K	20.44 K	2.85 K	
P _u (Factored R ₂)	5.11 K	10.22 K	1.42 K	**check with MathCAD calculations
M _u	212.53 lb-ft	212.53 lb-ft	212.53 lb-ft	**check with MathCAD calculations
α_x	0.541	0.083	0.872	**check with MathCAD calculations

Stud strength properties:

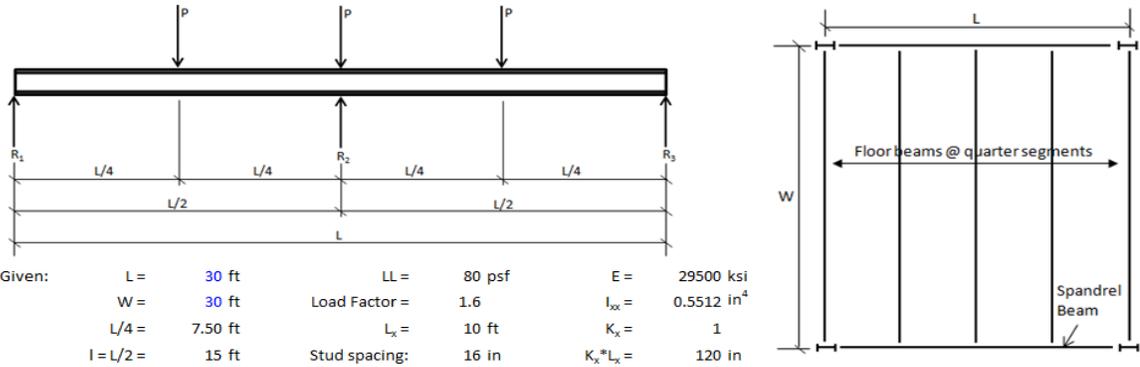
ϕP_n	3.6969 K	**From CFS program Calculations	P _{ex}	11.1447 Kips	**check with MathCAD calculations
ϕM_n	396.77 lb-ft	**From MathCAD Calculation	C _m	1	**From MathCAD Calculation
ϕP_{no}	5.4061 K	**From CFS program Calculations	Ends of beam are pinned-pinned		

Interaction Check:

		100% LL	50% LL	13.92% LL	Controlling %
	$\frac{P_u}{\phi P_n}$	2.7651	1.3825	0.3850	
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_n}$	3.301	1.918	0.921	
C5.2.2-3		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN	
2	$\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_{no}}$	2.427	1.481	0.799	
C5.2.2-2					
3	$\frac{P_u}{\phi P_n} + \frac{C_m \cdot M_u}{\phi M_n \cdot \alpha_x}$	9.237	2.372	0.999	
C5.2.2-1		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK	Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate	

Stud Capacity: (30' X 30' Bay)

Due to 80 psf live load only



Given: L = 30 ft LL = 80 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 0.5512 in⁴
 L/4 = 7.50 ft L_x = 10 ft K_x = 1
 I = L/2 = 15 ft Stud spacing: 16 in K_x*L_x = 120 in

Live Load: P @ 50% = 4500 lb -----> 7.2 Kips factored
 P @ 100% = 9000 lb -----> 14.4 Kips factored

Reactions: R₁ @ 50% = $\frac{5P}{16} = 2.25$ Kips
 R₃ = R₁ = 2.250 Kips
 R₂ @ 50% = (P - R₁)2 + P = 17.10 kips
 R₁ @ 100% = $\frac{5P}{16} = 4.50$ Kips
 R₃ = R₁ = 4.500 Kips
 R₂ @ 100% = (P - R₁)2 + P = 34.20 kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 3625162-33

φP_n = 3.6969 K

From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

φP_u = $\frac{\phi P_n}{2.375} = 1.56$ kips
 Percentages: $\frac{P_u}{P_u @ 100\%} = 17.30\%$ of LL For one stud
 = 10.81% of 1.6LL for one stud Two studs: 21.62% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination: 1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSS for wall studs.

