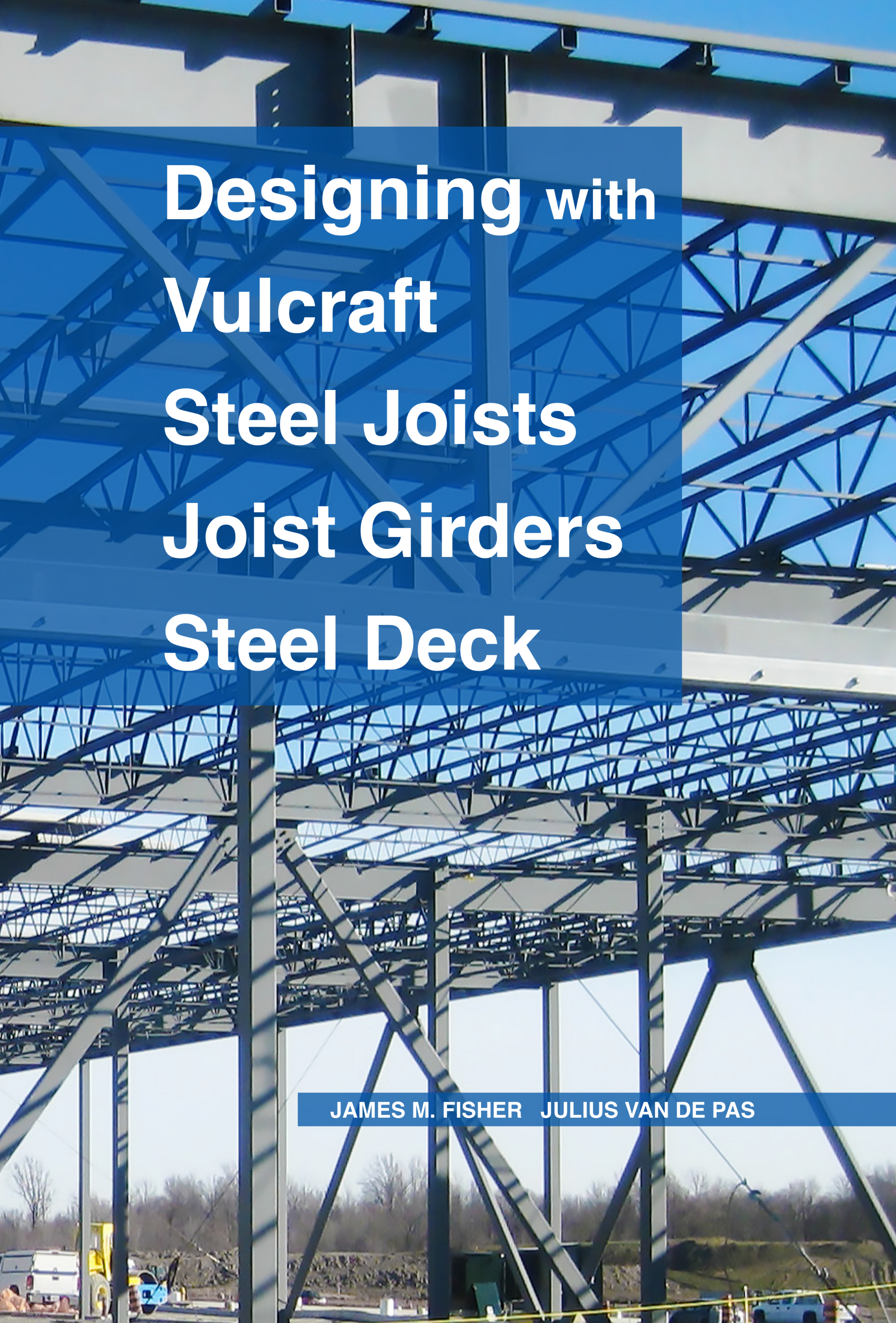


# Designing with Vulcraft Steel Joists Joist Girders Steel Deck

JAMES M. FISHER JULIUS VAN DE PAS

Vulcraft, a division of Nucor Corporation, has provided this book for use by engineers and architects in designing and using Vulcraft steel joists, Joist Girders and deck. It includes all products available at the time of printing. Vulcraft reserves the right to change, revise or withdraw any products or procedures without notice

The information presented in this publication has been prepared in accordance with recognized engineering principles and is for general information only. While it is believed to be accurate, this information should not be used or relied upon for any specific application without competent professional examination and verification of its accuracy, suitability and applicability by a competent licensed structural engineer, architect or other licensed professional. Publication of the material contained herein is not intended as a representation or warranty on the part of Nucor Corporation, or of any other person named. General use of this information will not result in infringement of any patent(s). Anyone making use of this information assumes all liability arising from such use.

www.vulcraft.com

NUCOR®
VULCRAFT/VERCO

2nd Printing 2024

## ACKNOWLEDGEMENT

This book was produced under the guidance of Russ Balvin, Marlon Broekemeier, Bruce Brothersen, Wes Frampton, Dave Henley, Scott Russell and John Whiteman. A special thanks to Benton Cooper of Vulcraft for his assistance in drawing the figures and for getting the text in final form for publication. The authors wish to thank all the Vulcraft personnel involved for their many suggestions and comments.



Authors:

James M. Fisher, Ph.D., P.E., DIST.M.ASCE

Julius P. Van de Pas, P.E., S.E.

## TABLE OF CONTENTS

# Chapter 1

# Introduction and General Information

## 1.1 PURPOSE

This book serves as a manual to provide a better understanding for specifying professionals on proper use and employment of steel deck, steel joists and Joist Girders. It is not a book which describes how these components are designed by Vulcraft. While steel deck, steel joists and Joist Girders have been in use for over 90 years, they are used in applications of greater complexity than initially contemplated. Their potential for innovative use has not yet been fully exploited. The book covers the use of steel deck, steel joists and Joist Girders so that their advantages are best employed and the process of using them is straight forward and efficient.

When reference is made to AISC Specification equations (AISC, 2016d), or to equations in the AISC 15th Edition Manual (AISC, 2017), the reference is designated as AISC Eq. or AISC Manual Eq. with the equation denoted within parenthesis, e.g. AISC Eq. (D2-1). In a few cases reference is made to the 2010 AISC Specification (AISC, 2010) and is so noted. AISI Specifications (AISI, 2016a) are similarly noted. Other equations pertinent to this text are designated only by an equation number.

## 1.2 HISTORICAL DEVELOPMENT

The development of joists and Joist Girders begins with the development of the steel truss which dates from the mid-nineteenth century. Open web joists are trusses; however, their design is per the Steel Joist Institute (SJI) “Standard Specification for K-Series, LH-Series, DLH-Series Open Web Steel Joists and for Joist Girders” SJI 100-2015, (SJI, 2015b). This contrasts with structural steel trusses that are fabricated per AISC standards. In the beginning of the twentieth century steel joists were produced to individual manufacturer’s patents and standards. These individual producers were unified under a common design standard when the SJI was formed in 1928. The establishment of the Standard Specification for steel joists allowed building designers to specify rather than design a structural component of the building frame. This Standard Specification is included in the Vulcraft “Steel Joist & Joist Girder Systems Manual, hereafter referred to as the Vulcraft Manual, (Vulcraft, 2017c). The Vulcraft Manual should also be consulted for all cases where references are made to the “SJI catalog.”(SJI, 2017b). The acceptance of the Standard Specification by building codes and building officials, allows the use of steel joists in buildings without the need to reconfirm by engineering design the sizes, materials, and welds used in joists conforming to standard designations for given loads and spans. Over the years each Standard Specification has had accompanying load tables which provides allowable uniform per foot loads for varying spans for each of several standard designations. This remains the basic format of the load tables to date, except for the addition of tables for KCS joists, top chord extensions, joist substitutes, outriggers and Load/Load joists.

While the standard load tables have always presented allowable strengths as uniform loads only the application of load in the completed project rarely met this requirement to the letter. For many years designers have used various strategies to account for both concentrated and non-uniform loads. The principal method being used was to cover the actual shear and moment diagrams with the shear and moment diagram of an appropriate standard designation joist. This method was generally thought workable, although it was technically incomplete since in some instances there were high localized top chord loadings and force reversals in some web members.

Joist manufacturers have also recognized that savings could be realized by designing joists to the exact uniform load requirements. This saving results from providing joists with properties which fall between the sizes and weights of joists conforming to the SJI standard designations.

## 1.3 CURRENT USAGE

At present, the usage of steel deck, steel joists and Joist Girders has already expanded beyond the elementary use contemplated originally by the Steel Deck Institute (SDI) and the Steel Joist Institute (SJI), (See list of SDI reference documents in Section 1.8) and the SJI "Standard Specification for K-Series, LH-Series, DLH-Series Open Web Steel Joists and for Joist Girders" SJI 100-2015, (SJI, 2015a), hereafter referred to as the SJI Specifications. In addition to being simple span members carrying uniform loads, these components are frequently used in continuous and statically indeterminate systems requiring greater sophistication in their specification and design. These systems require the use of rigid connections and in many cases the system resists lateral as well as gravity loads. Steel deck diaphragms are also employed in the lateral load resisting system.

The usage of steel deck, steel joists and Joist Girders includes both single and multistory structures for both floors and roofs. Steel joists have also been employed in curtainwall systems as girts. These components are applicable over a broad range of building types such as: warehouses, industrial plants, offices, commercial shops/malls, academic facilities, civic/institutional structures and large clear span structures such as fieldhouses and convention centers.

### Standard joists

K-Series, LH-Series and DLH-Series joists can be referred to as "Standard Joists." The SJI Specification covers the design, manufacturer, application and erection stability/handling of standard joists. The SJI Specifications are adopted by the International Building Code (IBC, 2015), hereafter referred to as the IBC. Not only is the specification adopted, but also adopted are the ASD and LRFD Load Tables for Standard Joists along with KCS-Series joists, joist Top Chord Extensions, Extended Ends, Joist Substitutes and Outriggers. The major advantage of having IBC adopt the SJI Specifications is that when specifying standard joists calculations/design calculations are not required to be submitted to the authority having jurisdiction for any given project, the specifying professional need only indicate the proper standard designation for the joist.

### KCS joists:

KCS joists, ASD and LRFD load tables are published in the SJI Catalog (SJI, 2017b). The CS in the KCS designation stands for constant shear. These tables provide moment strength and shear strength for 40 different designations in depths ranging from 10 inches to 30 inches. If KCS joists can be selected for a given arrangement of loads, the need for a custom design is eliminated with the exception that concentrated loads must still be specified. Concentrated loads must be located at panel points or else field installed webs must be added.

### Top chord extensions (TCX) and extended ends:

ASD load tables were first introduced in the SJI Catalog in 1989, and LRFD load tables followed in 2000. Top chord extensions are referred to as S "Type" and have only the two top chord angles extended. Top chord extensions are the most economical solution to support overhanging loads. If the S "Type" does not have sufficient strength, then the R "Type" should be specified. The R "Type" (extended ends) have the standard bearing (2.5 in.) end bearing seat extended. If the 2.5 in. extension does not have sufficient strength the specifying professional can specify a deeper seat be used so that the extension is a deeper member.

### Joist substitutes and outriggers:

Joist substitutes are 2.5 in. deep members commonly used for uniformly loaded simple spans up to 10 feet in length. For members less than 8', joist substitutes should be used instead of open web joists. Vulcraft can supply joist substitutes in a variety of profiles so long as the member is 2.5 inches in depth. The load table in the SJI Catalog were revised in 2010. Joist substitutes can also be specified with cantilever outriggers up to 6 feet in length. The Vulcraft Manual contains load data for joist substitutes and out riggers for ASD and LRFD.

### Load/Load Joists:

Weight tables for Load/Load joists were first introduced in the SJI Catalog in 2010. The SJI Catalog contains Standard Weight Tables for Load/Load LH-Series Joists. The weight tables are intended to provide the specifying professional with a tool to assist in the preliminary design and as an estimate of weight for joists used in floors and roofs that require high capacity loading. Many efficient floor framing layouts require joists for which the uniform load per foot exceeds the 550 plf limit used for K-Series joist tables or where the uniform load exceeds the tabulated safe loads for LH-Series joists. The tables apply only to joists with parallel chords.

The reader is referred to the Vulcraft Manual for additional information on KCS-joists, Joist Extensions, Joist substitutes and outriggers, and Load/Load joists.

