A comparison of double clip angle shear connections to shear tab connections in industrial applications

by

BRANDI NICHOLE MARTIN

B.S., Kansas State University, 2008

A REPORT

submitted in partial fulfillment of the requirements for the degree

MASTER OF SCIENCE

Department of Architectural Engineering
College of Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

2017

Approved by:

Major Professor
Kimberly Waggle Kramer, P.E., S.E.

Abstract

In structural steel connection design, simple shear connections are one of the most common connection types utilized. The industry, especially from the side of the engineer, tends to lean toward using Double Clip Angle Connections as the default standard for simple shear connections. A double clip angle connection is a connection consisting of two angles transferring the shear forces from one member to the next either through bolts or welds. The design of Double Clip Angle Connections is efficient and the connections themselves are easy to fabricate. However, benefits to utilizing other types of shear connections exist. Many of these benefits are seen in the fabrication shop or during erection and construction. This is especially true of single shear plate or shear tab connections when applied to open structure design.

Shear tab connections consist of a single plate that transfers the shear forces from one member to the next with bolts or with welds. The design of shear tab connections can be a more involved process than the design of double clip angles. Sometimes the shear plate or shear tab has to be longer than is typical. This is called an extended shear plate connection. These extended shear plates can bring other variables into the design that typically don’t occur with Double Clip Angle Connections such as bending of the plate or the need for multiple bolt columns. However, with proper planning and detailing, the benefits and savings experienced in the fabrication or construction phase may outweigh what can be seen as a more laborious design task.

The purpose of this report is to identify the possible benefits achieved in using each of these connections, highlight the differences in the design approach for each, and use a study model to compare the outcome of using one connection over another in the design of a typical open structure. Double clip angles are typically the most efficient approach when speed of design

and simplicity of fabrication are the desired outcomes. However, shear plate or shear tab connections have the potential to provide safer erection alternatives and materials savings if used in appropriate ways and with the right applications.

Table of Contents

List of Figures

List of Tables

Dedication

I dedicate this report to my parents who have continuously supported my lofty goals, my husband who has been my biggest cheerleader, my children and “evil” stepchildren who bring perspective and laughter to my every day, and to God who has walked beside me holding my hand even when I thought He had let it go.

Chapter 1 - Introduction

Structural steel shear connections are the most commonly used connection in the steel construction industry (S. Ashton, personal communication, July 2015; S. Schroeder, personal communication, March 2016; R. Krueger, personal communication, July 2015). These connections are used to transfer shear forces from one steel member into another. This report focuses on two types: Double Clip Angle Connections (DCAC) and Single Shear Plate Connections (SSPC) or Shear Tab Connections. The application under consideration is shear connections used in open structures, such as pipe racks or equipment support structures. Shear connections, when used in this application, have certain advantages or disadvantages in their design and fabrication. Advantages and disadvantages to each of these types of connections during construction exist in addition to structural behavior and design.

Chapter 2 introduces the concept of shear connections. Shear connections are defined and the different types of shear connections are discussed. Double Clip Angle Connections and Single Shear Plate Connections typical industry applications and fabrication processes are addressed. A brief history of how each type developed as steel fabrication and construction became more prominent. The standard framed beam definitions in AISC for each connection type is presented in Chapter 3. It develops the study models and study connections used as a basis of comparison of the clip angles to shear tab connections. Chapter 3 outlines the results of the study model when using all clip angles or all shear tabs.

The differences in the design approach for each connection is briefly introduced in Chapter 4. The results of the study model and how those results can be applied to the discussion of advantages and disadvantages of each connection is presented in Chapter 4. The results are studied with four different perspectives in mind: the design perspective, fabrication perspective, cost perspective; and safety and efficiency in steel erection perspective.

Chapter 2 - Structural Steel Shear Connections

In structural steel design, a variety of connection types are utilized to transfer forces through the structural components and into the foundation. Of these connections, one of the most common are shear connections. Industry-wide, shear connections are thought to be simple, but shear connections can range in complexity depending upon the connected components and the load being transferred through the connection. The science of connection design involves equilibrium, limit states, load paths, and lower bound theorem of limit analysis. To further understand shear connections, what shear connections are, how and when they are typically used in the construction industry, which types are most commonly used in design and the advantages or disadvantages of the commonly used types is presented.

Definition

In the design of a steel superstructure, the vertical and horizontal loads imposed on the structural components must be transferred to the foundation system and on into the earth. The lower bound theorem of limit analysis states: any solution for a connection that satisfies equilibrium and the limit state yields a safe connection. The difficulty is finding the internal force distribution that maximizes the external load at which a connection fails. Each component of the structure, including the roof deck, beams, girders, braces, columns and base plates are first designed to independently handle the loads imposed upon them throughout the life of the structure. These loads can include combinations of dead load, live load, ice/snow load, construction loads, wind load, and seismic load. Once those components are designed to properly transfer the load to the other supporting pieces of the structure, each component must be properly connected to allow for the transfer of those loads to other members. These connections are the components that bring the structural design together and are critical to the integrity of the overall design.

Shear connections are designed to transfer only shear forces and not to properly transfer bending moment. They are typically used in locations on a structure where gravity loads are the main force being transferred through the connecting member. Shear connections do not provide rigidity and therefore cannot be used independent of a lateral force resisting system, such as braced or moment resisting frames, somewhere within the structure.

A variety of shear connection types are utilized in the steel construction industry. For the purposes of this paper, only shear connections of wide flange shapes are discussed. These connection types include double and single clip angle connections, shear end-plate connections, seat connections, single shear plate connections, tee connections, and shear splice connections. The most commonly used of these connections are double clip angle and single shear plate connections and therefore, the focus of this report.

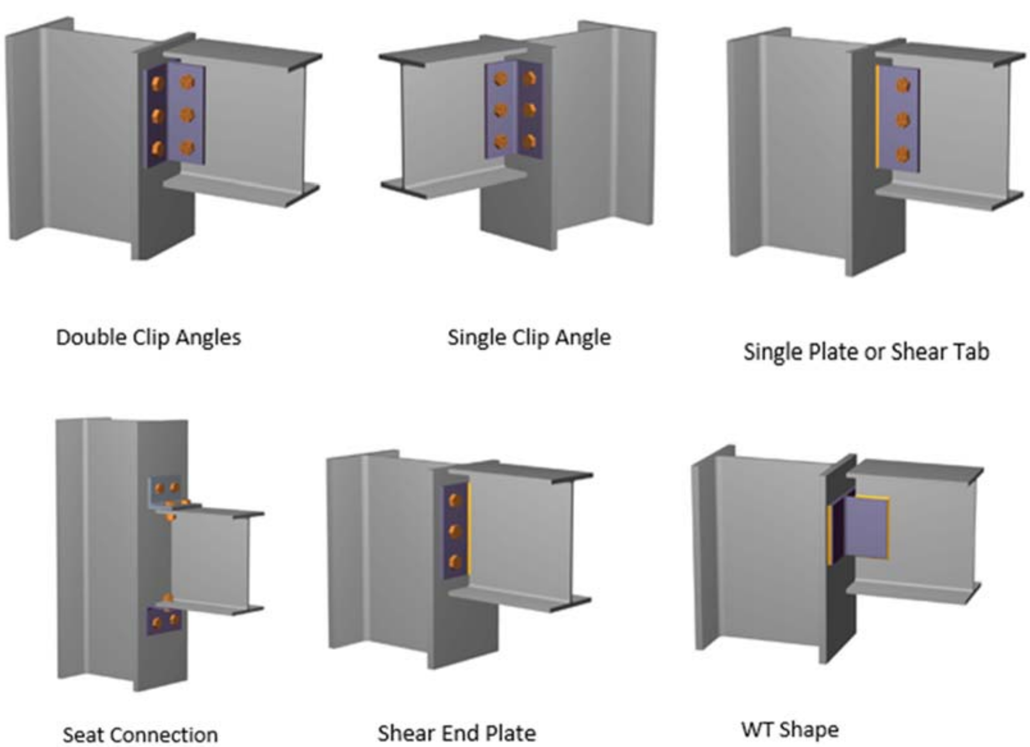


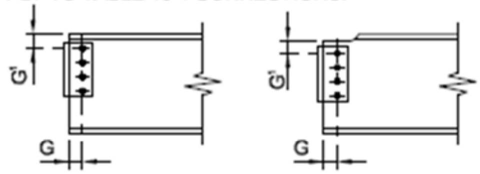
Figure 2.1 - Types of Shear Connections

Double Clip Angle Connections

A double clip angle connection is used in locations where a beam frames into another beam or into a column flange or web. The connecting beam web is placed between the legs of the two clip angles. The other legs of the angles are then connected to the supporting beam or column member. These connections are only meant to transfer shear load, but can transfer some axial load depending on the connection design. Double Clip Angle Connections are used so widely in industry, many engineering firms refer to these types of connections as a "standard shear connection" and typically include a standard set of details for a steel fabricator to default to unless a special connection is required. An example of how these standard connections are handled within a set of engineered drawings is shown in Figure 2.2. The basic design is covered in notes indicating what is considered "standard" and provides the fabricator with direction on the assumptions to make if no other design considerations are noted. AISC tables are typically referenced and a minimum number of bolts is indicated based simply on the depth of the connected member. For example, a W12 beam would get 3 rows of bolts unless the design drawings indicate something different. These connections are designed by the engineer unless the fabricator is specifically contracted to design these and is provided the loading conditions.

NOTES:

1. UNLESS NOTED OTHERWISE ALL BEAMS ON COLUMN ROW SHALL BE TABLE 10-2 CASE 1 CONNECTIONS WITH SHOP WELDS.
2. THE NUMBER INDICATES THE MINIMUM NUMBER OF ROWS OF HIGH STRENGTH BOLTS FOR THAT END CONNECTION. END CONNECTIONS SHALL BE STANDARD "FRAMED BEAM" BEARING TYPE CONNECTIONS PER TABLE 10-1 OR 10-2 CASE 1 OF AISC MANUAL 13TH EDITION. UNLESS OTHERWISE INDICATED, THE FOLLOWING MINIMUMS APPLY TO TABLE 10-1 CONNECTIONS.