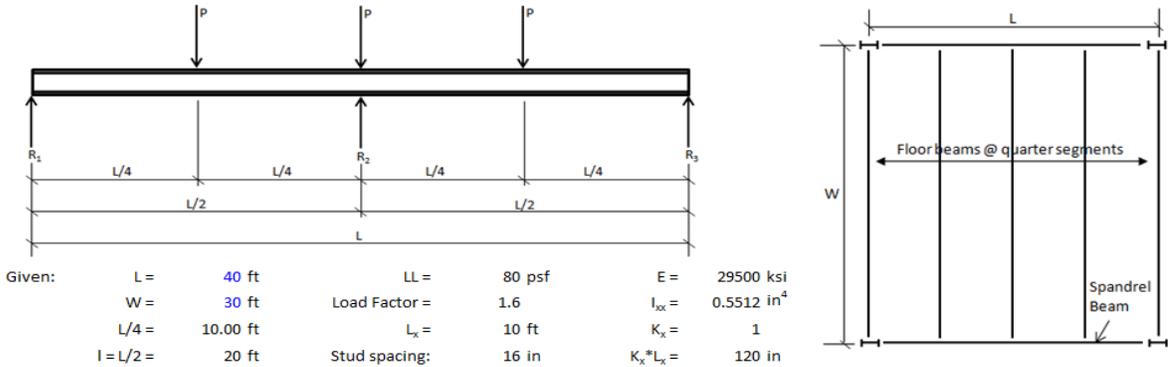
Loads on stud:	50% LL	100% LL	13.32% LL	
W =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	10.69 K	21.38 K	2.85 K	
P _u (Factored R ₂) =	5.34 K	10.69 K	1.42 K	**check with MathCAD calculations
M _u =	212.53 lb-ft	212.53 lb-ft	212.53 lb-ft	**check with MathCAD calculations
α _x =	0.521	0.041	0.872	**check with MathCAD calculations

Stud strength properties:
 φP_n = 3.6969 K **From CFS program Calculations
 φM_s = 396.77 lb-ft **From MathCAD Calculation
 φP_{no} = 5.4061 K **From CFS program Calculations
 P_{Ex} = 11.144702 Kips **check with MathCAD calculations
 C_m = 1 **From MathCAD Calculation
 Ends of beam are pinned-pinned

Interaction Check:	100% LL	50% LL	13.32% LL	Controlling %
$\frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	2.8909	1.4455	0.3850	
	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1 C5.2.2-3 $\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_n} \rightarrow \rightarrow \rightarrow$	3.427	1.981	0.921	
	2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN	
2 C5.2.2-2 $\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_{no}} \rightarrow \rightarrow \rightarrow$	2.513	1.524	0.799	
3 C5.2.2-1 $\frac{P_u}{\phi P_n} + \frac{C_m M_u}{\phi M_s \alpha_x} \rightarrow \rightarrow \rightarrow$	15.948 > 1.0 --> N.G. Stud is NOT adequate	2.475 > 1.0 --> N.G. Stud is NOT adequate	0.999 < 1.0 OK Stud IS adequate	Calculate

Stud Capacity: (40' X 30' Bay)

Due to 80 psf live load only



Given: L = 40 ft LL = 80 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 0.5512 in⁴
 L/4 = 10.00 ft L_x = 10 ft K_x = 1
 I = L/2 = 20 ft Stud spacing: 16 in K_x*L_x = 120 in

Live Load: P @ 50% = 6000 lb -----> 9.6 Kips factored
 P @ 100% = 12000 lb -----> 19.2 Kips factored

Reactions: R₁ @ 50% = $\frac{5P}{16} = 3.00$ Kips
 R₃ = R₁ = 3.000 Kips
 R₂ @ 50% = (P - R₁)2 + P = 22.80 kips
 R₁ @ 100% = $\frac{5P}{16} = 6.00$ Kips
 R₃ = R₁ = 6.000 Kips
 R₂ @ 100% = (P - R₁)2 + P = 45.60 kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 362S162-33

φP_n = 3.6969 K From CFS program calculation (See attached sheets)

Note: φP_n < P_u @ 50% LL **Stud is not adequate to take 50% of the LL

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

φP_u = $\frac{\phi P_n}{2.375} = 1.56$ kips Percentages: $\frac{P_u}{P_u @ 100\%} = 12.97\%$ of LL For one stud
 = 8.11% of 1.6LL for one stud Two studs: 16.21% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination:

1.2D + 1.6W + LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCSFS for wall studs.