### Virtual Joists and Joist Girders:

Virtual Joists and Virtual Joist Girders are a set of pseudo section properties, generated by SJI as tables, and are based on commonly available joist chord pairs. The major axis moment of inertial ($I_{xx}$) is based directly on a pair of double angle joist chords and pre-adjusted for approximate deflection contribution from web strain. Virtual Joists and Joist Girders allows joists and Joist Girders to be checked for flexure, shear, axial and combined forces by treating them as doubly symmetric I-shaped beam sections. They are currently inserted into several stiffness based structural analysis programs.

Virtual joist and Virtual Joist Girder tables allows joist member stiffness to be accounted for in the overall building model. Once the building model is completed, standard SJI joist and Joist Girder designations can be called out on the plans.

For additional information on the use of Virtual Joists and Joist Girders three Webinars are offered on the SJI Website, www.steeljoist.org. The titles are “Using the SJI Virtual Joist and Joist Girders in RISA,” “Using the SJI Virtual Joist and Joist Girders in RAM” and “Using the SJI Virtual Joist and Joist Girders in SCIA.”

### Building Information Modeling (BIM)

In addition to many other design tools, Vulcraft offers BIM tools for Revit and Tekla and has assisted in the development of joist components that are packaged with SDS/2.

Vulcraft’s NuBIM for Revit add-in allows users to specify and model all parallel chord joists and Joist Girders available from Vulcraft as well as many common special profile joists, Ecospan® and composite joists. Users can apply a variety of common loading conditions to all joists, as well as create schedules and diagrams. All Vulcraft and Verco deck profiles can be added to standard Revit floor and roof components through the add-in. When the project is complete, a file can be exported containing all information related to Vulcraft products, which can be sent to Vulcraft to aid in the estimating and detailing process.

With the NuBIM for Tekla Plug-In, the design professional can build and manage projects more effectively within Tekla Structures. Vulcraft’s Joist Plug-In for Tekla Structures enables the design professional to specify Vulcraft joists during the creation of the building model. The model can be exported containing Vulcraft joist and Joist Girder information and then be used by Vulcraft to aid in the estimating and detailing process.

Due to the complexity of the construction industry, the various states of deliverables through the construction process and the product itself, BIM is defined by the industry standard terminology of “Level of Development” (LOD). Within the construction industry the models can change both during the process stages and by individual construction products from 100 to 500. A LOD model 100 is represented with a symbol or generic representation whereas a LOD model 500 is a field verified representation in terms of size, shape, location, quantity and orientation. The Vulcraft BIM model falls within LOD of 300 to 350, or Manufacturer/Trade Level of Development. The Vulcraft 3D BIM model include chord and web sizes, webbing location, bearing seat slopes, depths and holes/slots, with bridging sizes and locations all included. The Vulcraft BIM model can coordinate with other trades and "clash detection" functions can be performed. In order to provide a realistic model, the appropriate information must be provided to Vulcraft by a third party, most often from a complete set of construction documents. The information for a complete model must include all loading, serviceability, geometric and connection requirements. With the appropriate information, Vulcraft designs a propriety product that is converted to a 3D model and it can be inserted into the overall building model.

A common way to set BIM expectations for the construction trades is through a BIM execution plan (BIMXP). It is strongly recommended that whenever a BIM is required, clear instructions and expectations be noted in the contract documents so that they can be properly included in the estimate. Vulcraft has a common BIMXP on the Vulcraft website, which Vulcraft suggests be used for such coordination.

For additional information related to BIM see the BIM tab on VULCRAFT.COM.

## 1.4 CUSTOM DESIGNS

The need to design for concentrated and non-uniform loads and the desire to provide designs for actual loads has prompted joist manufacturers to expand their engineering design capability and this in turn, along with automated controls, has fostered an environment of greater and greater variety of custom products. Currently the following products are offered. These products are:

1. Special designs for which the specifier indicates on the contract drawings the exact loading for which the joist is to be designed
2. Special profile joists: Bow string (curved top chord), scissors and offset ridges on double pitched joists are offered. These profiles are a significant departure from what would normally be thought of as joists but are a natural extension of the custom design process within the standards of joist construction

## 1.5 SHOP WELDING OF JOISTS AND JOIST GIRDERS

The SJI Technical Digest #8 states, "The SJI approach to welding is a performance-based specification that has been developed over decades with a specific focus on the design and fabrication procedures intrinsic to steel joists and Joist Girders. The essence of the SJI Specification's modification of several AWS Structural Welding Code D1.1(AWS, 2015) acceptance criteria is that it is possible a weld, or portion of a weld, that neither contributes to the overall structural performance, nor takes away from the overall structural performance could be rejected by the SJI Specification is based on welding in a controlled environment, in a repeatable and predictable manner, validated by full-scale physical testing. Unlike AWS acceptance criteria, a weld or portion of a weld that does not meet all the SJI acceptance criteria does not necessarily require removal or repair. Instead two questions are raised:

1. Is there an adequate length of defect free weld to meet the required strength (length)?
2. Is the defect detrimental to the structural integrity of the member being joined?

As an example, suppose a fillet weld specified to be 3 inches long was made 5 inches long, but the last inch of length was undersized. This would meet SJI acceptance criteria but not that of AWS D1.1. As another example, excessive porosity or an apparent lack of fusion are cause to reject and discount the length of weld affected by these defects. These types of defects affect only the integrity of the weld and not the connected materials, and hence, may not need correction if an adequate length of defect free weld is observed. This is not to suggest that these types of defects are not a cause for concern, with regard to the ability of the welding operator. Conversely, a defect such as excessive undercut is cause for rejection and repair, as it has affected the strength and capacity of the undercut member. The application of the acceptance criteria in a performance-based manner is the key difference between the SJI and AWS welding criteria."

The above citation outlines a few differences between AWS and SJI welding, there are, however, many similarities:

- Welding consumables for SJI products should meet an appropriate AWS standard
- SJI recognizes the AWS pre-qualified joints
- SJI requires a qualification process for welding procedures that are not AWS pre-qualified
- SJI requires Weld Procedure Specifications (WPS's) with the same basic information required as AWS
- For the qualification of individual welders and welding operators, SJI requires a program of testing and performance verification. SJI recognizes qualification to the AWS D1.1 and/or AWS D1.3(AWS, 2017) standards as the means to accomplish the qualification

In all cases, a consumable, procedure, operator, or acceptance criteria that meets the AWS Specification will also meet the SJI Specification.

For an in-depth treatise on SJI welding the reader should consult Technical Digest #8.

## 1.6 CODES AND SPECIFICATIONS

The providing of steel deck, steel joists and Joist Girders is done in a legal environment in which each individual project is permitted to be constructed by the issuance of a building permit. The issuance of such permit requires that the construction conform to minimum requirements which are set forth by statute and include the Building Code. Many jurisdictions have assembled their own unique requirements. However, the International Building Code, is the predominate code in the US. When the IBC is not adopted, in a given location, specifying professionals often rely upon ASCE/SEI 7, "Minimum Design Loads for Buildings and Other Structures" (ASCE/SEI, 2016), hereafter referred to as ASCE 7.

The Chapter 22 Section 2207 in, the IBC contains the requirements for the design of steel joists, including calculation requirements, drawings and certification.

Section 2207.1 General indicates that, "The design, manufacture and use of open web steel joists and [Joist Girders] shall be in accordance with the Steel Joist Institute 'Standard Specification for Steel Joists, K-Series, LH-Series, DLH-Series and Joist Girders.'"

Section 2207.1.1 Seismic design, indicates that, "Where required, the seismic design of buildings shall be in accordance with the additional provisions of Section 2205.2 or 2211.6."

In addition to the requirements in the IBC, the design, manufacture and use of open web steel joists shall be in accordance with the SJI Standard Specifications.

It is the adoption of the SJI Standard specifications that allows building designers to specify joists using standard designations. When special loads or special joists are required, joists must be custom designed following the standards cited. The design of standard and custom joists is the responsibility of the joist manufacturer. It is the responsibility of the building designer to establish the loadings to which the design must conform. This involves the judgment of the building designer in interpreting the requirements of the building code, the building user's requirements, and loads and forces from all sources on the joist and Joist Girders for the completed structure. The building designer expresses his judgment through the identification of joist and Joist Girders by standard designation or the presentation of loading diagrams, schedules or notation.

The AISI specification is applicable to the design of steel decks and references many of the SDI publications.

The Steel Deck Institute provides specifications, a Code of Standard Practice (SDI, 2017a) and load tables for common deck profiles to which SDI gives standard designations.

## 1.7 OTHER SPECIFICATIONS

Steel deck, steel joists and Joist Girders are frequently used in combination with structural steel.

Metal building systems conform to the "Low Rise Building Systems Manual" (MBMA, 2012) published by the Metal Building Manufacturers Association. This Manual may have applicability where joists are used in conjunction with a metal building system.

Where steel deck, steel joists and Joist Girders are used in conjunction with other materials, the following codes may apply:

1. Concrete: ACI 318, "Building Code Requirements for Reinforced Concrete and Commentary" (ACI, 2014)
2. Masonry: "Building Code Requirements and Specification for Masonry Structures and Related Commentaries" (ACI, 2011)
3. Masonry: "Reinforced Masonry Engineering Handbook, 8th Edition, (MIA, 2017), Brick Institute of America, Torrance, CA.
4. Wood: "APA Construction Guide" (APA, 2016), American Plywood Association, Tacoma, Washington

## 1.8 REFERENCE STANDARDS

Other reference standards are important and useful in designing structures employing steel deck, steel joists and Joist Girders. First, ASCE 7 "Minimum Design Loads for Buildings and Other Structures," is very helpful in understanding loads on structures because it goes into greater detail than many codes. It covers dead loads, live loads, wind loads, snow loads, (as distinguished from roof live loads), rain loads, earthquake loads and load combinations.

The Steel Joist Institute has published a series of Technical Digests covering a range of significant topics, which are:

TECHNICAL DIGEST 3 "Structural Design of Steel Joist Roofs to Resist Ponding Loads" (SJI, 2018a)

TECHNICAL DIGEST 5 "Vibration of Steel Joist Concrete Slab Floors" (SJI, 2015b)

TECHNICAL DIGEST 6 "Structural Design of Steel Joist Roofs to Resist Uplift Loads" (SJI, 2012)

TECHNICAL DIGEST 8 "Welding of Open Web Steel Joists" (SJI, 2008a)

TECHNICAL DIGEST 9 "Handling and Erection of Steel Joists and Joist Girders" (SJI, 2008b)

TECHNICAL DIGEST 10 "Design of Fire-Resistive Assemblies with Steel Joists (SJI, 2003)

TECHNICAL DIGEST 11 "Design of Lateral Load Resisting Frames Using Steel Joists and Joist Girders (SJI, 2007a)

TECHNICAL DIGEST 12 "Evaluation and Modification of Open Web Steel Joists and Joist Girders" (SJI, 2007b)

"90 Year Open Web Steel Joist Construction (1928-2018)" (SJI, 2018)

In addition to the Steel Deck Institute's "Design Standard for Steel Roof Deck" SDI (2017d), the SDI has several manuals relative to steel deck design:

"Diaphragm Design Manual (DDM04)," (SDI, 2015),

"Floor Deck Design Manual (FDDM)," (SDI, 2017e)

"Manual of Construction with Steel Deck (MOC3)," (SDI, 2017f)

"Roof Deck Design Manual (RDDM)," (SDI, 2017g)

"Steel Deck on Cold-Formed Steel Framing Design Manual (SDCFSFDM)," (SDI 2018)

Standard Practice Details - No. SPD2 (SDI, 2001)

Additional important references:

FM Global publishes an annual "Approval Guide" (FM Global, 2018) and a series of "Loss Prevention Data Sheets" (FM Global, various dates). The purpose of these documents is to raise the quality and integrity of building construction to limit insurance losses and improve conditions for underwriting insurance. These standards are frequently stricter than would be required by the Building Code and their applicability should be agreed to after consultation with the building owner.

Presented in the International Association of Plumbing and Mechanical Officials (IAPMO), are product descriptions and tabulated information which show conformity to the applicable standards.

Underwriters Laboratories of Northbrook, Illinois publishes the "Fire Resistance Directory" (UL, 2019) which contains descriptions and illustrations of numerous fire rated assemblies for floors and roofs, etc. These rated assemblies are required by code depending on the occupancy, size, height and construction class of a given building and are thus important in determining the required construction of floors and roofs.

## 1.9 SYNOPSIS OF CHAPTERS

*Chapter 2*: Discussion of roofing types, decking types, roof loadings, arrangement of framing and bays and serviceability criteria for roofs.