G & G1 = 2" MIN FOR STD HOLES (G1 =1 1/4" MIN FOR BEAMS LESS THAN A W12)
G & G1 = 2 1/4" MIN FOR OVERSIZED AND SHORT SLOTTED HOLES (G1 = 1 3/8" MIN FOR BEAMS LESS THAN A W12)
MINIMUM CLIP ANGLE THICKNESS SHALL BE 5/16" AT BEAM TO BEAM CONN
MINIMUM CLIP ANGLE THICKNESS SHALL BE 3/8" AT BEAM TO COLUMN CONN
TABLE 10-2 WELDS TO BE 3/16"

3. WHERE NO STANDARD "FRAMED BEAM" CONNECTIONS SYMBOL (X) IS INDICATED BEAM END CONNECTIONS SHALL BE IN ACCORDANCE WITH THESE NOTES AND WITH THE FOLLOWING MINIMUM NUMBER OF HIGH STRENGTH BOLTS:

C6, W6 OR LESS = 1 ROW (2 BOLTS ON WEB)

C8 OR W8 = 2 ROWS	W21 = 6 ROWS
C10 OR W10 = 2 ROWS	W24 = 7 ROWS
C12 OR W12 = 3 ROWS	W27 = 7 ROWS
W14 = 3 ROWS	W30 = 8 ROWS
C15 OR W16 = 4 ROWS	W33 = 9 ROWS
W18 = 5 ROWS	W36 = 10 ROWS

4. THE NUMBER OF BOLTS AND NUMBER OF ROWS OF BOLTS INDICATED OR STATED IS THE MINIMUM NUMBER OF BOLTS OR ROWS. PROVIDE ADDITIONAL BOLTS OR CONNECTION DEVICES, IF NECESSARY, TO COMPLY WITH OSHA REGULATION 29CFR1926 SUBPART R-STEEL ERECTION.
5. ROWS OF BOLTS: THE NUMBER OF FASTENERS IN A VERTICAL ROW
6. BEAM CONNECTIONS ARE BASED ON THE USE OF STANDARD, HOLES AS DEFINED BY AISC MANUAL 13th EDITION. SLOTTED AND OVERSIZED HOLES ARE NOT PERMITTED.

Figure 2.2 - Example of Standard Shear Connection Details

Different Types of Double Clip Angle Connections

Double Clip Angle Connections are designed to be one of three configurations: bolted-bolted connections; welded-bolted connections; and welded-welded connections.

Bolted-Bolted Connections

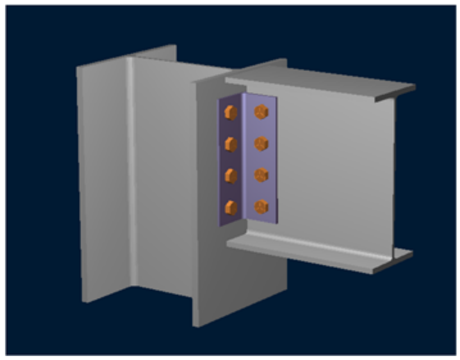


Figure 2.3 Bolted-Bolted Clip Angle

One configuration is called a bolted-bolted connection. For these connections, the clip angles are connected to both the connecting beam and the supporting member with bolts. The bolts on the connected beam are in double shear while the bolts on the supporting girder or column are in single shear. The clip angles are either shop-bolted to the connecting beam prior to being shipped to the job site or they are connected to the beam in the field. If the clip angles are not connected in the shop, the clip angle pieces are shipped loosely and need to be located and connected once the shipment arrives at the job site. It is often preferred by subcontractors to have the clip angles shop connected to reduce the amount of steps required once the steel arrives on site. In some cases, it may also be preferred to offset the holes of the beam and the supporting member to make erection easier or more feasible. If the bolts on the beam aren't offset from the supporting member, it can be difficult for the erector to get the tools in place and operating properly if they have to work around the set of bolts already installed at the supporting member.

Welded-Bolted Connections

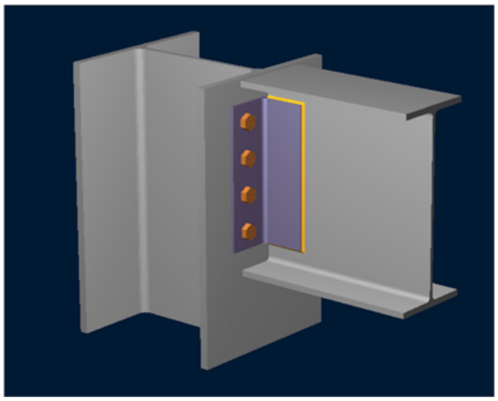


Figure 2.4 Welded-Bolted Clip Angle

The second configuration option is a welded-bolted connection. This connection type has the clip angles shop-welded to the connecting beam and then bolted to the supporting member in the field. This type of connection behaves in a similar manner to the bolted-bolted connection but requires a design of a weld to transfer the shear from the connecting beam through the clip angles and into the supporting member. When the angles are shop-welded, the beams arrive at the construction site with the clip angles already attached and the whole piece is ready to be installed and connected to the supporting member.

Welded-Welded Connections

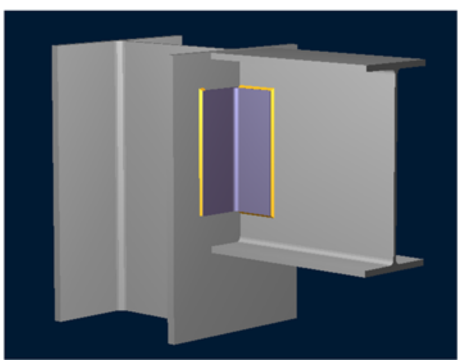


Figure 2.5 Welded-Welded Clip Angle

The third configuration option is a welded-welded connection. The clip angles in a welded-welded connection are shop-welded to the connecting beam and then field welded to the supporting member after arriving at the job site. The connection behaves similarly to the other configurations but in this case the weld at the supporting member will reduce the flexibility of the connection. Bolts and bolt holes allow for some movement, albeit small movement, at the connection. Welds create fixity and movement of the parts of the connection is not possible if they are welded in place. It is important when designing a welded-welded connection to make sure that making the weld in the field is possible with the geometry of the pieces being connected. For example, if the welder cannot fit their hands into the space available to make create the weld,it might be better to utilize a welded-bolted or bolted-bolted connection. Typically, engineers try to keep field welding to a minimum for cost effectiveness as well as safety reasons. The cost markup for a field weld on a controlled site like a building or low risk site can be an increase of 20%. For example, if 1’-0” of fillet weld in the shop costs $57, that same weld done in the field would cost $68. On a higher risk site, such as a refinery or petrochemical facility, the cost markup is closer to 35%. The $57 shop weld would end up costing closer to $77. (Burns & McDonnell Estimation Group, personal communication, March 2016).

Single Shear Plate Connections

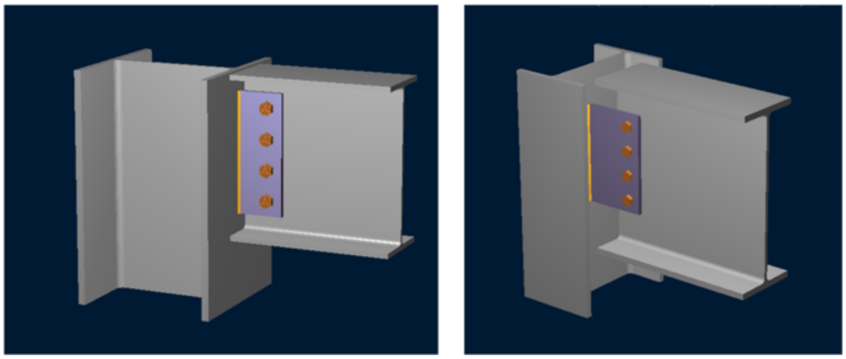


Figure 2.6 Single Shear Plate Connections

Single shear plate connections (SSPC) are used for connections where a beam frames into another beam or into a column flange or web. The connection consists of a structural steel plate (shear tab) welded to the supporting member on one end with the connecting beam bolted on the other end. The shear tab is typically located by the fabricator in the shop per the contract documents and shop-welded to the supporting member. The connecting beam then has the matching bolt pattern shop-drilled into its web. Each piece is then shipped to the site ready to be erected. The bolts for these connections are in single shear and the connection can transfer some moment into the supporting member.

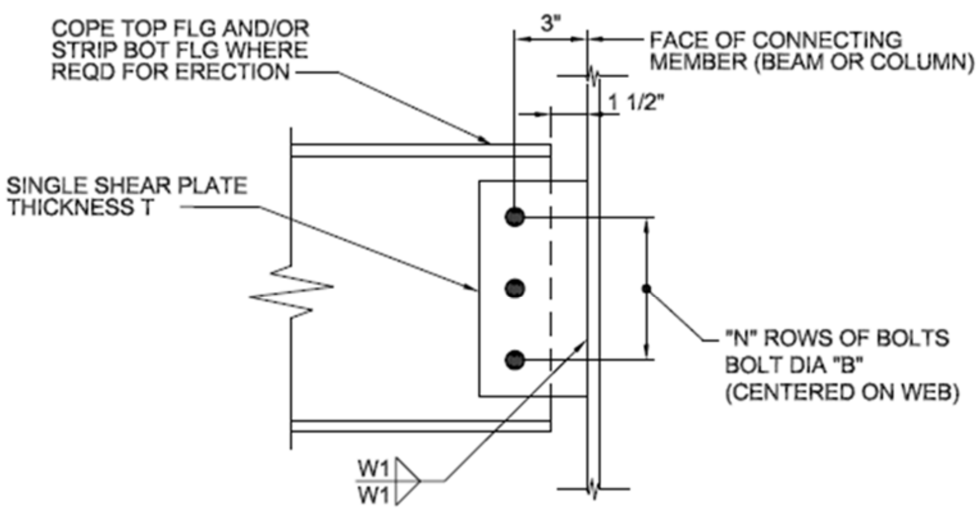
Different Types of Single Shear Plate Connections

Shear plate connections do not have options for different configurations like the Double Clip Angle Connections. They are categorized as standard shear tabs, extended shear tabs, and extended shear tabs with stiffeners.