Loads on stud:	50% LL	100% LL	9.99% LL	
W =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	14.25 K	28.50 K	2.85 K	
P _u (Factored R ₂) =	7.13 K	14.25 K	1.42 K	**check with MathCAD calculations
M _u =	212.53 lb-ft	212.53 lb-ft	212.53 lb-ft	**check with MathCAD calculations
α _x =	0.361	-0.279	0.872	**check with MathCAD calculations

Stud strength properties:

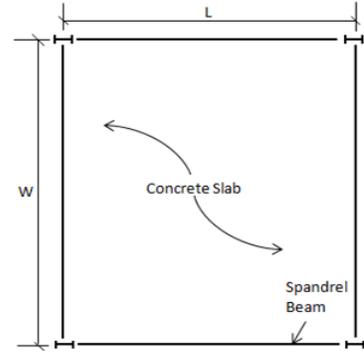
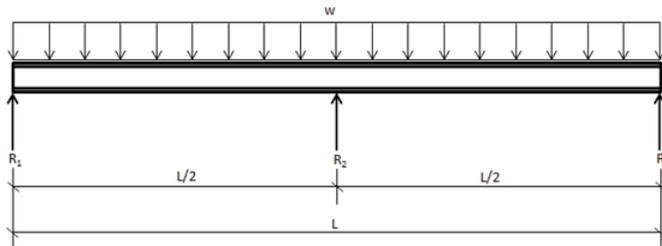
φP_n = 3.6969 K **From CFS program Calculations P_{Ex} = 11.1447 Kips **check with MathCAD calculations
 φM_s = 396.77 lb-ft **From MathCAD Calculation C_m = 1 **From MathCAD Calculation
 φP_{no} = 5.4061 K **From CFS program Calculations Ends of beam are pinned-pinned

Interaction Check:

		100% LL	50% LL	9.99% Controlling %
	$\frac{P_u}{\phi P_n}$ →→→	3.8546	1.9273	0.3850
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)
1	C5.2.2-3 $\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_n}$ →→→	4.390	2.463	0.921
		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN
2	C5.2.2-2 $\frac{M_u}{\phi M_s} + \frac{P_u}{\phi P_{no}}$ →→→	3.172	1.854	0.799
3	C5.2.2-1 $\frac{P_u}{\phi P_n} + \frac{C_m * M_u}{\phi M_s * \alpha_x}$ →→→	1.932	3.412	0.999
		> 1.0 --> N.G. Stud is NOT adequate	> 1.0 --> N.G. Stud is NOT adequate	< 1.0 OK Calculate

Stud Capacity: (40' X 30' Bay)

Due to **80 psf** live load only



Given: L = 40 ft LL = 80 psf E = 29500 ksi
 W = 30 ft Load Factor = 1.6 I_{xx} = 0.5512 in⁴
 l = L/2 = 20 ft L_x = 10 ft K_x = 1
 Stud spacing: 16 in K_x*L_x = 120 in

Live Load: w @ 50% = 600 plf ----> w_u @ 50% = 0.96 klf *factored*
 w @ 100% = 1200 plf ----> w_u @ 100% = 1.92 klf *factored*

Reactions: R₁ @ 50% = $\frac{3wl}{8}$ = 7.20 Kips R₃ = R₁ = 7.200 Kips R₂ @ 50% = $\frac{10wl}{8}$ = 24.00 Kips
 R₁ @ 100% = $\frac{3wl}{8}$ = 14.40 Kips R₃ = R₁ = 14,400 Kips R₂ @ 100% = $\frac{10wl}{8}$ = 48.00 Kips

Note: R₂ is considered to be the worst case scenario and is assumed to be the axial load induced on the metal stud.

Stud Capacity for Axial Load Only:

Stud : 362S162-33

φP_n = 3.6969 K

From CFS program calculation (See attached sheets)

Note: φP_n < R₂ @ 50% LL ****Stud is not adequate to take 50% of the LL**

Rearranging the equation for R₂ and substituting φP_n with R₂ the axial load is obtained to get a percentage of load the stud can take

w_u = $\frac{8*\phi P_n}{10*I}$ = 0.15 kips Percentages: $\frac{w_u}{w_u @ 100\%}$ = 12.32% of LL For one stud
 = 7.70% of 1.6LL for one stud Two studs: 15.40% of 1.6 LL

Checking Axial Compressive load + bending on stud: see attached calculations form MathCAD

Load combination: 1.6W + 0.5LL

Note: The wind load used is MWFRS since it is a combination of axial load and bending. See Section A3.1 of the 2007 NASCFSF for wall studs.