*Chapter 3*: Discussion of floor decking types, floor loadings, arrangement of framing and bays and serviceability considerations.

*Chapter 4*: Discussion of lateral load resisting systems, roof and floor diaphragms, horizontal bracing, braced frames, rigid frames, selection of appropriate systems.

*Chapter 5*: Presentation of solutions to special situations: hanging loads, headers and openings, roof top units, joist reinforcement, ponding, vibration, fire resistance, etc.

*Chapter 6*: Requirements and procedures for specifying steel joists, Joist Girders, and steel deck.

*Chapter 7*: Design of connections, strength and use of standard end connections, axial forces, reinforced seats, rollover of joist seats due to lateral loads, connection details and examples of designs accounting for forces on joist and Joist Girder ends.

*Chapter 8*: Discussion of requirements for construction documents, drawings and specifications, requirements for preliminary designs and budget/concept drawings, responsibilities of parties in construction, discussion of codes of standard practice and division of buyer/seller activities.

## 1.10 CONCLUSION

As stated initially it is the intention of this book is to give the building designer a complete and usable understanding of the design process where steel joists, Joist Girders and steel deck are used.

# Chapter 2

# Roofs

## 2.1 INTRODUCTION

This chapter deals with roofs and roof framing. It presents a discussion of six topic areas:

1. Roofing types
2. Deck types
3. Roof loading
4. Serviceability considerations
5. Framing considerations
6. Connections

## 2.2 ROOFING TYPES

Roofing types are classified by the roofing industry into two broad categories: low slope and steep slope. Low slope roofs are commonly characterized as flat but are generally pitched to gutters or internal drains. Low slope roofs are identified by their materials and/or installation procedures. They are:

1. Built-up roofing
2. Single-ply roofing
3. Liquid applied roofing
4. Structural metal roofs

Built-up roofing is composed of layers of roofing felt which are adhered together with alternating layers of bitumen. The roofing felt consists of a mat of organic or inorganic fiber which is saturated, impregnated and/or coated with asphalt. The inter-ply bitumen can be various kinds of asphalt or coal tar depending on the conditions of use. Built-up roofs are top surfaced with either aggregate, mineral surfaced cap sheet or other reflective or protective surfaces. They can also be finished with a top coating of asphalt. Built-up roofs are installed on a foundation of insulation which is attached to the decking. Built-up roofs by virtue of their continuous attachment and their limited elastic properties require the strictest attention to roof deck stiffness and control of lateral expansion and contraction (see Section 2.5).

Single-ply roofing is self-descriptive. It consists of a single sheet membrane which is either laid loose and ballasted, semi-attached (at discreet locations) or continuously adhered to the substrate which is insulation over the deck. These membranes are made from various materials. The most common are:

1. Ethylene Propylene
2. Diene Monomer (EPDM)

3. Poly Vinyl Chloride (PVC)
4. Polymer Modified Bitumen

The membranes are delivered to the site in rolls which are seamed together in the field to form continuous roofing. Over the years the rolls have become very wide and are often delivered to the job site in 12 foot and greater widths. Attachment to the deck and joists are only made at the seams which can cause severe overstresses in the deck and joist under wind uplift loads. Fisher, J.M. and Sputo, T. (2017) address this situation in their article "Are your roof members overstressed?"

When these roofs are adhered to the substrate, the limitations on an area are similar to those of built-up roofs but the requirements for deck stiffness can be somewhat relaxed. When the membranes are loose laid and ballasted, the requirements for both roofing area and deck stiffness can be much less restrictive than the adhered membranes. Many localities no longer permit the use of ballasted systems due to the ballast becoming projectiles during severe wind events.

Liquid applied membranes consist of a foamed in place insulation, usually urethane, which is covered by a protection and water barrier, usually a silicone-based product. Such membranes are highly individualized, so reference to specific manufacturer's literature is advised.

Structural metal roofs are divided into two main categories: standing seam and through fastened. In both cases, the roof is attached to the supporting joists or purlins, which are usually spaced at five feet, and spans between them to support the roof loads. A standing seam roof is formed from long narrow panels which are joined together by an inter-locking or seamed high rib joint running parallel to the drainage direction. The roof is attached to the joists with a sliding clip which permits longitudinal expansion and contraction of the panels due to thermal effects. Through fastened roofs, as the name implies, are fastened to the support with screws. This direct attachment limits the range of movement in response to thermal load.

Except for structural metal roofs, most roofs rely on deck for support.

## 2.3 DECK TYPES

Decking spans between joists or purlins and supports the weight of the roofing, insulation and the roof live and/or snow loads. Decks are made from steel, concrete or wood.

### Steel Roof Decks

Most of the steel deck is manufactured from steel conforming to ASTM A1008/A1008M, Structural Sheet for uncoated or painted deck or from ASTM A653/A653M, Structural Sheet for galvanized deck. The specifying professional should choose one finish or the other. However, both types of finish may be used on a given project, in which case the designer must indicate on the plans and project.

Vulcraft steel deck is manufactured from sheet that is obtained from the steel mill with either a galvanized or uncoated (black) finish. A shop coat of primer can be applied to either or both sides of the sheet. If primer is applied in the shop, a coating of iron phosphate is applied (phosphatized) before the primer is applied to enhance adhesion of the primer. Shop primer is only intended to protect the steel for a short period of ordinary atmospheric conditions. The Steel Deck Institute (SDI) recommends the field painting of shop coated deck especially where the deck is exposed to the weather. SDI also recommends the use of galvanized deck (G60 or G90) in corrosive or high moisture conditions. Selection of the steel deck finish is the responsibility of the Specifier.

Steel deck is supplied as galvanized or shop prime coated. The shop prime coat is only intended to protect the steel for a short period in ordinary atmospheric conditions. The Steel Deck Institute recommends the field painting of shop coated deck especially where the deck is exposed. SDI recommends the use of galvanized deck (G60 or G90) in corrosive or high moisture conditions. Selection of the steel deck finish is the responsibility of the Specifier.

Roof decks are commonly 1-1/2 in. deep but deeper units are also available. The Steel Deck Institute identifies the standard profile for 3-inch deck as DR. Vulcraft designates this deck as 3N deck (Vulcraft, 2018). The SDI has also identified three standard profiles for 1-1/2 in. steel deck, which are narrow rib, intermediate rib and wide rib, and has published load tables for each profile for gages varying from 22 to16 gage. These three profiles, NR, IR and WR, correspond to Vulcraft's designations of A, F and B. A comparison of weights for each profile in various gages shows that the weight to strength ratio for each profile is most favorable for wide rib deck and least favorable for narrow rib deck.

The following steel roof deck types are manufactured by Vulcraft for use in roof construction:

1. 1.5B/1.5BI, 1.5BA/1.5BIA, 3N/3NI, 3NA/3NAI, 3NL-32 and 3NI-32
2. PunchLok® II decks: PLN, PLB and PLN-32
3. 2.0D, 2DA and 3.5D, 3.56DA Dovetail Decks
4. Cellular Roof Decks: 1.5BP and 3NP
5. Cellular Acoustical Roof Decks: 1.5BPA and 3NPA

For further information on roof decks consult the SDI Standard for Steel Roof Deck (SDI, 2017d), Vulcraft's manuals: "Steel Roof & Floor Deck," "PunchLok® II Roof Deck Weld and Screw Support Connections," "Dovetail Roof Deck Welded Support Connections" (Vulcraft, 2018), (Vulcraft, 2016b) and (Vulcraft, 2017a) and online at www.vulcraft.com.

The Dovetail decks are ideally suited for sound control when infilled with 2 in. Poly-Iso or 2 in. fiberglass insulation. The Punchlok II™ deck is ideally suited when high diaphragm shears are required.

In general, the deck selection, which results in the least weight per square foot, is the most appropriate. However, consideration must also be given to the flute width because the insulation used must span the flute. In the northern areas of the United States, high roof loads in combination with thick insulation generally makes the wide rib (B) profile predominant. Where very thin insulation is used, narrow rib deck may be required although this is not a common profile and is the heaviest, most expensive profile with the lowest strength to weight ratio, thus it is almost never used. In general, the lightest weight per square foot deck consistent with insulation thickness and span should be used.

Steel deck must be designed to conform to the building code for maximum loads and serviceability criteria. The reader should contact Vulcraft/Verco for assistance in making deck calculations.

In addition to the load, span and thickness relations established by load tables, there are other considerations in the selection of a profile and gage for a given load and span. First, the Steel Deck Institute limits deflection due to the uniformly distributed live loads to span over 240. Secondly, the Steel Deck Institute has published a table of maximum recommended spans for construction and maintenance loads (SDI, 2017f), reprinted here as Table 2.3.1. The reader is cautioned about using the double and triple construction spans with the deep rib decks because individual sheet weights are so great that the deck erectors have great difficulty in handling them.

Shown in Table 2.3.2 are recommended maximum construction center to center spans for commonly used Vulcraft/Verco decks. As noted in the footnote, the spans are calculated using 50 ksi steel for 1.5B, 1.5BI, 1.5PLB, 3NL-32, 3NI-32 and 3PLN-32 decks and 40 ksi steel for Dovetail decks, as compared to the current SDI table which uses 33 ksi steel for the calculated values. The SDI "RD-2017 Standard for Steel Roof Deck" (SDI, 2017d) no longer requires a deflection criterion relative to the calculated values for maximum construction spans, thus in many cases for Vulcraft/Verco decks, deflection criteria for vertical loading or wind uplift will control the maximum span length. See Table 2.3.3, "Simplified FM Approved Spans (c-c)." Since the maximum rolling length of Vulcraft/Verco deck is 42 feet, deck center to center span lengths exceeding 21 feet cannot be used for double span conditions. This requirement is also applicable to the SDI Table 2.3.1.

| Recommended Maximum Spans for Constructions and Maintenance Loads Standard 1 1/2" and 3" Roof Deck | | | | | |
|---|---|---|---|---|---|
| Deck Type | | Span Condition | Gage Number | ASD Span (ft-in) | ASD Cantilever Span (ft-in) |
| NARROW RIB | NR22 | Single | 22 | 2'-11" | 0'-10" |
| | NR20 | | 20 | 3'-08" | 1'-00" |
| | NR18 | | 18 | 5'-00" | 1'-03" |
| | NR16 | | 16 | 6'-05" | 1'-07" |
| | NR22 | Double or Triple | 22 | 3'-07" | |
| | NR20 | | 20 | 4'-06" | |
| | NR18 | | 18 | 6'-02" | |
| | NR16 | | 16 | 7'-11" | |
| INTERMEDIATE RIB | IR22 | Single | 22 | 3'-05" | 0'-11" |
| | IR20 | | 20 | 4'-03" | 1'-01" |
| | IR18 | | 18 | 5'-10" | 1'-06" |
| | IR16 | | 16 | 7'-06" | 1'-10" |
| | IR22 | Double or Triple | 22 | 4'-03" | |
| | IR20 | | 20 | 5'-03" | |
| | IR18 | | 18 | 7'-02" | |
| | IR16 | | 16 | 9'-03" | |
| WIDE RIB | WR22 | Single | 22 | 5'-08" | 1'-06" |
| | WR20 | | 20 | 7'-00" | 1'-10" |
| | WR18 | | 18 | 9'-06" | 2'-05" |
| | WR16 | | 16 | 12'-02" | 3'-00" |
| | WR22 | Double or Triple | 22 | 6'-11" | |
| | WR20 | | 20 | 8'-07" | |
| | WR18 | | 18 | 11'-08" | |
| | WR16 | | 16 | 15'-00" | |
| DEEP RIB | DR22 | Single | 22 | 11'-11" | 3'-04" |
| | DR20 | | 20 | 15'-04" | 4'-02" |
| | DR18 | | 18 | 21'-01' | 5'-07" |
| | DR16 | | 16 | 27'-05" | 7'-01" |
| | DR22 | Double or Triple | 22 | 14'-07" | |
| | DR20 | | 20 | 18'-11" | |
| | DR18 | | 18 | 26'-00" | |
| | DR16 | | 16 | 33'-09" | |

Spans shown are calculated using 33 ksi steel and Allowable Strength Design and considered to be conservative. Longer spans may be permittted by LRFD designs or for higher strength steels. Consult deck manufacturer for further guidance.
Refer to the deck manufacturer's catalogs or the SDI Floor Deck Design Manual (FDCM) for construction span table for floor deck.