Standard Shear Tabs

Standard shear tabs are single plate shear connections that utilize all of the minimum requirements of AISC 360 Specification Chapter J and the Manual. The bolt centerline is often located 3” from the connecting face of the plate. When the bolt centerline is 3” or less from the weld at the connecting member, the connection maintains its status as simply supported because only small end moments develop at the connection. 3” is the distance considered a maximum before the eccentricity of the connection develops other forces into the plate and the plate weld. (Sherman & Ghorbanpoor, 2002) The beam being connected to the shear plate with bolts usually is only setback between ½” to 2” from the face of the connecting beam to help maintain that 3” distance and the minimum bolt edge distances. If needed, the bottom and top flanges are coped as needed to erect the system.

Figure 2.7 indicates the details typically provided to fabricators for standard shear tab connections. The 3” dimension to the bolt centerline is clearly defined, the beam setback is shown to be ½”, and the fabricator is directed to cope the flanges as needed for erection. The engineer of record is ultimately responsible for the design of the connection, but the fabricator is responsible for constructability. This creates a mutual investment on both parties to work together to make sure the beams and connecting parts can be erected as detailed.



BEAM TO BEAM OR
BEAM TO COLUMN FLANGE

BEAM CONNECTION BS-SPL
SINGLE SHEAR PLATE
(USE WHERE INDICATED IN PLANS)

REQUIRED DATA	DEFAULT DATA
ROWS OF BOLTS "N"	MAX
BOLT DIAMETER "B"	0.75
PLATE THICKNESS "T"	0.375
PLATE WELD W1	0.25

(DIMENSIONS IN INCHES)

(MAX DENOTES THE MAXIMUM NUMBER OF ROWS OF BOLTS THAT THE MEMBER DEPTH WILL ALLOW BASED ON 3" PITCH)

Figure 2.7 - Standard Shear Tab Connection Detail

Extended Shear Tabs

In many situations, a beam frames into the web of a deep column and is needed for vertical shear transfer. Due to the size of the column and beam, the erection can be difficult if a standard shear tab is utilized. The flanges of the column can make it almost impossible to swing the beam into position, even with the beam flanges coped. For example, if a W12X40 section is used as a column, the section flange width is 8". The space from the end of the flange to the face of the web is about 3.85". If a standard shear tab is to be connected to the web of the column, the bolts are actually located between the flanges as demonstrated in Figure 2.8.

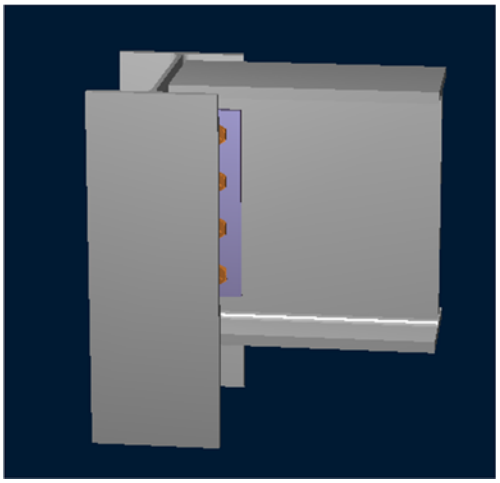


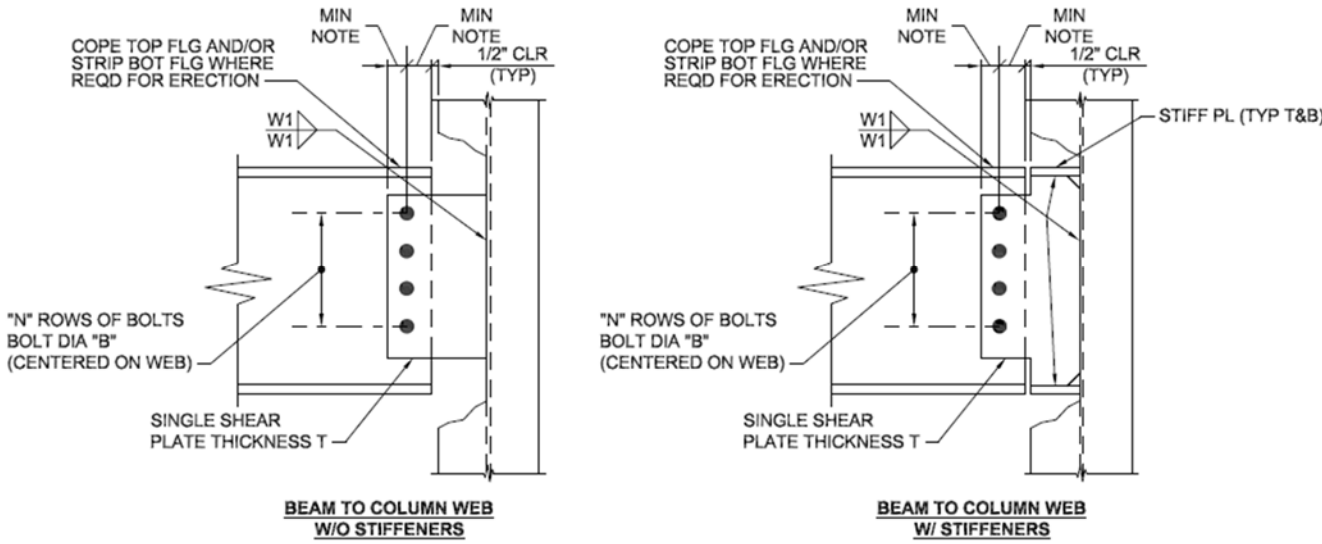
Figure 2.8 Shear Tab with Bolts between Flanges

The W16 beam shown in the figure would be very difficult to install. Typically, beams are hoisted up with cranes and then swung into place. With the above described scenario, the column flanges would block the end of the beam from getting into its final installed position and create unnecessary difficulties for the subcontractor in the field. This would be a waste of labor hours and may not be able to be solved without an engineered solution or new connection design.

In construction, every situation that stalls progress, especially if a new solution has to be engineered, creates a delay and a new project cost.

In these instances it is more practical to use an extended shear tab. Extended shear tabs have a much longer plate and the bolt line is actually located outside of the column flanges. The connected beam flanges are coped as needed similar to the standard shear tabs, but the beam is often setback beyond the column flanges in the same way the bolt centerline is setback. Depending on the depth of the column, the shear plate may end up being too long to be able to handle the eccentricity created by the connection. In these cases it can become necessary to add stiffeners to increase the strength of the connection and ensure the forces are transferred through the connection correctly. The loads dictate that multiple lines of bolts required to properly transfer the loads.

Figure 2.9 details typical extended shear plate connections. In these details, rather than dimensioning to the bolt centerline, the distance from the flange of the column to the flanges of the beam are dimensioned. Here that distance is noted as ½" clear. There are still notes to the fabricator regarding the coping of the beams for erection. The connection detailed in 2.9(b) shows how stiffeners should be detailed and fabricated if required.



(a) Extended Shear Connection (b) Extended Shear Connection with Stiffener plates

Figure 2.9 - Extended Shear Connection

Chapter 3 - Development of Study Model for Basis of Comparison

Both the double clip angle connections (DCAC) and the single shear plate connections (SSPC) are suitable for small shear load transfer and typical design conditions. However, each type has advantages and disadvantages along with limitations as to what the connections are capable of providing in design. The four main items to be compared for the purposes of this report are: the design approach, fabrication costs, installation costs including the effects on fire proofing block-outs, and how the installation procedures for each type affect life safety during erection.

For the purposes of this comparative study, an open-steel structure was chosen for the basis of comparison. The structure serves as an equipment structure which also contains multiple levels of pipe racks. This type of structure was chosen because often many simple shear connections and fireproofing of the structure are required. These types of structures present a good opportunity to study the design, installation and cost differences because they are so prevalent in the structural engineering industry. Figure 3.1 gives an example of how steel framing is typically laid out in open equipment structures and pipe racks. This is also the finished construction drawing associated with the study model utilized for parametric study.