Loads on stud:	50% LL	100% LL	9.49% LL	
Wind =	7.97 psf	7.97 psf	7.97 psf	**MWFRS See wind load Calculations
LL (Unfactored R ₂) =	15.00 K	30.00 K	2.85 K	
P _u (Factored R ₂) =	7.50 K	15.00 K	1.42 K	**check with MathCAD calculations
M _u =	212.53 lb-ft	212.53 lb-ft	212.53 lb-ft	**check with MathCAD calculations
α _x =	0.327	-0.346	0.872	**check with MathCAD calculations

Stud strength properties:

φP_n = 3.6969 K ***From CFS program Calculations* P_{ex} = 11.1447 Kips ***check with MathCAD calculations*
 φM_n = 396.77 lb-ft ***From MathCAD Calculation* C_m = 1 ***From MathCAD Calculation*
 φP_{no} = 5.4061 K ***From CFS program Calculations* *Ends of beam are pinned-pinned*

Interaction Check:

		100% LL	50% LL	9.49% LL	Controlling %
	$\frac{P_u}{\phi P_n}$	4.0575	2.0287	0.3850	
		> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	> 0.15 (check 2 & 3)	
1	C5.2.2-3 $\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_n}$	4.593	2.564	0.921	
		2 & 3 GOVERN	2 & 3 GOVERN	2 & 3 GOVERN	
2	C5.2.2-2 $\frac{M_u}{\phi M_n} + \frac{P_u}{\phi P_{no}}$	3.310	1.923	0.799	
3	C5.2.2-1 $\frac{P_u}{\phi P_n} + \frac{C_m * M_u}{\phi M_n * \alpha_x}$	2.509	3.667	0.999	
		> 1.0 --> N.G.	> 1.0 --> N.G.	< 1.0 OK	Calculate
		Stud is NOT adequate	Stud is NOT adequate	Stud IS adequate	

APPENDIX D – COPYRIGHT RELEASE AUTHORISATIONS

Subject: RE: Copyright Release

▼ Sent By "Nabil Rahman" <nabil@steelnetwork.com> On: December 16, 2009 1:55 PM
To: bmonroy@k-state.edu
Cc: sstephen@ksu.edu

Hi Barbara,

You have my permission to use the details in your research report. There is no special form to fill out. Only request is to cite TSN technical note and web site as the source of the details.

I appreciate if you send me a copy of your report once complete.

Regards,

Nabil

Nabil A. Rahman, Ph.D., P.E.
The Steel Network, Inc.
Office: (919) 845-1025 ext.116
nabil@steelnetwork.com
www.steelnetwork.com

-----Original Message-----

From: Barbara Monroy [mailto:bmonroy@k-state.edu]
Sent: Tuesday, December 15, 2009 11:05 PM
To: Nabil Rahman
Subject: Copyright Release

Mr. Rahman,

My name is, Barbara Monroy, I am a graduate student at Kansas State University in the Architectural Engineering Department. I am writing a master report on The Deflection Gap for a Cold-Formed Steel Curtain Wall System. I have attained a copy of "Design of single Deep Leg Track to Accomodate Vertical Deflection" that you wrote in 2003; which I reference in my report. I would also like to use some of the details from the report in my report and would like your permission to do so. There are also other images I attained from The Steel Network that I would like to use in my report.

I was referred to you by my graduate professor Dr. Sutton Stephens. If it is alright with you I would like to submit a formal request for the use of the images. Please let me know if there is a copyright release form I would need to fill out or any other type of documentation I might need. I will be more than glad to send you a copy of the report once it is complete.

I have attached a document that shows the images I would like to use.

Thank you for your time,

Barbara L. Monroy
Structural Engineering Student
Architectural Engineering Program
Kansas State University
bmonroy@ksu.edu

Subject: RE: Copyright Release

▼ Sent By "Don Allen" <steeldon@earthlink.net> On: December 17, 2009 6:42 AM
To: bmonroy@k-state.edu

Dear Ms. Monroy,

Thank you for your request. As Technical Director for the Cold-Formed Steel Engineers Institute (CFSEI), I authorize you to use the image in your report under the following conditions:

- The Cold-Formed Steel Engineers Institute is given appropriate credit for the image
- Any journals or other publications where the report is summarized or published, and the image is used, gives the Cold-Formed Steel Engineers Institute appropriate credit for the image.