Note: Center to center spans greater than 21 feet exceed rolling lengths for Vulcraft/Verco deck for the two-span condition.

**Table 2.3.1 Steel Deck Institute Recommended Spans**

**RECOMMENDED MAXIMUM CONSTRUCTION C. TO C. SPANS FOR COMMON VULCRAFT/VERCO DECKS**

| Deck Type | Span Condition | Gage Number | ASD C. to C. Span (ft.-in.) |
|---|---|---|---|
| 1.5B, 1.5BI or 1.5PLB | Double | 22 | 8'-3" |
| | | 20 | 10'-5" |
| | | 18 | 14'-0" |
| | | 16 | 17'-0" |
| 3NL & 3NI-32 or 3PLN-32 | Double | 22 | 15'-3" |
| | | 20 | 19'-3" |
| | | 18 | 23'-5"* |
| | | 16 | 26'-6"* |
| 2.0D Dovetail | Double | 22 | 9'-11" |
| | | 20 | 12'-3" |
| | | 18 | 15'-9" |
| | | 16 | 18'-11" |
| 3.5D Dovetail | Double | 20 | 21'-3"* |
| | | 18 | 24'-9"* |
| | | 16 | 28'-0"* |

Clear Spans are calculated using 50 ksi steel for 1.5B, 1.5BI, 1.5PLB, 3NL-32, 3NI-32 and 3PLN-32 decks and 40 ksi steel for Dovetail decks.
*Center to center spans exceed maximum rolling length of 21'for a two-span condition. Consult Vulcraft/Verco for further guidance.

FM Global in its Loss Prevention Guide (LPG)1-28 "Wind Loads to Roof Systems and Roof Deck Securement," gives a standard for attachment of insulation to steel deck. LPG 1-29 "Roof Deck Securement and Above Deck Roof Components," gives a standard for the required weight and distribution of ballast for roofs that are not adhered and the attachment of insulation to steel deck.

LPG 1-28 requires a sidelap fastener between supports for spans greater than 3 feet. This fastener prevents adjacent panels from deflecting differentially when a load exists at the edge of one panel but does not exist on the edge of the adjacent panel. The Steel Deck Institute requires that the side laps in cantilevers be fastened at twelve inches on center.

Steel decks are attached to supports by welding or by fasteners which can be power or pneumatically installed or with self-drilling self-tapping screws. The Steel Deck Institute in its "Standard for Steel Roof Deck," requires a maximum attachment spacing of 18 inches along supports. FM Global requires the use of 12 inch spacing as a maximum and this is more common. The attachment of roof deck must be sufficient to provide bracing to the joist top chord, to anchor the roof to prevent uplift, and in many cases to serve as a diaphragm to distribute lateral loads to vertical braced frames.

Diaphragm strength is a function of the deck profile, thickness and attachments, both to the supports and at sidelaps. Diaphragm strength tables have been developed by the Steel Deck Institute and are contained in the SDI "Diaphragm Design Manual DDM04 (SDI,2015)." Further information on steel deck diaphragms is presented in Chapter 4.

The reader is encouraged to go to the Vulcraft website, www.vulcraft.com/design-tools, for diaphragm strengths and stiffness coefficients tools that can be used in calculations of diaphragm deflection.

The website tools are divided into the following three categories:

1. Joist Design Tools
    - Joist Analysis Aid
    - Shear Diagram Assistant
    - SJI Design Tools
2. Roof Deck Design Tools
    - 2018 IBC Deck Diaphragm
    - Steel Deck Roving Load
    - 2015 Deck Diaphragm
3. Floor Deck Design Tools
    - Unshored Span Calculator
    - Deck-Slab Diaphragm Strength
    - Composite Deck-Slab Strength

### Concrete Deck

Concrete decks on steel joists are available in a wide variety of forms. They are:

- Insulating lightweight concrete
- Gypsum concrete
- Precast concrete slabs

Lightweight insulating concrete is cast on steel form deck or form boards. It has a density of 20 to 40 pounds per cubic foot and should not be confused with light weight structural concrete which has a density of 100 to 120 pounds per cubic foot. Light weight structural concrete is frequently used on steel deck in floor construction. It is rarely used in roof construction.

Lightweight insulating concrete is made using light weight aggregate such as vermiculite or perlite. Additional information on light weight insulating concrete can be found in American Concrete Institute Committee Report "ACI 523.1 Guide for Cast-in-Place Low Density Concrete"(ACI, 2006). Aggregates for such concrete are covered in ASTM Specification "C332 Standard Specification for Light Weight Aggregates for Insulating Concrete"(ASTM, 2017).

Roofs of insulating lightweight concrete rely on the substrate for the strength to support dead and roof live loads. The lightweight insulating concrete is only a fill which contributes to dead load. Steel decks which are used to support light weight concrete fill are form decks such as Vulcraft's CSV. Because of the high moisture content of insulating concrete, it is necessary to provide slot vented decks so that water and vapor can dissipate from both the top and bottom sides of the concrete. Vulcraft 0.6, 1.0 and 1.3 CVS decks have sidelap vents which are adequate for venting structural concrete. Vermiculite aggregate lightweight insulating concrete requires the use of CSV-Series deck which is slot vented in the bottom of the deck. The high moisture content and the need for permanence of the steel form indicate that steel forms used with insulating light weight concrete must be galvanized.

The attachment to supports of the steel form deck are as described in the section on steel roof deck, i.e. welded or mechanically fastened. The Steel Deck Institute has done research to establish diaphragm values for steel form decks supporting lightweight insulating concrete. SDI has established two construction types for decks with insulating lightweight fills. Type 1 consists of at least 2-1/2" of vermiculite aggregate concrete over the top of the steel deck. Type II is a built-up composite in which a board of at least two inches in thickness, made of expanded cellular polystyrene, is embedded in the light weight insulating concrete. It should be noted that most light weight insulating concrete is used as part of proprietary insulating systems and that manufacturers' literature should be consulted. Provided in Vulcraft's manual, "Steel Roof & Floor Deck" are diaphragm values for Type 1 and Type 2 fills on 0.6, 1.0 and 1.3 CVS decks.

Gypsum concrete decks are cast on gypsum form boards which span between metal bulb tees which span between joists. The usual spacing of bulb tees is 32 inches on center. The gypsum concrete is reinforced with galvanized reinforcing fabric which is draped in the cross section. The design of such systems involves the use of proprietary products and systems. Their manufacturers should be consulted for design criteria. Because gypsum concrete decks are not generally considered to have diaphragm capacities their use may require a separate roof bracing system, such as rod or strap bracing or perimeter in-plane trusses.

Precast concrete deck elements span between joists and are available from manufacturers in these configurations: channel slabs, hollow core slabs and solid tongue and groove edged planks. Manufacturer's load tables should be consulted for spans and load capacities. Precast slabs are used both topped and un-topped. Topping is used to create roof pitches, to increase insulating value or to create a reinforced concrete roof diaphragm. The slabs are attached to the joists using clips or by welding depending on the manufacturer's standard details. The attachment of the precast deck units may or may not have the strength to provide bracing of the top chord of the joist. Most precast concrete decks do not provide diaphragm capabilities so other bracing in the plane of the roof must be provided. The specifying professional shall define the type and spacing of attachment to all joists and Joist Girders.

### Wood Deck

Wood decks are available in the following general categories:

1. Plank and laminated wood decks
2. Plywood and OSB decks
3. Structural wood fiber decks

Plank and laminated wood decks are field assembled from long narrow wood pieces. They are frequently installed with tongue and groove edges. The length of individual pieces is either uniform to produce single or multi-span members or random which produces a multi-span condition the length of the roof. Random layup installations have plank end joints in the span and may be visually objectionable. Uniform layouts have end joints over the supports. Plank and laminated decks are used when an exposed wood surface is required in the building design, or when a nailable top surface is required. They are usually attached to joists by means of nailer strips which are bolted or screwed to the top chord providing both lateral bracing to the joist and a tie down to resist uplift loads. Plank and laminated wood decks are not normally used as diaphragms, as this has not been adequately studied by the industry. One inch and two-inch nominal decks have limited diaphragm strength. The strength is limited to the force couple which can form between two face nails driven through each plank into each support. Generally, plank and laminated wood decks must have an alternate in plane bracing systems to transfer lateral loads. Such systems either cover the deck with a plywood or diagonal plank diaphragm or provide diagonal or strap bracing or perimeter bracing trusses.

Plywood and OSB panels are marked with an Identification Index which gives a maximum span over which a given panel grade and thickness can be used for roofs and floors. For example, 48/24 indicates a maximum recommended support spacing of 48 inches in roof construction and 24 inches in floor construction. The loadings associated with these index numbers are 35 psf roof live load and 100 psf floor live load. In both cases live load is intended to mean total superimposed load. In many cases the 35 psf roof superimposed load may not be adequate, in which event reference should be made to load tables published by the American Plywood Association (APA, 2016). Plywood and OSB is readily used as a diaphragm and diaphragm values for various panel layouts, panel thicknesses and nailing patterns have been tabulated. Values are published by the American Plywood Association and others are adopted by the model building codes. The specifying professional must denote the edge nail spacing, panel orientation with sub-purlin size and nailing requirements.

Structural Wood Fiber Deck is a panel product composed of long wood fibers which are bonded with a cementitious matrix. These panels are either designed to span from joist to joist or are used with bulb tees in a two-way system. These panels have unique properties and their manufacturer's literature should be consulted for load and span information. Depending on the panel and its manufacturer, these decks may or may not be used as a diaphragm.

## 2.4 ROOF LOADING

Roof structures support a variety of loads. These loads are applied uniformly, non-uniformly or as concentrated loads. Only uniformly distributed and non-uniform loads are discussed in this chapter. Uniformly distributed loads on roofs are:

- Dead loads
- Collateral loads
- Code specified roof live loads
- Snow loads
- Rain loads
- Wind and wind uplift loads
- Load combinations
- Concentrated loads

Roof dead loads represent the self-weight of the roof construction. They include the weight of the roofing membrane, insulation, the decking and joists. The self-weight of the Joist Girder must be added for its design, and the weight of the columns must be added for their design and the design of foundations.

### Dead Loads

Unit dead loads are also found in technical publications and manufacturer's literature. A very complete presentation is given in ASCE 7 in Tables C3.1-1a and C3.1-1b.

Representative dead weights of the roofing and decking types discussed previously are as follows:

| | |
|---|---|
| Roofing | |
| Built-up roof gravel surface | 5.5 to 6.5 psf |
| Adhered or attached single ply membrane | 1 to 2 psf |
| Ballasted membrane | 10 to 15 psf |
| Liquid applied | 2 to 5 psf |
| Structural metal roofs | 1 to 2 psf |
| Decking | |
| Steel decking | 2 to 3 psf |
| Lightweight insulation concrete (30 pcf) and steel deck | 10 psf |
| Gypsum concrete (50 pcf) | 15 psf |
| Precast concrete | 10 to 20 psf |
| 2" nominal wood planks | 5 psf |
| 3" nominal wood planks | 9 psf |
| Plywood per 1/8" of thickness | 0.4 psf |
| Structural wood fiber board | 3 to 6 psf |
| Insulation (per inch of thickness) | |
| Cellular glass | 0.7 psf |

| | |
|---|---|
| Fibrous glass | 1.1 psf |
| Fiberboard | 1.5 psf |
| Perlite | 0.8 psf |
| Polystyrene foam | 0.2 psf |
| Urethane foam | 0.5 psf |

### Collateral Dead Loads

Collateral loads represent a category of dead loads which are not part of the building structure but are required for the building's function. These include:

- Mechanical equipment
- Piping
- Electrical equipment
- Conduit
- Sprinkler piping
- Fire proofing
- Ceilings

When these collateral loads can be attached to the structure with multiple uniformly spaced hangers such that each hanger reaction on the joists is less than 100 pounds, these systems can be accounted for as uniform loads. In some cases, a theoretical overstress may exist due to the hanger reaction; however, from a practical point of view the overstress can be neglected. When hanger loads exceed 100 pounds, they should be accounted for by special designs using concentrated loads or bend check loads.

### Roof Live Loads

Roof Live Load is defined by ASCE 7 as "A load on a roof produced (1) during maintenance by workers, equipment, and materials, and (2) during the life of the structure by movable objects, such as planters or other similar small decorative appurtenances that are not occupancy related. An occupancy-related live load on a roof such as rooftop assembly areas, rooftop decks, and vegetative or landscaped roofs with occupiable areas, is a 'live load' rather than a 'roof live load'."