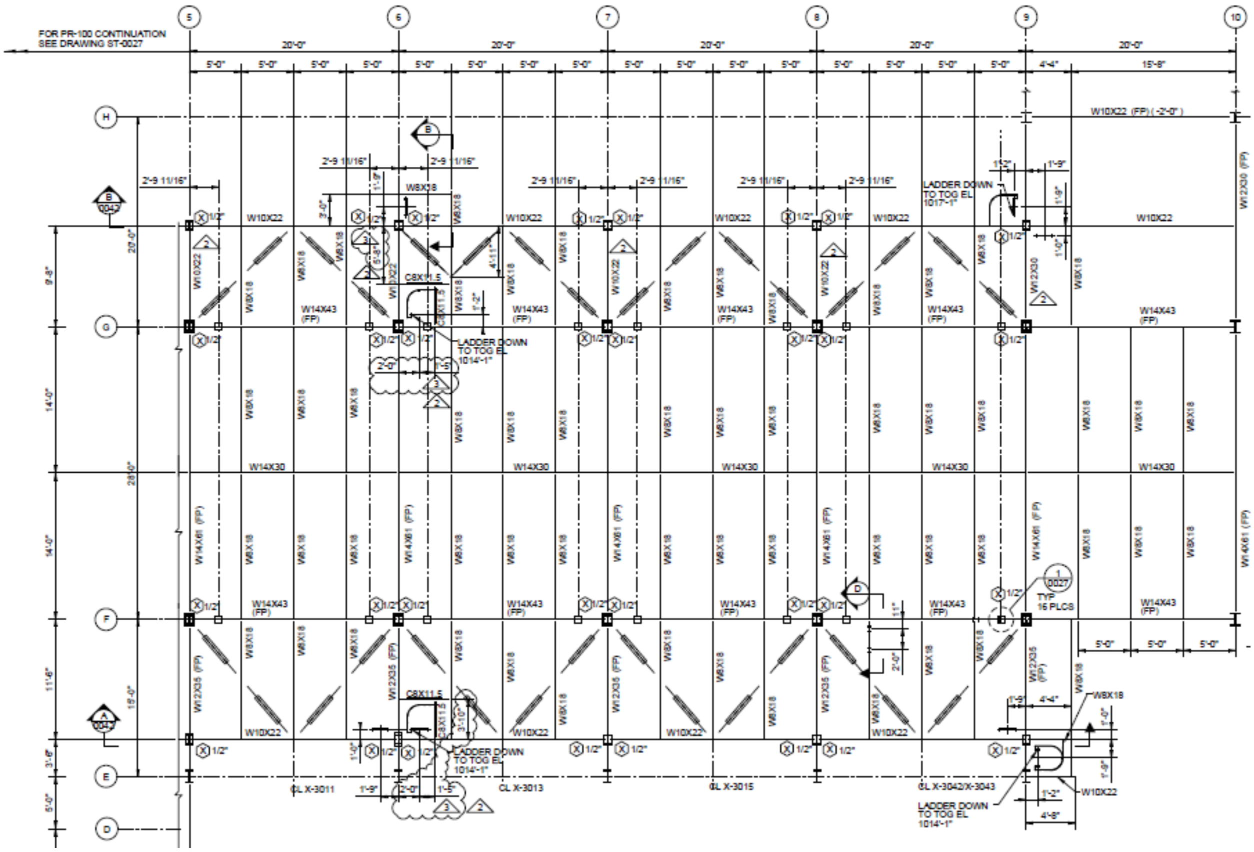


Figure 3.1 Open Equipment Structure Framing Plan Example

Development of Study Model

Codes, Standards, Load Cases and Load Combinations

The equipment structure was analyzed using the software RISA 3d, version 13.0. The codes used for the design and analysis were IBC 2009, ASCE 7-05, and AISC 13th edition. This project was located in Oklahoma and the refinery operated as their own governing code entity. At the time of the project, they had not adopted the IBC 2012 or ASCE 7-10. Process Industry Practices (PIP) were also used as a reference for the design because of the location within an active refinery. The structure was assumed to be geographically located in Oklahoma. The Basic

Load Cases defined for the RISA Model are shown in Table 3.1. These load cases are typically developed from the PIP standards alongside the IBC and ASCE 7 to correspond to the Load Combinations that will be required for the analysis. The load combinations used for this study can be seen in Table 3.2.

Table 3.1 Basic Load Cases for RISA 3d Model.

Load Case	Description
D_s	self-weight
D_e	empty dead load
D_o	operating dead load
D_t	test dead load
L	live load
S	snow load
F_f	friction forces
A_f	pipe anchor/guide forces
W(X)	equip/pipe wind load in +/
W(Z)	equip/pipe wind load in +/
$E_o(X)$	operating earthquake X-di
$E_o(Z)$	operating earthquake Z-di
$E_e(X)$	empty earthquake X-dir
$E_e(Z)$	empty earthquake Z-dir
$NL_o(X)$	operating notional load
$NL_o(Z)$	operating notional load
$NL_e(X)$	empty notional load
$NL_e(Z)$	empty notional load
$NL_t(X)$	test notional load
$NL_t(Z)$	test notional load
FP	Fireproofing

Table 3.2 Load Combinations for RISA 3d Model.

ASD LOAD COMBINATIONS	LRFD LOAD COMBINATIONS
ASD 1: 1.0(Ds + Do + Ff + Af) + NLo(X)	STR 1: 1.4(Ds + Do + Ff + Af)+1.4NLo(x)
ASD 1: 1.0(Ds + Do - Ff + Af) - NLo(X)	STR 1: 1.4(Ds + Do - Ff + Af)-1.4NLo(x)
ASD 1: 1.0(Ds + Do + Ff + Af) + NLo(Z)	STR 1: 1.4(Ds + Do + Ff + Af)+1.4NLo(z)
ASD 1: 1.0(Ds + Do - Ff + Af) - NLo(Z)	STR 1: 1.4(Ds + Do - Ff + Af)-1.4NLo(z)
ASD 2: 1.0(Ds + Do) + 1.0L + 1.0Ff + 1.0Af + NLo(X)	STR 2: 1.2(Ds + Do) + 1.6L+ 0.5S + 1.2Af+1.2NLo(x)
ASD 2: 1.0(Ds + Do) + 1.0L - 1.0Ff - 1.0Af - NLo(X)	STR 2: 1.2(Ds + Do) + 1.6L+ 0.5S + 1.2Af-1.2NLo(x)
ASD 2: 1.0(Ds + Do) + 1.0L + 1.0Ff + 1.0Af + NLo(Z)	STR 2: 1.2(Ds + Do) + 1.6L+ 0.5S + 1.2Af+1.2NLo(z)
ASD 2: 1.0(Ds + Do) + 1.0L - 1.0Ff - 1.0Af - NLo(Z)	STR 2: 1.2(Ds + Do) + 1.6L+ 0.5S + 1.2Af-1.2NLo(z)
ASD 3: 1.0(Ds + Do + Af) + 0.7W(X) + S + NLo(X)	STR 3: 1.2(Ds + Do + Af) + 1.6S + 1.0L+1.2NLo(x)
ASD 3: 1.0(Ds + Do + Af) - 0.7W(X) + S - NLo(X)	STR 3: 1.2(Ds + Do + Af) + 1.6S + 1.0L-1.2NLo(x)
ASD 3: 1.0(Ds + Do + Af) + 0.7W(Z) + S + NLo(Z)	STR 3: 1.2(Ds + Do + Af) + 1.6S + 1.0L+1.2NLo(z)
ASD 3: 1.0(Ds + Do + Af) - 0.7W(Z) + S - NLo(Z)	STR 3: 1.2(Ds + Do + Af) + 1.6S + 1.0L-1.2NLo(z)
ASD 4: 1.0(Ds + Do + Af) + 1.0W(X) + 0.75L + 0.75S + NLo(X)	STR 4: 1.2(Ds + Do + Af) + 1.0L + 0.5S + 1.6W(X)+1.2NLo(x)
ASD 4: 1.0(Ds + Do + Af) - 1.0W(X) + 0.75L + 0.75S - NLo(X)	STR 4: 1.2(Ds + Do + Af) + 1.0L + 0.5S + 1.6W(Z)+1.2NLo(z)
ASD 4: 1.0(Ds + Do + Af) + 1.0W(Z) + 0.75L + 0.75S + Nlo(Z)	STR 4: 1.2(Ds + Do + Af) + 1.0L + 0.5S - 1.6W(X)-1.2NLo(x)
ASD 4: 1.0(Ds + Do + Af) - 1.0W(Z) + 0.75L + 0.75S - NLo(Z)	STR 4: 1.2(Ds + Do + Af) + 1.0L + 0.5S - 1.6W(Z)-1.2NLo(z)
ASD 5: 1.0(Ds + Do + Af) + 0.7Eo(X) + 0.75L + 0.75S + NLo(X)	STR 5: 1.2(Ds + Do + Af) + 1.0Eo(X) + 1.0L + 0.2S+1.2NLo(X)
ASD 5: 1.0(Ds + Do + Af) - 0.7Eo(X) + 0.75L + 0.75S - NLo(X)	STR 5: 1.2(Ds + Do + Af) - 1.0Eo(X) + 1.0L + 0.2S-1.2NLo(X)
ASD 5: 1.0(Ds + Do + Af) + 0.7Eo(Z) + 0.75L + 0.75S + NLo(Z)	STR 5: 1.2(Ds + Do + Af) + 1.0Eo(Z) + 1.0L + 0.2S+1.2NLo(Z)
ASD 5: 1.0(Ds + Do + Af) - 0.7Eo(Z) + 0.75L + 0.75S - NLo(Z)	STR 5: 1.2(Ds + Do + Af) - 1.0Eo(Z) + 1.0L + 0.2S-1.2NLo(Z)
ASD 6: 0.6(Ds) + De + 1.0W(X) + NLo(X)	STR 6: 0.9(Ds + De) + 1.6W(X)+0.9NLe(X)
ASD 6: 0.6(Ds) + De - 1.0W(X) - NLo(X)	STR 6: 0.9(Ds + De) - 1.6W(X)-0.9NLe(X)
ASD 6: 0.6(Ds) + De + 1.0W(Z) + NLo(Z)	STR 6: 0.9(Ds + De) + 1.6W(Z)+0.9NLe(Z)
ASD 6: 0.6(Ds) + De - 1.0W(Z) - NLo(Z)	STR 6: 0.9(Ds + De) - 1.6W(Z)-0.9NLe(Z)
ASD 7: 0.6(Ds) + De + 0.7Eo(X) + NLo(X)	STR 7: 0.9(Ds + De) + 1.0Ee(X)+0.9NLe(X)
ASD 7: 0.6(Ds) + De - 0.7Eo(X) - NLo(X)	STR 7: 0.9(Ds + De) - 1.0Ee(X)-0.9NLe(X)
ASD 7: 0.6(Ds) + De + 0.7Eo(Z) + NLo(Z)	STR 7: 0.9(Ds + De) + 1.0Ee(Z)+0.9NLe(Z)
ASD 7: 0.6(Ds) + De - 0.7Eo(Z) - NLo(Z)	STR 7: 0.9(Ds + De) - 1.0Ee(Z)-0.9NLe(Z)
ASD 8: 1.0(Ds + Dt) + Wp(X) + NLt(X)	STR 8: 1.2(Ds + Dt) + 1.6*Wp(X)+1.2NLt(X)
ASD 8: 1.0(Ds + Dt) - Wp(X) - NLt(X)	STR 8: 1.2(Ds + Dt) - 1.6*Wp(X)-1.2NLt(X)
ASD 8: 1.0(Ds + Dt) + Wp(Z) + NLt(Z)	STR 8: 1.2(Ds + Dt) + 1.6*Wp(Z)+1.2NLtZ)
ASD 8: 1.0(Ds + Dt) - Wp(Z) - NLt(Z)	STR 8: 1.2(Ds + Dt) - 1.6*Wp(Z)-1.2NLt(Z)

Structure Dimensions, Layout and Member Shapes

The equipment structure is 181’-6” long with column spacing at 20’-0” on center. The main part of the structure is set at 43’-0” wide to accommodate the equipment placed on the top level. A stair tower and piping access platforms add some width to the ends of the structure for a total of 68’-0” at these locations. All top of steel elevations are measured from the bottom of base plate. Each column is assumed to bear on a 1’-0” tall concrete pedestal. The main pieces of equipment are located on the main level at top of steel elevation 37’-0”. The upper platforms and walkways are located at top of steel elevations 46’-0” and 69’-0”. Additional levels at top of steel elevations 17’-0” and 27’-0” are to be used to support operating pipe. Cable tray is assumed to be routed through the level located at top of steel elevation 33’-0” to allow power to reach the elevated equipment. Figure 3.2 shows the main structural plan view.