Thank you for considering Cold-Formed Steel as a topic for your graduate report.

Sincerely,

Don Allen, P. E. (dallen@cfsei.org)

Technical Director, Cold-Formed Steel Engineers Institute

DC Office: 202-785-2022 x14 / 1201 15th Street NW Suite 320, Washington, DC 20005-2842

GA Office: 706-597-8076 / 1480 Cobbham Road, Thomson, GA 30824-4141

Cell: 202-497-9375 www.steel framing.org / www.cfsei.org

-----Original Message-----

From: Barbara Monroy [<mailto:bmonroy@k-state.edu>]
Sent: Thursday, December 17, 2009 12:39 AM
To: steeldon@earthlink.net
Cc: Sutton Stephens
Subject: Copyright Release

Mr. Allen,

My name is, Barbara Monroy, I am a graduate student at Kansas State University in the Architectural Engineering Department. I am a writing a master report on The Deflection Gap for a Cold-Formed Steel Curtain Wall System. I have attained a copy of Technical Note on Cold-Formed Steel Construction "Common Design issues for Deflection Track" put out bu the CFSEI in September 2009; which I reference in my report. I would also like to use an image from the tech note in my report and would like your permission to do so.

I was referred to you by my graduate professor Dr. Sutton Stephens. If it is alright with you I would like to submit a formal request for the use of the images. Please let me know if there is a copyright release form I would need to fill out or any other type of documentation I might need. I will be more than glad to send you a copy of the report once it is complete.

I have attached a document that shows the image I would like to use.

Thank you for your time,

Barbara L. Monroy

Structural Engineering Student

Architectural Engineering Program

Kansas State University

bmonroy@ksu.edu

Subject: RE: Copyright Release

▼ Sent By "Augie Sisco" <augies@cmservnet.com> On: December 17, 2009 10:02 AM
To: bmonroy@k-state.edu

Sorry, you may use these images as long as you indicate somewhere a credit line stating that they were used with permission of the Steel Stud Mfrs. Assn. (SSMA), Glen Ellyn, IL

Good luck in your studies.

Augie Sisco

August L. Sisco
Executive Director
Steel Stud Manufacturers Association
800 Roosevelt Rd., Bldg. C-312
Glen Ellyn, IL 60137
(630) 942-6592
Fax (630) 790-3095
Direct Line: (630) 942-6525
www.ssma.com

-----Original Message-----

From: Barbara Monroy [mailto:bmonroy@k-state.edu]
Sent: Thursday, December 17, 2009 9:06 AM
To: Augie Sisco
Subject: Fwd: Copyright Release

Mr. Sisco,

I have not received a response from you regarding the email below. If you are alright with me using the images, all I would need is a reply by email stating that you authorize the use of the images in my report.

Once again, Thank you

Barbara L. Monroy
Structural Engineering Student
Architectural Engineering Program
Kansas State University
bmonroy@ksu.edu

----- Forwarded Message -----

From: "Barbara Monroy" <bmonroy@k-state.edu>
To: augies@cmservnet.com
Sent: Tuesday, December 15, 2009 9:52:30 PM GMT -06:00 US/Canada Central
Subject: Copyright Release

Mr Sisco,

My name is, Barbara Monroy, I am a graduate student at Kansas State University in the Architectural Engineering Department. I am writing a master report on The Deflection Gap for a Cold-Formed Steel Curtain Wall System. I have attained a copy of the "SSMA Cold-Formed Steel Details" which I reference in my report. I would also like to use some of the details from the "Cold-Formed Steel Detail" in the report and would like your permission to do so.

I was referred to you by Don Allen and by my graduate professor Dr. Sutton Stephens. If it is alright with you I would like to submit a formal request for the use of the images. Please let me know if there is a copyright release form I would need to fill out or any other type of documentation I might need. I will be more than glad to send you a copy of the report once it is complete.

I have also attached a document that shows the images I would like to use.

Thank you for your time,

Barbara L. Monroy
Structural Engineering Student
Architectural Engineering Program
Kansas State University
bmonroy@ksu.edu