Appropriate values for roof live load are defined in Table 4.3-1 of ASCE 7 and can be reduced per Section 4.8 of ASCE 7 based on the tributary area of the structural element being designed. Vulcraft will not perform this load reduction process but will assume that it has been considered and applied, if warranted, by the specifying professional.

### Snow Loads

The design roof snow load is determined per ASCE 7 Section 7 based on the defined 50-yr MRI Ground Snow Load defined in Figure 7.2-1. In some cases, the location may fall within a region defined as 'CS', which means a Case Study is warranted to determine the value at this location. In most 'CS'-defined locations, one can determine the appropriate ground snow load via consultation with local building officials.

The calculation of design roof snow load is a factoring of the ground snow load. This factoring accounts for the documented difference between ground snow and roof snow and is modified for exposure and thermal conditions and the importance of the facility. Snow loads are further modified to account for buildup of snow at roof offsets and roof obstructions (drift loads). Some methods also account for the change in expected snow density in drifts. Where drift loads exist on joists and Joist Girders load diagrams shall be provided by the specifying professional.

## Rain Loads

Codes are beginning to recognize rain loads as separate from roof live loads and snow loads. Currently the requirements are more descriptive than prescriptive. Their goal is to prevent the buildup of water beyond that anticipated in the provision of roof live load. Water can accumulate on a roof either intentionally when a controlled flow roof drain scheme is used or accidentally when drains are blocked. In both these conditions relief or overflow drainage must be provided. Relief may be obtained via other drains, overflow at roof edges or parapet scuppers. The governing code must be carefully reviewed by the building designer for these provisions. In any event, as a minimum, a roof area tributary to a drain must be designed for the depth of water which accumulates if that drain is plugged.

ASCE 7 also recognizes the potential for rain-on-snow and requires the addition of 5 psf to snow loads where the ground snow is ≤ 20 psf (but not zero) for all roofs with slopes (in degrees) less than W/50 where W is defined as the horizontal distance from eave to ridge (in feet). It is also stated that, "The additional load applies only to the sloped roof (balanced) load case and need not be used in combination with drift, sliding, unbalanced, minimum, or partial loads."

Rain loads may also create a condition called ponding. This word has different meanings in the literature on roofs and roof loading. Among roofers, ponding means the accumulation of water in low spots which evaporation will not remove in forty-eight hours. Building codes use the word to describe the accumulation of water due to blocked drains. Lastly, the engineering profession uses it to describe the potential for instability of the roof due to the progressive accumulation of water in the deflected curve of the roof framing. This last condition is presented in detail in Chapter 5.

## Wind and Wind Uplift

Roofs are subjected to uplift forces induced by wind blowing on and over the building. These forces vary in intensity depending on building exposure, building geometry and wind velocity. The force also varies in intensity over the roof surface. It is greater in intensity at roof edges and corners.

Building codes provide minimum wind forces on both main wind-force resisting systems as well as components and the cladding thereof. For example, Chapters 26 to 30 of ASCE 7 provide multiple approaches for the determination of wind loads on the various elements of a structure.

Should the project require Factory Mutual insurance underwriting, the specifying professional for the project should consult with the FM Regional Engineer to determine any provisions they may require be met as part of the design of the structural system. The specifying professional remains responsible for the interpretation of these requirements on the job. The following guidelines outline Vulcraft's understanding of two possible approaches that may be taken concerning additional roof deck design requirements FM may require.

**FM Wind Rating Specified only**: In this situation, the FM Regional Engineer specifies that the roof must have an FM wind rating such as 1-60, 1-75 or 1-90. This may be all that is specified. In this case, the specifier may refer to the Vulcraft document "Vulcraft FM Deck Data" (located at www.vulcraft.com/decks/Factory%20Mutual) and determine needed information as follows:

1. The user will refer to Section 9.1.1 for mechanical support fasteners and 9.1.2 for welds. Definition of the fastener patterns at both the supports and sidelaps is provided in the initial paragraph of each of these sections. This fastener pattern is appropriate for Zone 1, Field, only
    a. The fastener pattern in Zone 2, Edge, should be two times the requirement in Zone 1
    b. The fastener pattern in Zone 3, Corner, should be two-and-one-half times the requirement in Zone 1

2. Once the appropriate table for the desired deck type is located within the section chosen in Step 1, choose deck gages for the secondary support spacing desired and that match the FM wind rating specified. Values are provided for one, two and three-span conditions. Please note that these maximum span values include consideration of:
    a. A construction loading of 200 pounds, including strength and a serviceability limit of L/240
    b. Pull-over resistance of the fasteners at the FM wind rating given per the AISI Specifications
    c. Flexural strength of the deck loaded at the FM wind rating given, assuming the attached membrane spacing is less than one-half the deck span
        i. If the membrane attachment spacing is greater than one-half the deck span, the specifier should switch to the method defined below from FM Data Sheets 1-28 and 1-29

The approach outlined in the Vulcraft document is sufficient only for wind ratings up to 1-90. In situations where deck profile or gages desired are not listed in the Vulcraft FM documents available at www.vulcraft.com/decks/Factory%20Mutual or the wind rating required is greater than 1-90, the specifier should contact Vulcraft for assistance in seeking special consideration from the FM Regional Engineer.

**Required to Meet FM Data Sheets 1-28 and 1-29**: If the project requires adherence to the full provisions outlined in FM's Data Sheet 1-28 (Wind Design) and 1-29 (Roof Deck Securement and Above-Deck Roof Components), the following approach may be followed.

1. Determine the required wind ratings for the field, edge and corner zones of the roof per requirements in FM 1-28. This is done by:
    a. Determining a Roof Design Negative Pressure (RDNP) from Tables 3, 4 or 5, as appropriate
    b. Determining the RDNP multipliers for each roof zone from Table 6
    c. The wind rating for each roof zone is the value from Step 'a' multiplied by the RDNP Multiplier and then by two and rounded up to the nearest multiple of 15. See Section 3.3.2, page 24, for an example
2. Determine the maximum steel deck span from FM 1-29 Tables 1A-1D (for 1.5B-decks) or 1E (for 3N decks) for the roof covering required. Please note also the needed deck gage and yield strength. This will apply for Zone 1
    d. Please note that FM 1-29 Section 2.2.3.12.2 recommends that the spans in Tables 1A-1E should be reduced by 10% if Acoustical Deck is required
    e. Also, should the Zone 1 wind rating required be greater than 1-90, the yield strength of the deck is required to be 80 ksi and the number of support fasteners per flute should be doubled
3. The Support Fastener Pattern should be determined for the type desired using the RDNP determined in Step 1a above. This pattern would then be used in Zone 1
    f. The "Connection Tension" tab of the Vulcraft Online Design Tool called "2018 IBC Deck Diaphragm" can be used to quickly determine needed fastener uplift capacities
    g. Please note that FM 1-29 Section 2.2.3.5 suggests that mechanical fasteners are preferred over welding for fire-prevention reasons. If welding is to be used, FM Data Sheets 1-0 and 10-3 must be considered as well
4. Zone 2 and Zone 3 support fastening patterns are most-easily determined using the prescriptive approach outlined in FM 1-29 Section 2.2.3.4, which are:

   h. For Zone 2, double the fasteners required for Zone 1
   i. For Zone 3, use two-and-one-half times that required for Zone 1

5. Sidelap fastener patterns can be determined from FM 1-29 Table 4 using the Zone 1 wind rating

Ballasted roofs present a different situation. With ballasted schemes the edges and corner zones require increased ballast to counterbalance the uplift forces. The roof design must provide capacity for the extra weight of ballast. Roof deck attachment is only of concern for uplift where schemes combine ballast and insulation attachment in the perimeter and corner zones. Such schemes are only recommended for roofs where the structure cannot support increased ballast loads.

The Steel Deck Institute in its Standard Specification for roof deck gives the following requirement. "Deck Support Attachment: Steel deck shall be anchored to structural supports by arc spot welds, fillet welds, or mechanical fasteners. The average attachment spacing of deck supports perpendicular to the span of the deck panel shall not exceed 16 inches (400 mm) on center, with the maximum attachment spacing not to exceed 18 inches (460 mm), unless more frequent fastener spacing is required for diaphragm design. The deck shall be adequately attached to the structure to prevent the deck from slipping off the supporting structure."

The strength of the connections mentioned above can be obtained from manufacturer's published test values or from the AISI Specifications (AISI, 2016a), Chapter J. Welds can also be evaluated using the AISI S100 Equations J2.2.3-1 and J2.2.2-2.

### Connection Design for Net Uplift Forces

Net uplift due to wind loading is one key design consideration for open web steel joists used in roof systems. The uplift resistance of the joist seat itself, along with the capacity of the welds or bolts, which connect the seat to the supporting structure, are vital links in the load path when considering wind uplift in a roof system. The specifying professional must specify net uplift loads required for Vulcraft to incorporate in the joist design.

In the 44th Edition of the SJI Standard Specifications and Load and Weight Tables for Steel Joists and Joist Girders Catalog (SJI, 2017b), in the section on end anchorage for uplift, it states that "The adequacy of the end anchorage (bolted or welded) between the joist or Joist Girder bearing seat and the supporting structure is the responsibility of the specifying professional. The joist manufacturer is responsible for the design of the bearing seats of the joists and Joist Girders for the loads designated by the specifying professional on the contract documents. See Section 6.1(b) of the SJI Code of Standard Practice (COSP)."

For additional information on welded and bolted end anchorage uplift capacities and design examples, refer to Steel Joist Institute Technical Digest 6, "Structural Design of Steel Joist Roofs to Resist Uplift Loads."(SJI, 2012)

### Load Combinations

Building Codes specify the combination of the various load types for the design of systems and elements. These combinations reflect a judgment as to the probability of the simultaneity and intensity of the loads in question. It is not always necessary to bring all of the loads from all sources to bear on a system or element at full intensity at one time. The applicable building code requirements should be studied and followed regarding load combinations. Vulcraft combines multiple loadings per the applicable building code unless instructed otherwise by the specifying engineer.

### Concentrated Loads

The treatment of concentrated loads and the specification of loading on joists and Joist Girders is covered in Chapters 5 and 6, and in the Vulcraft Manual.

## 2.5 SERVICEABILITY CONSIDERATIONS

The proper function of roofing, ceilings and other building components related to the roof structure is a building serviceability concern. A strength design which adequately supports the weight of the roofing and the roof system may not provide adequate functional performance. In roof structures, serviceability design largely concerns the control of deflections, but also concerns design for roofing expansion and contraction, as well as, building movement.

### Control of Roof Deflections

What follows is a brief review of deflection limits and loads for steel deck and joists from various sources.

<u>International Building Code (per Table 1604.3)</u>

1. Span over 360, uniformly distributed live load supporting plaster or stucco ceiling
2. Span over 240, uniformly distributed live load supporting non-plastered ceiling
3. Span over 180, uniformly distributed live load not supporting ceiling

<u>Steel Deck Institute:</u>

1. Span over 240, uniformly distributed live load
2. Span over 240, 200 lb. concentrated load at midspan on a one-foot section of deck

<u>Steel Joist Institute:</u>

1. Span over 360, design live load where plaster ceilings are attached or suspended
2. Span over 240, design live load in all other cases

<u>FM Global:</u>

Span over 200, 300-pound concentrated load at midspan

Note that the Commentary to IBC Section 1604.3 states the following. "In computing deflections to verify compliance with Table 1604.3 limits, the loads shown in the column headings of Table 1604.3 are the only loads that must be applied to the member. It is not necessary to use the load combinations of Section 1605.3 for verifying that the deflection limits have been met."

ASCE 7 Commentary Section CC.2.1 further explains that, "For serviceability limit states involving visually objectionable deformations, reparable cracking or other damage to interior finishes and other short-term effects, the suggested load combinations are: $D + L$ and $D + 0.5S$. … The dead load effect, $D$, may be that portion of dead load that occurs after attachment of nonstructural elements. Live load, $L$, is defined [as Occupancy-Based Live Load and not Roof Live Load (separately labeled as $L_r$)]."

Thus, it is appropriate to check roof systems for a serviceability load combination of $D_{col} + 0.5S$ only.

The NRCA also points out that the roof structure must provide positive slope to drains (NRCA, 2015). This generally means that internal roof drains should be located at columns, i.e. non-deflected low points, and that the roof pitch versus member deflection should be checked.

Partitions and vertical plumbing stacks which are attached to the roof or penetrate the roof must be detailed in a compatible fashion to allow roof deflections and not allow inadvertent loading of partitions and stacks. Vertical slip joints are needed.