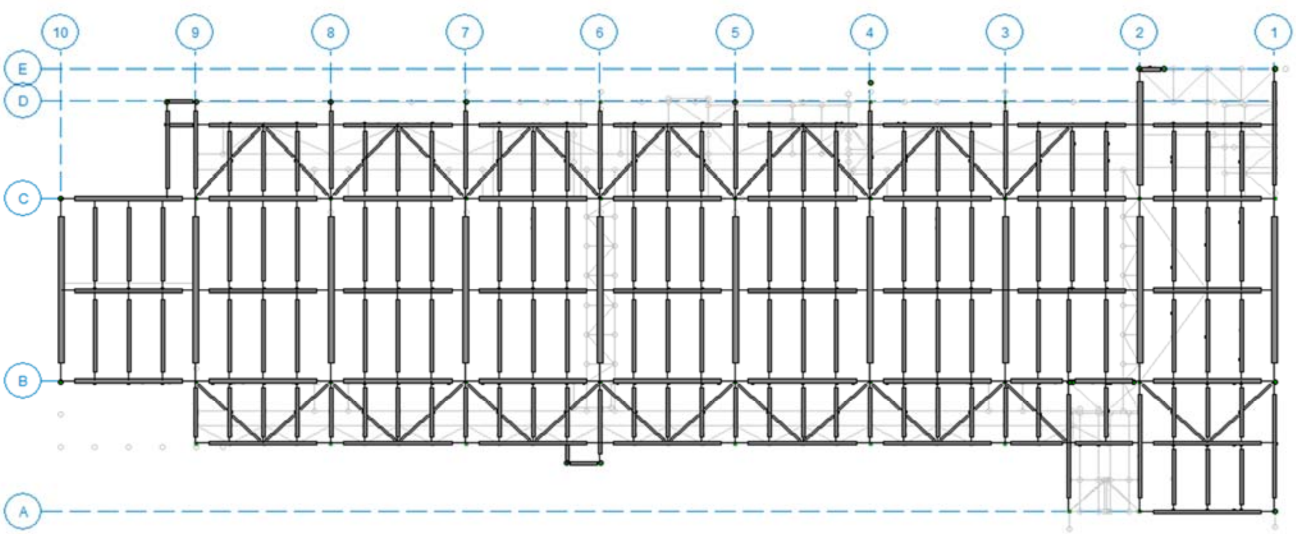


Figure 3.2 - Structural Plan View

The structural form adopted for this structure is moment frames in the shorter direction and braced bays in the longer direction. The moment frames allowed for the routing of piping, equipment and cable tray through spaces that would otherwise be blocked by vertical braces. The vertical brace type used on this structure is Chevron bracing to further allow pipe and cable tray to route in and out of the rack as needed to reach equipment. The bracing layout can be seen in Figure 3.4. The vertical braces are located in 2 Bays on each side of the structure. Wide flange members are used exclusively for the main structural columns and beams. WT shapes are utilized for all main vertical and horizontal bracing members including small kicker braces. Figure 3.3 is an example of a standard kicker brace and its connections. Angle shapes are used for horizontal bracing at some small walkway locations. Channel shapes are used for stair stringers and for miscellaneous ladder supports.

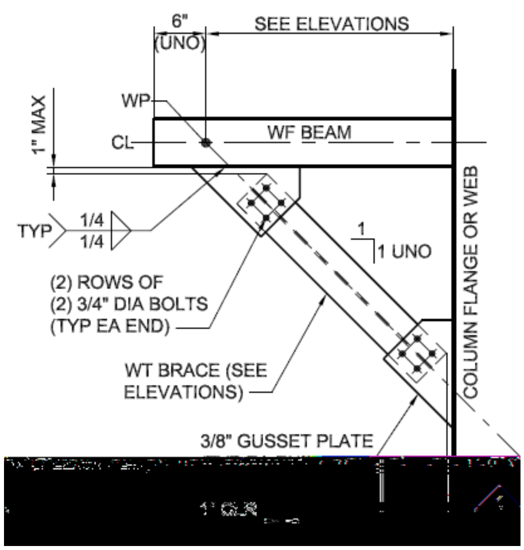


Figure 3.3 Typical Kicker Brace Detail

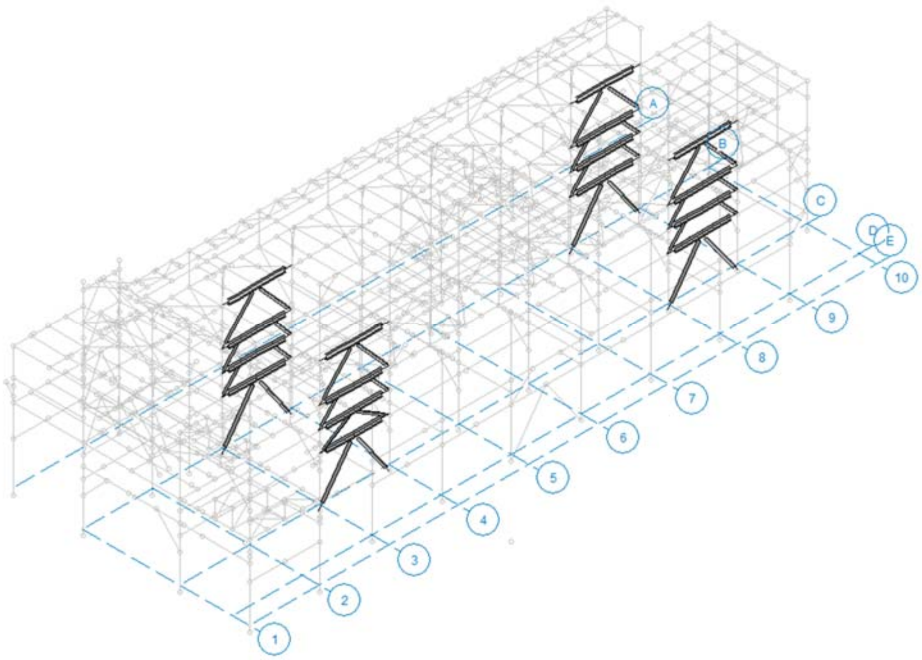


Figure 3.4 - Bracing Layout

Applied Loads

The loads applied on the structure are outlined in Table 2.3 below. Individual equipment gravity loads were provided by vendors and incorporated into the final design. Wind load is distributed to the individual structural components based on the size of the member. A seismic C_s value is applied in the Basic Load Cases table for the structure. A C_s value is a factor applied to the dead load of a structure to determine the forces induced on a structure in a seismic event. Wind and Seismic reactions from equipment base plates are applied per the information provided by the vendors.

Table 3.3 Loads Applied to Structure in RISA Model

Loads Applied	Column1	Column2
Ds	Grating Load	12 psf
Ds	Handrail Load	15 plf
De	Empty Pipe Load	20 psf
De	Empty Equipment Loads	Varies*
De	Empty Cable Tray Load	16 psf
Do	Operating Pipe Load	40 psf
Do	Operating Equipment Load	Varies*

Do	Operating Cable Tray Load	20 psf
Dt	Testing Pipe Load*	40 psf
Dt	Testing Equipment Loads	Varies*
L	Elevated Platform Live Loads	75 psf
S	Snow On Platforms/Stairs	16 psf
Ff	Friction Force on Pipe	Varies*
Af	Anchor Force from Pipe	Varies*
W(X)/W(Z)	Wind on Structure/Equip.	90 mph
Eo(X)/Eo(Z)	Cs Value on Structure/Equip.	0.06

Connections For Basis of Comparison

A number of connection locations of beams into column webs as well as beam to beam interface connections from the structure were chosen for the comparison between DCAC and SSPC. A standard stringer beam from both sides of the structure was chosen (Figure 3.5) to compare how a regularly loaded shear connection may be analyzed with both types of connections being studied. A beam from a braced bay was chosen from both sides of the structure in different bays (Figure 3.6) to analyze how axial loads may affect the design of each type of connection. Two beams supporting large equipment loads were chosen to study how increased shear loads may affect the efficiency of each type of connection (Figure 3.7). Four beam to beam connections with small and simple loads were chosen to see how interchangeable the two connection types are when loads do not significantly affect the connection (Figure 3.8).