Building codes do not indicate total load deflection limit, nor do they dictate camber.

## Expansion Joints

The number and location of roof expansion joints or building expansion joints is a design issue not fully treated in technical literature. The National Roofing Contractors Association (NRCA, 2015) gives the following recommendations for the location of roof expansion joints.

1. Where expansion or contraction joints are provided in the structural system
2. Where steel framing, structural steel, or deck material change direction
3. Where separate wings of L, U, T or similar configurations exist
4. Where the type of deck material changes; for example, where a precast concrete deck and a steel deck abut
5. Where additions are connected to existing buildings
6. At junctions where interior heating conditions change, such as a heated office abutting unheated warehouse, canopies, etc. A two-piece reglet and counterflashing assemblies can accommodate the movement at main building-to-canopy intersections
7. Wherever differential movement between vertical walls and the roof deck may occur."

NRCA standard details show that the roof structure under roof expansion joints is intended to be discontinuous.

In 1974 the Building Research Advisory Board of the National Academy of Sciences (NAS) published Federal Construction Council Technical Report No. 65 "Expansion Joints in Buildings" (NAS, 1974). It cites recommendations of the Brick Institute of America and the National Concrete Masonry Association, that buildings supported by continuous exterior unreinforced masonry walls, be expansion jointed at intervals not exceeding 200 feet.

The report also presents the figure shown in Figure 2.5 as a guide for spacing expansion joints in beam and column frame buildings based on design temperature change. The report includes temperature data for numerous cities. The data given are Tw, the temperature exceeded only 1% of the time during summer months, Tm, the mean temperature during the normal construction season and Tc, and the temperature exceeded 99% of the time during winter months. The design temperature change is the larger of the two temperature differences either ($T_w$-$T_m$) or ($T_m$-$T_c$). The (NAS) figure gives five modifying factors which should be applied to the Allowable Building Length as appropriate.

"Maximum allowable building length without use of expansion joints for various design temperature changes. These curves are directly applicable to buildings of beam and column construction, hinged at the base and with heated interiors. When other conditions prevail, the following rules are applicable.

1. If the building will be heated only and will have hinged column bases, use the allowable length as specified
2. If the building will be air conditioned as well as heated, increase the allowable length by 15 percent (provided the environmental control system will run continuously)
3. If the building will be unheated, decrease the allowable length by 33 percent
4. If the building will have fixed column bases, decrease the allowable length by 15 percent
5. If the building will have substantially greater stiffness against lateral displacement at one end of the plan dimension, decrease the allowable length by 25 percent

When more than one of these design conditions prevail in a building, the percentile factor to be applied should be the algebraic sum of the adjustment factors of all the various applicable conditions."

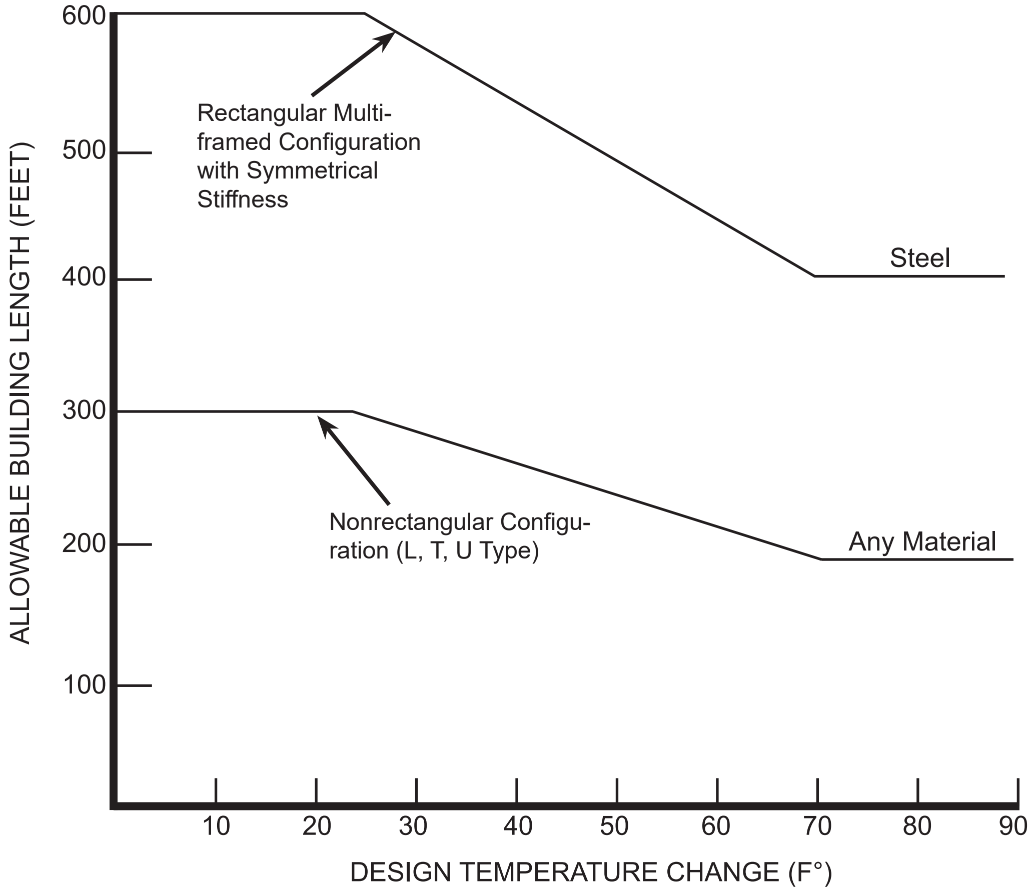



**Fig. 2.5 Expansion Joint Spacing Graph**

[Taken from F.C.C. Tech. Report No. 65, "Expansion Joints in Buildings"]

## Area Dividers

Area dividers are provided in roof membranes to control the effects of thermal loads. They are relief joints in the membrane and do not require a joint in the roof structure below. They are also used to divide complex roof plans into simple squares and rectangles. In large roofs they are placed at intervals of 150 to 250 feet depending on the membrane manufacturer's recommendations. The limits given above apply to built-up roofs and adhered single ply membranes. Greater distances between area divider joints can be used in ballasted systems.

Structural steel roofs are more strictly limited in the allowable dimension parallel to the ribs. Standing seam roofs are limited by the range of the sliding clips. Depending on the manufacturer, it is in the range of 150 to 200 feet. Through fastener roofs rely on purlin roll to prevent slotting of the roof panels. Because of their greater stiffness steel joists should rarely be used with through-fastened roofs. A practical limit between dividers is in the range of 100 to 200 feet when these roofs are attached to light gage cold-formed purlins.

Structural steel roofs are more flexible in the direction perpendicular to the ribs, so area dividers can be spaced at greater distances. The roof manufacturer's recommendations should be consulted and followed.

NRCA also comments on roof area dividers indicating that if they are used, they should be flashed to a minimum height of 8 inches above the roof surface and that they should not restrict or impede drainage.

### Roof Slope

Roof slope is also a factor in roofing performance. For membrane roofs, ¼ in. pitch per foot is generally recommended. For structural steel roofs the minimum pitches are on the order of ¼ in. per foot for standing seam roofs and ½ in. per foot for through fastener roofs. The International Building Code requires a minimum slope of ¼ in. per foot except for coal tar roofs where a slope of ⅛ in. may be used.

### Free Drainage

All roofs should be designed and constructed so that water is not retained on the roof surface. Even in roofs that are constructed with ¼ in. per foot slope there are instances where free drainage may not occur. A classic example is a roof with no interior drains that drains to an eave gutter. This situation occurs when the first upslope joist or purlin deflects under snow load more than the eave member deflects. Often the eave member does not deflect as it is supported by the building siding. A check can be made by the specifying professional for this situation. The SJI Roof Bay Analysis Spreadsheet can also be used for this check. The situation is also discussed in SJI TD #3 on "Ponding." The tool can be downloaded free of charge from the SJI Website, www.steeljoist.org under the tab "Design Tools."

When through fastened or standing seam metal roofs are used, it is extremely important that the manufacturer's installation procedures for closures be followed, especially at the eaves, to prevent wicking of the water under the roof panels.

## 2.6 FRAMING CONSIDERATIONS

### Bay Size

The designer may or may not have the opportunity to select the bay size for a proposed project. Owner requirements and functional requirements often dictate a certain bay size. In addition, the building footprint, which is often dictated by the building site, has an impact upon the bay size selected. In general, for single story buildings without cranes, bay sizes ranging from 30'x30' to 60'x60' have proven to be economical. Square bays have been shown to provide greater economy than rectangular bays. Gravity loads have the greatest impact on the optimum bay size if the size is not dictated by one of the aforementioned items. Lighter roof loads allow larger bays without cost penalty.

When the structure has a high ratio of perimeter length to enclosed area, e.g., a long narrow building, then a 30'x40' or a 30'x50' bay where the 30' dimension is parallel to the long building dimension often proves to be the most economical. This is since with long narrow buildings the economy is heavily influenced by the wall system. For example, if a metal wall system is to be used, then the most economical girt system is one in which light gage cold-formed steel girts are used. These are typically C or Z girts. The maximum span of such girts is approximately 30 feet. If a bay spacing larger than 30 feet is required, then wind columns are required to laterally support the C or Z girts in mid-bay. The wind columns and their attachments to the structural steel at the roof have a significant impact on the cost of the framing system. For metal wall structures with bays larger than 30 feet, the designer is encouraged to investigate the use of steel joists for the girt system as an alternative to wind columns and cold formed purlins. If the owner wishes to use cold formed purlins, then a 30'x40' or 30'x50' bay size may prove to be the most economical system.

For structures with a low ratio of perimeter length to area, e.g., square buildings of significant size (200'x200'), the percentage of steel that would be contained in the wall framing is less of a cost factor, and thus a 40'x40' often proves to be the most economical system. Larger bays of 40'x50', 50'x50' or 40'x60' are also economical.

In general, soil conditions will not have a major impact on the selection of the bay size when shallow foundations can be used. However, if very poor soils exist and deep foundations are required, larger bays will tend to be more economical because of the reduced number of deep foundations. This assumes, of course, that the floor slab can be placed on grade and there does not have to be a structural floor system.

Similar judgments as to bay spacing are required when overhead top running cranes are to be contained in the structure. Typically, bay spacings should be limited to approximately 30 feet for these structures. This is since the crane runway beams will have a significant impact on the cost of the structure. A 30 -foot runway girder is significantly less expensive than a 40-foot runway girder. In addition, the AISC Specification requirements for tension flange bracing also begin to dictate costs with runways spanning 40 feet. This adds structural steel and expensive miscellaneous details.

### SJI Roof Bay Tool

The SJI provides a "Design Tool" to assist the specifying professional on optimizing roof bay size. The tool is entitled, "Roof Bay Analysis Tool." The tool can be down loaded free of charge from the SJI Website, www.steeljoist.org under the tab "Design Tools." The user can input various scenarios to arrive at the least weight or the least cost bay size. Cost data can be input by the user along with other design data. Bays can be evaluated using either ASD or LRFD. In addition, the bay can be evaluated for roof ponding stability, using an iterative analysis. Pull down menus allow for easy selection of steel deck, joist (K, LH, DLH- Series) and Joist Girder selections.

### Direction of Joist Span

One question that often occurs is whether it is best to span the joist in the long direction or in the shorter direction when a rectangular bay has been selected. The designer is encouraged to examine alternate framing schemes for a given project using the "SJI Roof Bay Tool" mentioned previously, and to contact Vulcraft to discuss the least expensive system. Prices can vary for joists and Joist Girders depending upon plant workload and market conditions.

### Joist Spacing

Joist spacing should be maximized depending on the type of deck being used. Typically, the fewer number of pieces which need to be erected will reduce the cost of the erected steel. Longer joists should be spaced further apart since each one requires significant shop labor. The joist should be spaced to maximum values of the deck, but with spaces no greater than those recommended for construction practice as contained in the Steel Deck Institute specification. In addition, the designer should check to see if FM Global requirements must be followed. If so, then the FM Global recommended joist spacings should be followed.

If a standing seam roof is being used, typically a 5-foot joist spacing is used. This is since UL 90 uplift requirements for most standing seam roof systems can only be met with a 5-foot joist spacing. It should be noted that Nucor Building System's standing seam roof system has a UL 90 uplift rating with a joist spacing of 5'-6".

### Joist and Girder Depth

The optimum Joist Girder depth in inches is approximately equal to the span of the girder in feet. The designer should generally follow this rule of thumb. However, for expensive wall systems, such as architecturally treated, tilt up and precast systems, a one-foot savings in height of structure may prove more economical as compared to the extra cost of shallower Joist Girders.

Joist depth should be selected based on the Economical Joist Guide contained in the Vulcraft Joist and Joist Girder Systems Manual; but the designer should also examine bridging requirements for the selected joist. It may be that by selecting a slightly heavier joist, a line of bridging can be eliminated thus resulting in a substantial decrease in the total cost of erected steel. If possible, joist selections should also be made so that X-bridging is not required.

The reader is referred to Vulcraft's "Steel Joist & Joist Girder Systems" (Vulcraft, 2017c) Manual (www.vulcraft.com/catalogs/steel-joists-joist-girder-systems) for additional information on joists and Joist Girders.

## 2.7 CONNECTIONS

For roof framing, the most widely used and least expensive connection for joist to Joist Girder to column framing is shown in Figure 2.7.1. The Joist Girder is placed directly on top of the column and the joist which frames into the column at right angles to the girder is placed atop the girder. The joist is bolted to the girder seat and the girder seat is bolted to the column cap to satisfy OSHA requirements and for ease of construction. When tube columns are used, the Joist Girder seat bolts are often placed outside of the face of the column in order to eliminate the need for threaded studs. These connections can later be welded if specified by the designer. Vertical stabilizer plates are positioned to line up with the bottom chord of the Joist Girders. Holes are provided in the stabilizer plates per OSHA to secure guying cables. The bottom chord of the Joist Girder slides around the stabilizer plate. This prevents twisting of the girder during erection and is required by SJI. The stabilizer also serves to laterally brace the bottom chord of the girder after erection. The bottom chords of the joists and Joist Girders should not be welded to the stabilizer plates unless the resulting continuity effects are investigated by the building designer. Per OSHA, stabilizer plates with guying cable holes are also required for joists located at columns.

This connection will be referred to frequently throughout this book. Since the connection is basic in nature to most framing systems, it will be referred to as the Basic Connection. In many cases joist and Joist Girder seats may require welding to their supporting elements to prevent slip or for additional strength.

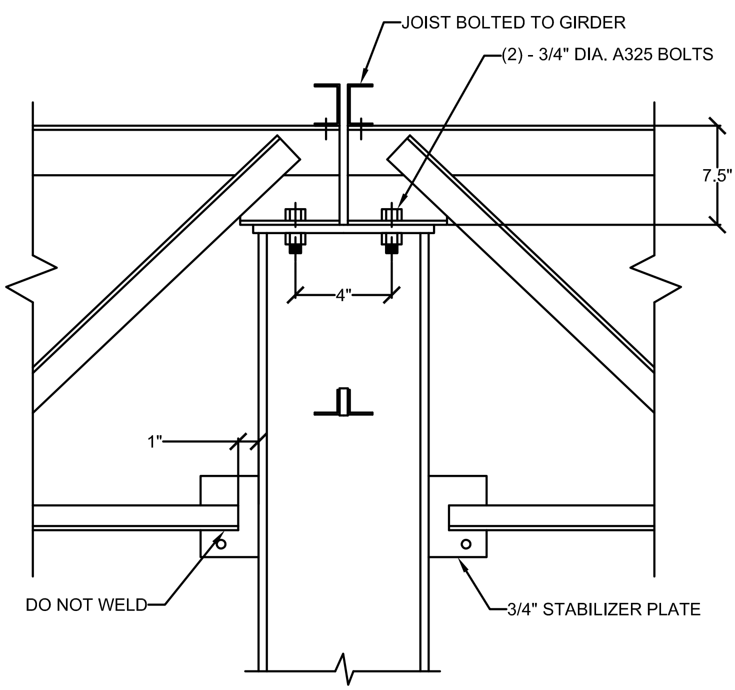



**Fig. 2.7.1 The Basic Connection**

The Basic Connection is also used when framing Joist Girders into the weak axis of wide flange columns. This condition is shown in Figure 2.7.2.

The stabilizer plates should be detailed to extend beyond the cap plate so that the girder can be erected without tilting the columns.

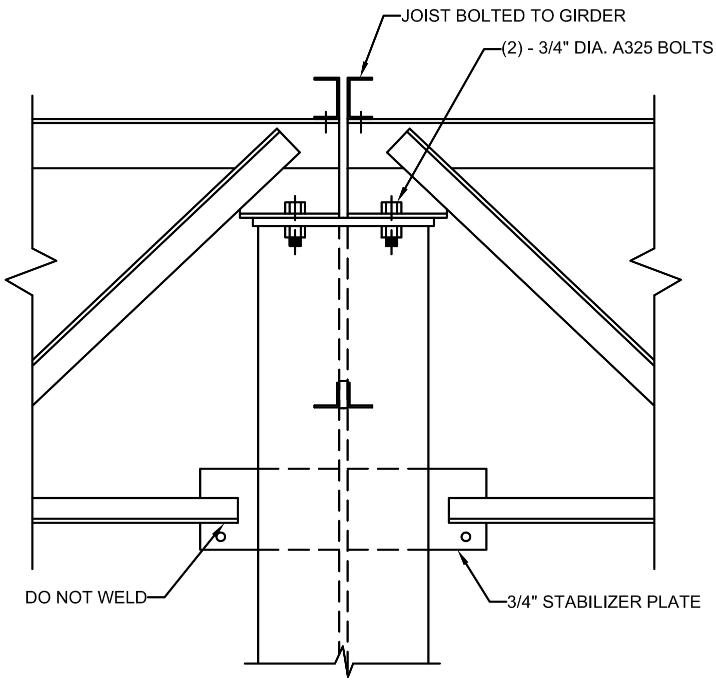



**Fig. 2.7.2 The Basic Connection**

Various modifications can be made to the Basic Connection in order for the connection to resist moments in the joists or Joist Girders. These modifications are discussed in Chapters 4 and 7.

# Chapter 3

# Floors

## 3.1 INTRODUCTION

This chapter presents considerations for floor system designs using steel deck, steel joists and Joist Girders.

This discussion covers the following topic areas:

- Floor decks
- Floor loading
- Serviceability considerations
- Framing considerations
- Shear connectors
- Connections

## 3.2 FLOOR DECKS

Floor decks on steel joists are created using three basic approaches.

- Cast-in-place concrete on steel deck
- Precast slabs
- Plywood and wood decking

### Cast-In-Place Concrete on Steel Deck

The following steel deck types are manufactured by Vulcraft and are used in floor construction.

1. Vulcraft Non-Composite Decks, 0.6C/0.6CSV, 1.0C/1.0CSV, 1.3C/1.3CSV, 1.5C, 2C and 3C
2. Vulcraft VL Composite Floor Decks, 1.5VL/1.5VLI, 1.5VLR, 2VLI and 3VLI
3. Vulcraft Acoustical Cellular Decks 1.5VLPA, 2VLPA and 3VLPA
4. Vulcraft Cellular Decks, 1.5VLP, 2VLP and 3VLP

For further information on Vulcraft Floor Decks consult Vulcraft's "Steel Roof & Floor Deck" Manual (Vulcraft, 2018) and SDI's "Standard for Non-Composite Steel Floor Deck" (SDI, 2017c).

### Form Deck

Conform decks are form decks. As form decks, they must safely support the weight of wet concrete and construction activity. The Steel Deck Institute's manuals, "Roof Deck Design Manual (RDDM)" (SDI, 2017g), and "Floor Deck Design Manual (FDDM)" (SDI, 2017e), contain requirements for loads during construction for the weight of the deck and the weight of wet concrete plus the greater effect of either 20 psf uniform load or 150 lbs. concentrated load on a one-foot width of decking. A deflection limit of span over 180 with a maximum of ¾ in. is given for the load case consisting of the weight of deck and concrete. This deflection is to be taken as relative to the deck supports. Vulcraft's manual, "Steel Floor and Roof Deck," provides deck load tables which give profiles, gages and spans of steel deck which comply with the SDI criteria.

Steel form deck for floors is manufactured from sheet steel and is available in three finishes: painted, galvanized (ASTM A924 or ASTM A653 (G60)) and unfinished (black). Form deck which is permanent carries the weight of itself and the concrete both in its wet state and when it has hardened. Thus, load tables for reinforced slabs on form deck do not include the weight of the slabs. Galvanized form deck and painted form deck in most applications are permanent form decks. The appropriateness of painted form deck should be considered as one would evaluate painted roof deck and thus is appropriate for most situations. Neither painted roof nor floor deck is appropriate in certain high moisture environments. Unfinished steel form deck is not considered a permanent form.

Form decks must be designed for construction loads, because it is rarely feasible either from the standpoint of time or money to shore them. Thus, the profile and thickness should be selected from the tables in the Vulcraft Manual based on an un-shored condition to carry the weight of concrete and construction loads. The Vulcraft Manual (Vulcraft, 2018) also gives allowable uniform load capacities using three criteria: allowable stress of 36,000 psi, deflection of span over 240 and deflection of span over 180 for single, double and triple clear spans.

The Vulcraft Manual also presents load tables for the finished slabs. For form decks, flexure reinforcement for superimposed loads is provided using welded wire reinforcement. For thin slabs and light loads this reinforcement is to be installed at a constant depth. For thicker slabs and heavier loads, the reinforcement is to be draped, that is high over the supports for negative moment and low in mid span for positive moment. Proper reinforcement supports, to ensure the correct positioning of the fabric, must be specified on the drawings to ensure that the proper supports are supplied. At times loads, spans and the need to accurately position the reinforcement may require the use of reinforcing bars as opposed to reinforcing fabric. The Vulcraft load tables give the required reinforcement to meet the flexural requirements of the tabulated load and span. In some cases, these reinforcement areas do not meet the ACI 318 requirements for minimum shrinkage and temperature requirements. It should be noted, however, that these less than ACI reinforcement ratios have been used with success historically.

## Composite Deck

Composite decks serve a dual purpose. During construction they serve as a form to support the weight of wet concrete and construction loads. After the concrete has hardened, it is engaged with the deck by interlocking in the embossed sides of the flutes so that the concrete and steel deck act compositely. The steel deck provides positive moment reinforcement. Because the deck forms the positive reinforcement, it must be permanent. SDI in its commentary to its composite deck specification, recommends the use of ASTM A653 (G60) galvanizing. The specification itself only requires that the finish "be suitable to the environment." Vulcraft has found that good long-term performance can be achieved under usual conditions using phosphatized/painted deck. If the deck is to be fireproofed the finish must be compatible with fire proofing. Also, electrical raceway decks must be galvanized.

Composite decks are treated as a series of simple spans for the purpose of carrying live loads, and as such are not provided with negative moment reinforcement. Rather shrinkage and temperature reinforcement are provided. The amount of reinforcement is recommended in the SDI Specification as "0.00075 times the area of concrete above the deck but shall not be less than the area provided by 6x6-10/10 (6x6-W1.4xW1.4) welded wire fabric."

This reinforcement ratio is less than that which would be required by the American Concrete Institute code but represents an amount which has provided good historical performance. This approach to reinforcement may allow negative moment cracking to form over the supports. This is a serviceability concern, not a strength concern. Where deck bears on girders, transverse cracking may also occur. The designer should consider additional reinforcement over girders.

Published in the Vulcraft Manual are load tables giving superimposed live load capacities for various slab thicknesses, gages, profiles and spans for both normal weight and lightweight concrete. These tables also give the maximum span using the SDI criteria for one, two and three span conditions in an un-shored condition. The tabulated maximum spans for an unshored condition do not include the effect of web crippling, which must be checked using the tabulated allowable reactions presented elsewhere in the Vulcraft Manual or with the Vulcraft Span Calculator available at www.vulcraft.com/design-tools.

Example 3.2.1 illustrates a situation where web crippling governs the thickness and profile selection. This illustrates the importance of taking this extra step when selecting a thickness and profile from the tables. The use of un-shored deck is almost universally preferred. The tables should be used to select a proper thickness and profile based on an un-shored condition.