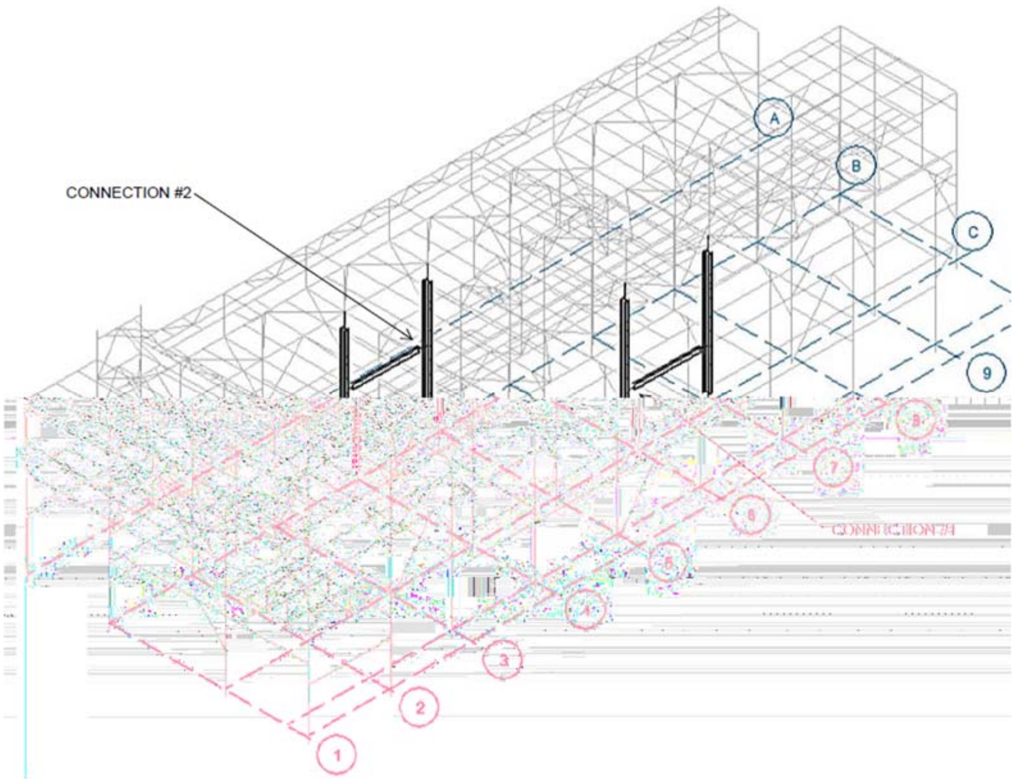


Figure 3.5 - Connection #1 and Connection #2

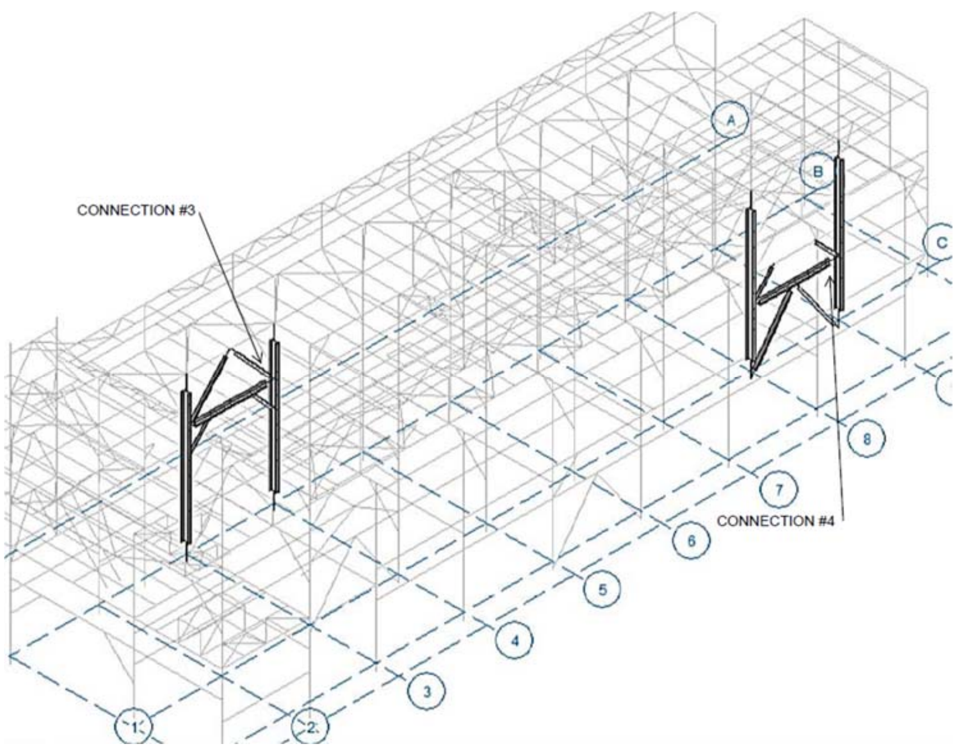


Figure 3.6 - Connection #3 and Connection #4

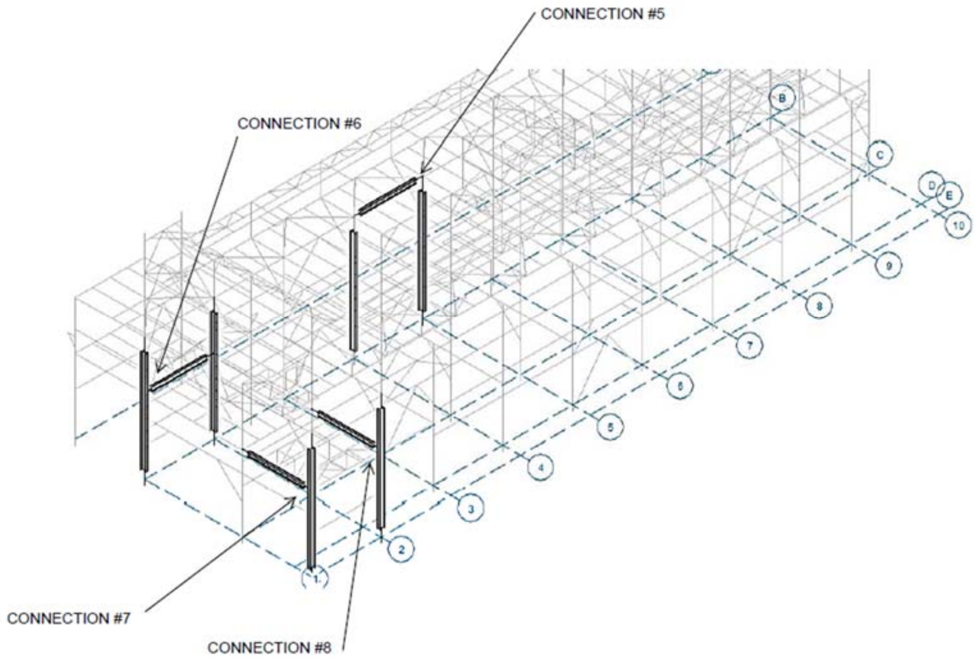


Figure 3.7 - Connection #5, #6, #7 and #8

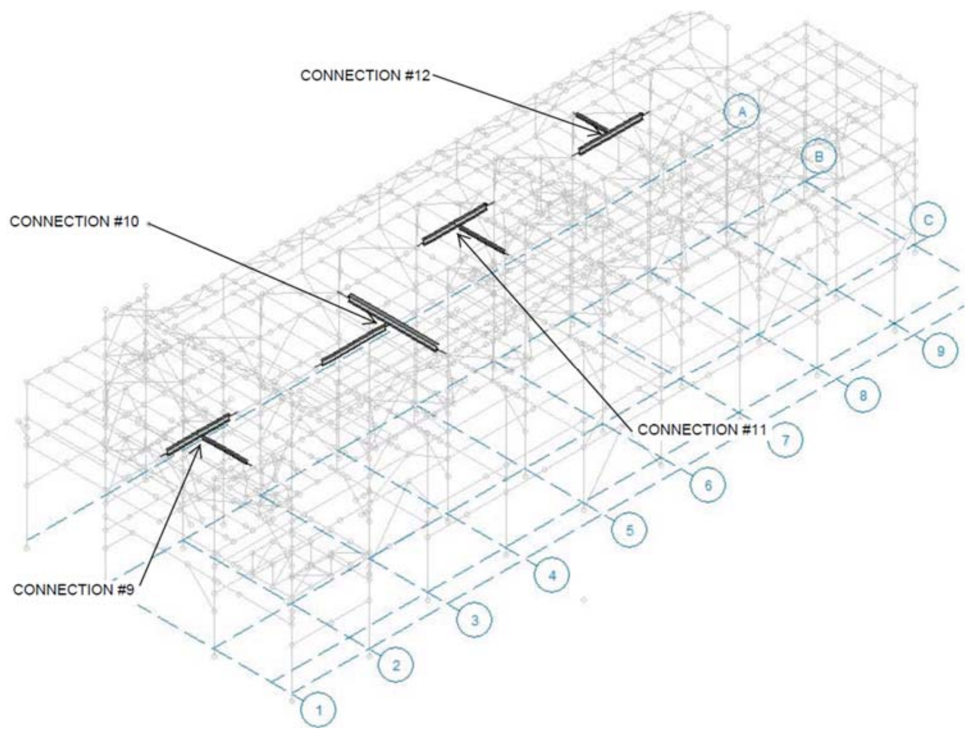


Figure 3.8 - Connection #9, #10, #11 and #12

Gathering Results from RISA Model

The RISA Model was solved using the ASD Load Combinations from Table 3.2. Each connection joint's load results were recorded from the model to be used for the connection design. The loads for each connection are identified in Table 3.4.

Table 3.4 Worst Case Envelope Loads for Each Connection

Connection	Member ID	Axial Load (kips)	Load Combination	Shear Load - vertical (k)	Vertical Shear Load Combination	Shear Load - Lateral (k)	Lateral Shear Load Combination
1	M198	-9.7	8	1.5	2	.29	14
2	M195A	-2.2	16	1.5	2	.32	14
3	M827B	-27.3	16	-8.3	16	.37	14
4	M202	-27.3	16	.79	15	-.46	2
5	M75	20.1	7	30.6	7	-1.6	14
6	M1135E	-3.1	4	23.3	6	.75	14
7	M588	19.9	6	-7.8	6	.76	16
8	M594	-5.4	3	-5.9	6	.39	16
9	M855	-1.8	7	6.12	6	.05	29
10	M803	.36	21	-9.38	6	-.38	14
11	M372	.45	17	3.13	6	.03	21
12	M323	2.33	14	2.01	7	.02	21

The envelope results from Table 3.4 were used to identify the worst case loads transferred to each connection. This is a conservative design for comparative purposes. The connections only need to be designed for specific load cases. The RISA model was then solved with a batch routine so each of the load combinations could be reviewed for connections #1 through #8. The worst case load combinations for each of the connections are outlined in Table 3.5.

Table 3.5 Worst Case Batch Routine Load Combinations for Each Connection

Connection	Load Combination	Axial Load (kips)	Shear Load - vertical (kips)	Shear Load - Lateral (kips)
1	2	-9.4	1.5	.05
1	8	-9.7	1.5	.05

1	14	-6.2	1.5	.29
2	2	-.36	1.5	.05
2	14	-.63	1.5	.32
2	16	-2.2	1.5	0
3	14	-21.9	-5.6	.37
3	16	-27.3	-8.3	.02
4	2	-25.1	.79	-.46
4	16	-27.3	.69	.16
5	7	20.1	30.6	-.93
5	14	19.7	29.9	-1.6
6	4	-3.1	13.8	.67
6	6	-3.0	23.4	.72
6	14	-1.7	22.5	.74
7	6	19.9	-7.8	.14
7	16	1.2	-3.3	.76
8	3	-5.4	2.5	.16
8	6	5.2	-5.9	.02
8	16	-.07	-1.8	.39
9	7	-1.78	6.11	0
9	6	-.23	6.12	0
9	29	0	.53	.05
10	21	.36	-8.69	.13
10	6	.01	-9.38	-.23
10	14	.06	-8.69	-.38
11	17	.45	2.90	0
11	6	.40	3.13	0
11	21	.36	2.89	.03
12	14	2.33	1.86	0
12	7	1.63	2.01	0
12	21	1.31	1.86	.02

Connection Design

Connections were designed using the Bentley analysis program RAM Connection Standalone, version 9. Hand calculations were done to verify the accuracy of each type of connections. Each connection was first designed as a clip angle connection and then designed as a shear tab connection. The loads from each of the load combinations identified in Table 3.5 were input into the connection design program.

Connections as Double Clip Angle Connections

Each connection was first designed assuming a Welded-Bolted connection of the beam into the column web. Standard framed beams follow the parameters in AISC 13th Edition Table 10-1 or 10-2.

Table 3.6 Double Clip Angle Connection Results

Connection	Governing Load Combination	Clip Angle Size Required	Number of Bolts Required	Weld Size Required	Standard Fabrication for "Framed Beam"?	% Connection Capacity Used
1	8	L3X3X0.375	3	3/16	YES	11%
2	53	L3X3X0.375	3	3/16	YES	5%
3	16	L3X3X0.375	3	3/16	YES	35%
4	16	L3X3X0.375	3	3/16	YES	30%
5	7	L3X3X0.375	3	3/16	YES	51%
6	6	L3X3X0.375	3	3/16	YES	37%
7	6	L3X3X0.375	3	3/16	YES	49%
8	6	L3X3X0.375	3	3/16	YES	13%
9	7	L3X3X0.3125	2	3/16	YES	38%
10	6	L3X3X0.3125	2	3/16	YES	59%
11	6	L3X3X0.3125	3	3/16	YES	5%
12	14	L3X3X0.3125	2	3/16	YES	16%

Connections as Shear Tab Connections

After the connections were designed as Welded-Bolted clip angle connections, the same files were updated and designed as shear tab or extended shear tab connections. Standard framed beam connections for the shear tab utilize the parameters of AISC 13th Edition Table 10-9a or 10-9b.

Table 3.7 Shear Tab Connection Results

Connection	Governing Load Combination	Shear Tab Thickness Required	Number of Bolts Required	Weld Size Required	Standard Fabrication for "Framed Beam"?	Plate Stiffeners Required?	% Connection Capacity Used
1	8	0.375	3	1/4	YES	NO	38%
2	16	0.375	3	1/4	YES	NO	17%
3	16	0.375	6	1/4	NO	YES	64%
4	16	0.375	3	1/4	YES	NO	79%
5	7	0.5	12	5/16	NO	YES	85%
6	6	0.375	8	1/4	NO	YES	85%
7	6	0.375	4	1/4	YES	NO	71%
8	6	0.375	3	1/4	YES	NO	59%
9	7	0.375	2	1/4	YES	NO	51%
10	6	0.375	2	1/4	YES	NO	77%
11	6	0.375	3	1/4	YES	NO	11%
12	14	0.375	2	1/4	YES	NO	18%

Chapter 4 - Comparison of Results

Using the results outlined in Chapter 3, assumptions can be made regarding how the remaining connections on the structure would be designed. The following tables were developed by assuming either all double clip angles or all shear tabs would be utilized for the entire structure. The entire structure is assumed to have 100 simple shear beam-to-column web connections and 100 simple shear beam-to-beam connections.

Table 4.1 Double Clip Angle Connection Distribution – Beam-to-Column

Connection	% of Total Shear Connections	# of Connections
Standard Clip Angle	85%	85
Non-Standard Clip Angle*	15%	15
*Non-Standard Clip Angle assumed to require 1/2" thick angles		

Table 4.2 Shear Tab Connection Distribution – Beam-to-Column

Connection	% of Total Shear Connections	# of Connections
Standard or Extended Shear Tab w/o Stiffeners	60%	60
Extended Shear Tab w/ Stiffeners	25%	25
Extended Shear Tab w/ Stiffeners & Multiple Rows of Bolts	15%	15

Table 4.3 Double Clip Angle Connection Distribution – Beam-to-Beam

Connection	% of Total Shear Connections	# of Connections
Standard Clip Angle	100%	100
Non-Standard Clip Angle*	0%	0
*Non-Standard Clip Angle assumed to require 1/2" thick angles		

Table 4.4 Shear Tab Connection Distribution – Beam-to-Beam

Connection	% of Total Shear Connections	# of Connections
Standard or Extended Shear Tab w/o Stiffeners	100%	100
Extended Shear Tab w/ Stiffeners	0%	0
Extended Shear Tab w/ Stiffeners & Multiple Rows of Bolts	0%	0

Comparison of Design Approach

Design of Double Clip Angle Connections

After the main structure has been designed, the connection design begins by analyzing the worst case loading from the load combinations for each connection. Once this data has been collected, the worst loads are applied to the design of connections. When utilizing DCAC connections, most of the iterative process has been removed from the initial design and from the detail development. Tables have been developed and verified, and many companies have standards which they utilize to make the design more efficient. DCAC have been used in major engineering firms for so long they are typically the default. The design engineer simply needs to look at the standards that apply to each of their connections and check that the standard is in-fact sufficient for the loads being applied to the connection in question. The design checks for welded-bolted double angle connections are:

1. Strength of the bolt Group
    a. Single shear
    b. Bearing on supporting member
    c. Bearing on angles
2. Shear yielding of the angles
3. Shear rupture of the angles

4. Block shear rupture of the angles
5. Shear strength of coped section (if applicable)
6. Bending and Buckling of coped section (if applicable)
7. Strength of weld

This check can now be completed with most standard connection software such as RAM Connection or Descon. Design engineers are often familiar with DCAC and this process can happen in a quick and efficient manner depending on the loads being applied. These connections rarely require special detailing and so utilizing them can save time in the drawing development or modeling phase of a project as well.

Design of Shear Tab Connections

Shear tab connection design, unlike double clip angles, is less familiar to many design engineers, detailers and steel fabricators. This is mainly because the design of them requires more work on the front end. Shear tab connections at the outset typically are not as strong as their double clip angle counterparts and the farther the plate is extended out the less strength is available because of the eccentricity of the connection. Chapter 10 and Table 10-9 in the AISC 13th Edition manual provide basic starting points based on the shear being transferred, but each connection can present issues depending on the geometry. The available strength of a single plate or shear tab connection is dependent upon the limit states of the bolts, the plate, the web of the connected beam, the web or flanges of the connecting member, and the weld of the plate to the connecting member. The design checks for shear tab connections are:

1. Strength of the Bolt Group:
    a. Bolt shear
    b. Block shear rupture

c. Bolt Bearing

d. Bolt Bearing with Eccentricity (on extended plates)

2. Strength of the Plate:

    a. Shear yielding

    b. Shear rupture

    c. Block shear rupture (on extended plates)

    d. Flexure (on extended plates)

    e. Buckling (on extended plates)

3. Maximum plate thickness (on extended plates)

    a. Plate moment strength must not exceed the moment strength of the bolt group in shear.

4. Strength of weld

5. Stiffeners if required (on extended plates)

6. Strength of stiffener weld connection (on extended plates)

Comparison of Fabrication

The fabrication approach for simple shear connections can be affected by a few variables. These variables include the size and quantity of product output of the shop, the setup and configuration of the fabrication and production lines of the shop, and the project schedule or contract dates agreed upon between the shop and the procuring party.

Many smaller steel fabrication shops are setup to do Double Clip Angle Connections very efficiently. These can be either bolted-bolted or welded-bolted depending on the equipment they utilize, but generally they default to these types of connections unless the member size dictates a shear tab is required. This allows them to be competitive with larger shops and maintain efficient schedules on jobs that are mainly comprised of simple shear connections. (S. Shroeder of Kreco Steel, personal communications, February 2016)

Larger fabrication shops can typically produce either type of connection easily, but the standard shear tabs tend to be more efficient because the only thing required is a single plate and weld. (R. Sardelli of Markle Manufacturing, personal communications, April 2016).

Comparison of Cost

Fabrication Costs

In comparing the costs associated with the two connection types, both large and small shops charge about the same for the two basic connection types. Smaller shops may push to utilize one over the other because of the way their shop is setup for fabrication scheduling, but this doesn't significantly affect the end price of a job. However, SSPC without stiffeners do result in about half the welding and/or material than DCAC so some cost savings to utilizing the shear tabs exists. Both large and small shops charge significantly more for the stiffened shear tab connection.

Working with the welding prices previously mentioned and the average depth of the members utilized in the study model, the costs associated with the results in Tables 4.1 through 4.4 lead to the following cost breakdowns:

Table 4.5 Fabrication Costs Associated with Beam-to-Column Double Clip Angles

Connection Type	Material Weight (Lbs.)	Cost per Pound	Length of Weld in Connection (inches)	Cost per Foot	Cost Per Connection	Number of Connections	Total Cost
Standard Clip Angle	11	$1.75	30	$57	$161.75	85	$13748.75
Thickened Clip Angle	14	$1.75	30	$57	$167.00	15	$2505.00
						TOTAL CONNECTION COST	$16253.75

Table 4.6 Fabrication Costs Associated with Beam-to-Column Shear Tabs

Connection Type	Material Weight (Lbs.)	Cost per Pound	Length of Weld in Connection (inches)	Cost per Foot	Cost Per Connection	Number of Connections	Total Cost
Standard Shear Tab	6.5	$1.75	9	$57	$54.13	60	$3247.50
Extended With Stiffeners	26.50	$1.75	72	$57	$388.38	25	$9709.50
Extended with Stiffeners & Extra Bolt Columns	35.0	$1.75	72	$57	$403.43	15	$6051.45
						TOTAL CONNECTION COST	$19008.45

Table 4.7 Fabrication Costs Associated with Beam-to-Beam Double Clip Angles

Connection Type	Material Weight (Lbs.)	Cost per Pound	Length of Weld in Connection (inches)	Cost per Foot	Cost Per Connection	Number of Connections	Total Cost
Standard Clip Angle	11	$1.75	30	$57	$161.75	100	$16175.00
Thickened Clip Angle	14	$1.75	30	$57	$95.75	0	$0
						TOTAL CONNECTION COST	$16175.00

Table 4.8 Fabrication Costs Associated with Beam-to-Beam Shear Tabs

Connection Type	Material Weight (Lbs.)	Cost per Pound	Length of Weld in Connection (inches)	Cost per Foot	Cost Per Connection	Number of Connections	Total Cost
Standard Shear Tab	6.5	$1.75	9	$57	$54.13	100	$5412.50
Extended With Stiffeners	26.50	$1.75	72	$57	$388.38	0	$0
Extended with Stiffeners & Extra Bolt Columns	35.0	$1.75	72	$57	$403.43	0	$0
TOTAL CONNECTION COST							$5412.50

Reviewing just the fabrication costs between the four scenarios outlined in this section, potential cost savings with pointed changes to the design approach become apparent. One approach might be to utilize DCAC for all beam-to-column connections, but change all beam-to-beam connections to SSPC. This utilizes the most cost effective connection for the different loads being applied. Another approach might be to utilize shear tabs in all shear connections unless extended stiffeners are required. This would present an opportunity to avoid the costs associated with the extra material and welding. Also, if it would not add significant labor and time to the erection process, using Bolted-Bolted DCAC would cut down on a lot of the welding associated with the preferred Welded-Bolted DCAC configuration. (Burns & McDonnell Estimating Group, personal communications, March 2016; R. Sardelli of Markle Manufacturing, personal communications, April 2016).

Fireproofing Costs

Many open structures on high risk sites (sites with highly flammable processes and materials) require fireproofing to ensure the stability of the structure in a catastrophic event. The rating of the fireproofing as well as the type is typically dictated by the insurance underwriter for the client or site. Two of the types most often used in this type of construction are Lightweight

Cementitious and Intumescent (Burns & McDonnell Construction Group, personal communications, March 2016). Fireproofing of the steel is typically done in a shop after the steel is fabricated. The fireproofing supplier will then cover as much of the steel member as possible with the fireproofing, leaving connections exposed as needed for field erection. See figure All of the block-outs then have to be fire-proofed in the field after erection is complete and before the start-up of the process or equipment being supported. Field fire-proofing is a labor intensive and time consuming process which can add a lot of cost to a project. Figure 4.1 is a picture of a portion of this structure before the field fireproofing has been applied.

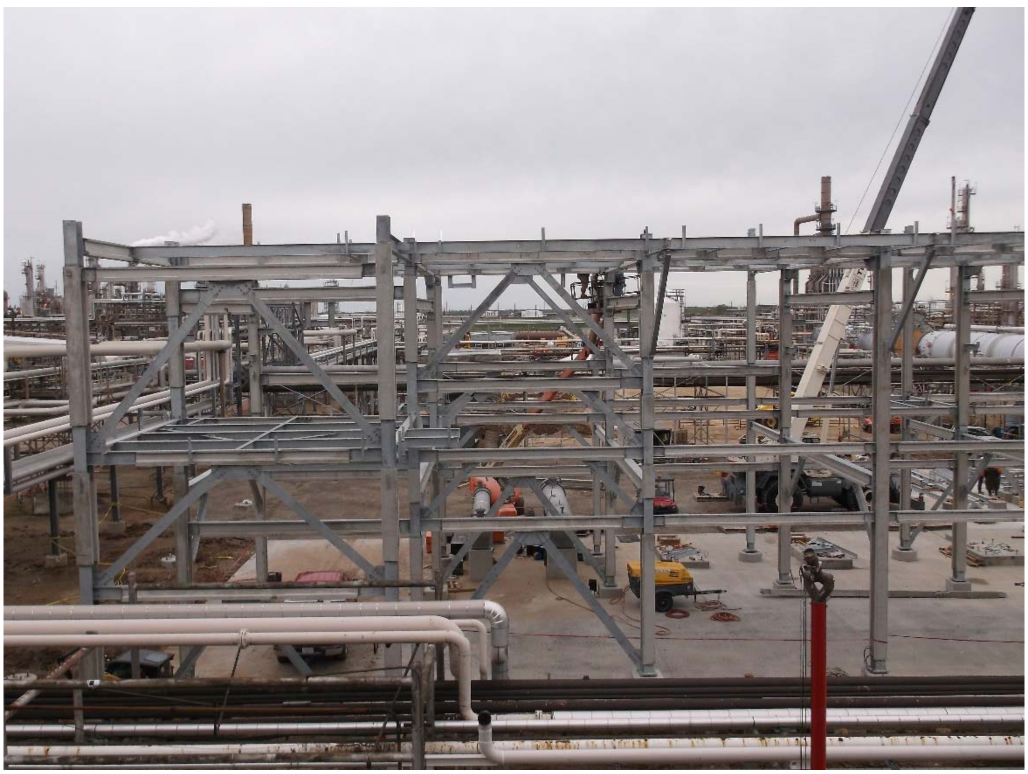


Figure 4.1 – Fireproofing Block-outs on Erected Steel

One of the benefits to be explored in utilizing shear tabs over clip angles is a reduction in the amount of fireproofing block-outs required for fit-up. The following tables explore the same four scenarios from the fabrication cost section and how they compare with fireproofing block-outs and costs. For the purposes of comparison, W14 size members were used for the basis of the beam and column block-outs.

Table 4.9 Fireproofing Costs Associated with Beam-to-Column Connections

Connection Type	Total Blockout Required for Field Erection (ft^2)	2 Hour Shop Applied Cost per ft^2	2 Hour Field Applied Cost per ft^2	3 Hour Shop Applied Cost per ft^2	3 Hour Field Applied Cost per ft^2	Cost of Leaving a Blockout – 2 Hour	Cost of Leaving a Blockout – 3 Hour	Number of Blockouts	Total Cost Increase for Block Out – 2 Hour	Total Cost Increase for Block Out – 3 Hour
Standard Clip Angle	8.1	$33.50	$116.75	$48.65	$132.70	$674.33	$680.81	100	$67433.00	$68080.50
Shear Tab	3.5	$33.50	$116.75	$48.65	$132.70	$291.38	$294.18	100	$29138.00	$29417.50
	Cost Differential								$38295.00	$38663.00

Table 4.10 Fireproofing Costs Associated with Beam-to-Beam Connections

Connection Type	Total Blockout Required for Field Erection (ft^2)	2 Hour Shop Applied Cost per ft^2	2 Hour Field Applied Cost per ft^2	3 Hour Shop Applied Cost per ft^2	3 Hour Field Applied Cost per ft^2	Cost of Leaving a Blockout – 2 Hour	Cost of Leaving a Blockout – 3 Hour	Number of Blockouts	Total Cost Increase for Block Out – 2 Hour	Total Cost Increase for Block Out – 3 Hour
Standard Clip Angle	8.8	$33.50	$116.75	$48.65	$132.70	$732.60	$739.64	100	$73260.00	$73964.00
Shear Tab	2	$33.50	$116.75	$48.65	$132.70	$166.50	$168.10	100	$16650.00	$16810.00
	Cost Differential								$56610.00	$57154.00

Reviewing the overall costs associated with the different block-outs required, it is apparent that field fireproofing costs add up quickly. The block-outs associated with clip angles tend to be larger and require more field applications than shear tabs. Even on a project where beam-to-column web connections need to be clip angles to keep the welds and materials down, changing the simply loaded beam-to-beam connections to shear tabs presents a large cost savings in fireproofing.

Comparison of Field Erection

When steel is erected in the field, it can be a time consuming process. Each piece has to be lifted into place, held steady for the connections to be made stable, and then disconnected from the lifting equipment. This process can become more time consuming when connections share bolts, as is the case with shear connections into column webs and beams. When beams share an end connection, one beam has to be held in place with a crane or other overhead lifting system while the bolts are removed. Then the second beam must be swung into place with a secondary lifting system while the bolts are being re-installed to make the connection stable. It takes double the people and double the coordination to make this happen. This increases not only the time required for erection but the safety risks to the people involved.

SSPC utilized at these connection locations can aid in reducing erection time and safety risks during steel installation. The shear tabs are welded to the columns or beams in the shop so they are already in place when the members are ready to be connected. In situations where beams share a connection end at a column or girder web, they each have their own shear tab. This removes the need for the secondary lifting system and bolt removal to get both beams installed at this shared location making the process more efficient and safe.

Chapter 5 - Summary

In summary, there is not a single perfect way to configure simple shear connections on an open structure. Utilizing connections that may save money may make the engineering and fabrication more difficult. Working to make things more safe in the field at the construction site may increase cost. Simplifying the engineering does not necessarily make the fabrication smooth or more efficient. It is important to look at all of the perspectives presented in this report and make an educated decision on the best approach to the connection design for each structure. There could be situations where more than one approach, or a combination of different connection types, may end up being the most beneficial design.

References

Sherman, D.R., & Ghorbanpoor, A. (2002). Design of Extended Shear Tabs

Jones, Clarence – Professional Engineer at Burns & McDonnell

Sardelli, R. – Sales representative at Markle Manufacturing

Shroeder, Sonny – Sales representative at Kreco Steel

Burns & McDonnell Estimating Group

Appendix A - Notations and Abbreviations

- Ds – Dead load of the structure or self-weight
- De – Dead load of vessels or equipment when empty or shipped
- Do – Dead load of vessels or equipment when full and operating
- Dt – Dead load of vessels or equipment when running their testing cases
- L – Live load of platforms or structures
- S – Snow load of platforms or structures
- Ff – Friction load applied at pipe or equipment supports generated by the operating stresses
- Af – Force applied to pipe supports when the pipe is anchored down to the structure
- W(X)/W(Z) – Wind load applied on the structure and equipment from any direction
- Eo(X)/Eo(Z) – Seismic forces applied to the structure and equipment based on the operating dead loads