The SDI "Standard for Composite Steel Floor Deck-Slabs" (SDI, 2017b) lists several areas of concern regarding the use of composite decks. The major points are:

1. Parking garages:
    a. Slabs should be designed with negative moment reinforcement
    b. Added shrinkage and temperature reinforcement should be provided
    c. Care must be taken when deicing salt or sea salt is present. As a minimum, the top surface of concrete should be sealed, galvanized deck should be used and the underside of the deck should be painted
    d. Strong consideration should be given to using the steel deck as a stay in place form only.
2. Cantilevers require special top reinforcement. The deck should be used as a form only. The design of which is the responsibility of the building designer
3. Dynamic loads, such as heavily loaded fork trucks, can destroy the mechanical interlock between deck and concrete. The use of composite deck is not recommended where forklifts are used unless the use is an infrequent occurrence

Vulcraft Manual gives uniform superimposed load capacities for various profiles, thicknesses and spans. At times it becomes necessary to check slab capacities for line loads and concentrated loads. Design approaches and test results have been put forward over the years to address these loading conditions. The current state of the art is presented in the SDI "Floor Deck Design Manual." As its title indicates it covers other areas as well as the treatment of line and concentrated loads.

## Concentrated Loads

The SDI "Standard for Composite Steel Floor Deck-Slabs" (Section 2.4) provides a method for analyzing concentrated loads. The method provided is appropriate for any magnitude of load and uses an effective distribution width in the context of defined dimension parameters and an upper limit on effective width. Based on the method of analysis provided, the slab's flexure and shear strengths can be established using ASD or LRFD principles. The Standard indicates that, "Concentrated loads shall be distributed laterally (perpendicular to the ribs of the deck) over an effective width, $b_e$. The load distribution over the effective width, $b_e$, shall be uniform." The Standard also indicates that, "The concrete above the top of steel deck shall be designed as a reinforced one-way concrete slab in accordance with ACI 318 Chapter 7, transverse to the deck ribs, to resist the weak axis moment. $M_{wa}$, over a width of slab equal to W."

The defined parameters and effective widths are[1]:

$b_m = b_2 + 2t_c + 2t_t$

$b_e = b_m + 2(1.0\text{-}x/L)x \leq 106.8(t_c/h)$ for single span bending

$= b_m + (4/3)(1.0\text{-}x/L)x \leq 106.8(t_c/h)$ for continuous span bending when reinforcing steel is provided

in concrete to develop negative bending

$= b_m + (1.0\text{-}x/L)x, \leq 106.8(t_c/h)$ for shear

[1] Also see ANSI/SDI C-2017 and the User Notes in Figures 2-2 and 2-3.

$W \; = L/2 + b_3 \leq L$

$M_{wa} = 12Pb_e/15W$ lb-in./ft

where

$b_e$ = Effective width of concentrated load, perpendicular to the deck ribs, in.

$b_m$ = Projected width of concentrated load, perpendicular to the deck ribs, measured at top of steel, in.

$b_2$ = Width of bearing perpendicular to the deck ribs, in.

$b_3$ = Length of bearing parallel to the deck ribs, in.

$h$ = Depth of composite deck-slab, measured from bottom of steel deck to top of concrete, in.

$L$ = Deck span length, measured from center of supports, in.

$M_{wa}$ = Weak axis bending moment, perpendicular to deck ribs, of width, in.-lbs per foot of width

$P$ = Magnitude of concentrated load, lbs.

$t_c$ = Thickness of concrete above top of steel deck, in.

$t_t$ = Thickness of rigid topping above structural concrete (if any)

$W$ = Effective length of concentrated load, parallel to the deck ribs, in.

$x$ = Distance from center of concentrated load to nearest support, in.

Examples 3.2.1 and 3.2.2 illustrate key issues in proper deck selection, i.e. unshored construction, web crippling, uniform load strength and live load strength.

### Example 3.2.1 Composite Floor Slab with a Line Load

Design a composite steel floor deck for the information given:

**Given:**

Deck clear span = 10 feet.

Unfactored superimposed live load = 80 psf

Unfactored 500 plf concentrated dead load perpendicular to the deck span located 2 feet from the left support.

A two-hour restrained assembly fire rating.

Concrete strength is $f'_c$ = 3000 psi.

### Solution:

1. Using the composite floor deck fire resistance ratings contained in the Vulcraft Steel Floor and Roof Deck Manual select a 3-1/4-inch lightweight concrete thickness above the deck. This thickness can be used to satisfy the concrete thickness required in many D 900-series fire resistant designs found in the UL Fire Resistance Directory where the deck is unprotected.
2. Try a 3VLI20 deck (DL = 48 plf, based on Vulcraft Unshored Span Calculator of 47.615 psf).
3. From the 3VLI Vulcraft load tables the Type 20 deck can support a uniform live load of 149 psf.

Determine the equivalent uniform load for bending:

Reactions:

$V_L$= (1.2)(48 plf)(10 ft)/2 + (1.6)(80 plf)(10.0 ft)/2 + (1.2)(500 plf)(8.0 ft)/(10.0 ft) = 1,410 plf.

$V_R$= (1.2)(48 plf)(10 ft)/2 + (1.6)(80 plf)(10.0 ft)/2 + (1.2)(500 plf)(2.0 ft)/(10.0 ft) = 1,050 plf.

Note the line load is continuous so for the wall load $b_e$ = 12.0 in.

From statics the point of zero shear is located 5.65 feet from the right support.

The required LRFD moment = (5.65 ft)(1,050 plf) [1.2(48 psf) +1.6(80 plf)](5.65 ft)$^2$/2 = 2,970 ft-lbs/ft = 35.6 kip-in./ft

The factored yield moment for the slab = 62.36 kip-in./ft
SDI Manual (Table 4D)

The slab is **o.k.** for bending.

Check shear strength:

The required LRFD shear = 1,410 plf

$\phi V_n$ = 5,660 lbs/ft
SDI Manual (Table 8C)

1410 lbs/ft ≤ 5,660 lbs/ft **o.k.**

The composite slab is **o.k.** for moment and shear.

Use the Vulcraft Unshored Span Calculator to check the bare deck subjected to the SDI construction loads. The Span Calculator is available on the Vulcraft website, www.vulcraft.com/design-tools.

The calculator determines the maximum simple span, two span and three spans for the specified bare deck when subjected to the SDI construction loading conditions. The checks are made in accordance with the AISI "North American Specification for the Design, Fabrication and Erection of Structural Steel Members" (AISI, 2016a). Detailed results can be viewed by "running" the "Calculation Tab."

Shown in Figure 3.2.1 is the input and the output for the "Unshored Span Calculator." The output indicates that even a simple span of up to 11.7 foot is permissible. Thus, the specified span of 10 foot meets all criteria.

## Maximum Unshored Span

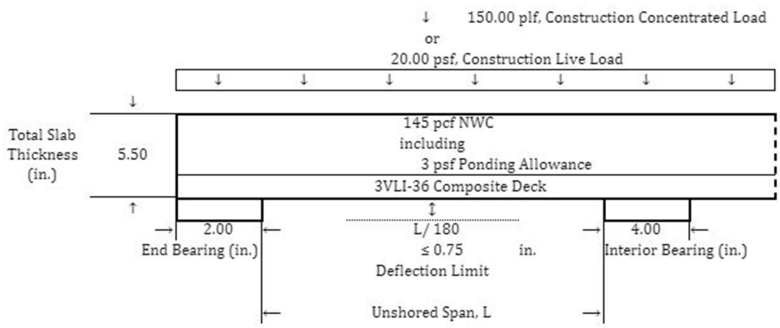
Input Design Criteria

NUCOR VULCRAFT GROUP

Print

*Design for Maximum Unshored Span of Composite Steel Deck*

| | | |
|---|---|---|
| Unit System | Imperial | |
| Design Method | ASD | |
| Deck Option | Composite Deck | |
| Deck Type | 3VLI-36 | |
| Total Slab Thickness (in.) | 5.50 | ≥ 5 |
| Concrete Unit Weight (pcf) | 145.00 | ≥ 90 |
| End Bearing (in.) | 2.00 | ≥ 0.75 |
| Interior Bearing (in.) | 4.00 | ≥ 0.75 |
| Construction Live Load (psf) | 20.00 | ≥ 20 psf |
| Construction Concentrated Load (plf) | 150.00 | ≥ 150 plf |
| Concrete Ponding Allowance (psf) | 3.00 | |
| Construction Deflection Limit | L / 180 | |
| Const. Deflection not to exceed (in.) | 0.75 | ≤ 0.75 in. |




## 3VLI-36 Grade 50 Composite Deck (ASD)

with 5.5 in. 145 pcf NWC

NUCOR VULCRAFT GROUP

**Maximum Unshored Span**

| Gage | 1 Span | 2 Span | 3 Span |
|---|---|---|---|
| 22 | 9'-0" | 8'-7" | 9'-9" |
| 20 | 11'-4" | 11'-11" | 12'-4" |
| 19 | 11'-11" | 13'-4" | 13'-9" |
| 18 | 12'-4" | 14'-7" | 14'-5" |
| 16 | 13'-0" | 16'-2" | 15'-2" |

Maximum Unshored Span based on:

| | | | | | |
|---|---|---|---|---|---|
| Uniform Construction Load | 20.00 | psf | Minimum End Bearing | 2.00 | in. |
| Concentrated Construction Load | 150.00 | plf | Minimum Interior Bearing | 4.00 | in. |
| Concrete Ponding Allowance | 3.00 | psf | Maximum Deflection L/ | 180 | ≤ 0.75 in. |
| Concrete Volume | 1.23 | $yd^3$ / 100 $ft^2$ (Note: Does not include allowance for ponding) | | | |

Composite Steel Deck Properties (steel deck only)

| Gage | Fy ksi | wdd psf | Se+ $in.^3/ft$ | Se- $in.^3/ft$ | Id+ $in.^4/ft$ | Id- $in.^4/ft$ | Vn/Ω kip/ft |
|---|---|---|---|---|---|---|---|
| 22 | 50 | 1.70 | 0.387 | 0.410 | 0.732 | 0.737 | 1.406 |
| 20 | 50 | 2.10 | 0.512 | 0.539 | 0.919 | 0.921 | 2.485 |
| 19 | 50 | 2.40 | 0.639 | 0.669 | 1.099 | 1.101 | 3.390 |
| 18 | 50 | 2.70 | 0.761 | 0.794 | 1.253 | 1.253 | 4.818 |
| 16 | 50 | 3.50 | 1.013 | 1.013 | 1.580 | 1.580 | 6.126 |

Tables generated using calculator V2.0 based on ANSI/SDI C-2017 in accordance with 2018 IBC Section 2210.

Date: 8/21/2019

NOTICE: Design defects that could cause injury or death may result from relying on the information in this document without independent verification by a qualified professional. The information in this document is provided "AS IS". Nucor Corporation and its affiliates expressly disclaim: (i) any and all representations, warranties and conditions and (ii) all liability arising out of or related to this document and the information in it.

Page 1 of 1

**Fig. 3.2.1 Vulcraft Unshored Span Calculator**

### Example 3.2.2 Composite Floor Slab with a Concentrated Load

Determine if a concentrated dead load of 1,000 lbs/ft plus a live load of 80 psf can be safely supported by the 3VLI20 deck described in Example 3.2.1. Assume the concentrated load can be located at any position on the slab. The concentrated load rests on a 4-inch square bearing plate. Solve using LRFD.

**Given:**

$b_2 = b_3 = 4.0$ in.

$t_c = 3.25$ in.

$t_t = 0$ in.

$Span = 10$ feet.

$h = 3.25 + 3.0 = 6.25$ in.

**Solution:**

Determine the effective slab width for flexure and shear.

For flexure:

$b_m = b_2 + 2t_c + 2t_t$

$b_m = 4$ in. $+ 2(3.25$ in.$) = 10.5$ in.

The most critical location for moment calculations is to place the load at the center of the span.

$b_e = b_m + 2(1.0\text{-}x/L)x \leq 106.8(t_c/h)$ for single span bending (no reinforcing)

$b_e = 10.5$ in. $+ 2(1.0\text{-}5.0$ ft$/L)x \leq 106.8(t_c/h)$

$x = L/2 = 60$ in.

$b_e = 10.5$ in. $+ 2(1.0 - 60.0$ in.$/120$ in.$)(60.0$ in.$) = 70.5$ in.

$b_{e(max)} = 106.8(t_c/h)$

$b_{e(max)} = 106.8(3.25$ in.$/6.25$ in.$) = 55.5$ in. $= 4.63$ feet. (controls)

For shear:

Place the load the slab depth away from the support ($x = h$) = 6.25 in.

$b_{ev} = b_m + (1.0\text{-}x/L)x, \leq 106.8(t_c/h)$

$=10.5$ in. $+ (1.0 - 6.25$ in.$/120$ in.$)(6.25$ in.$) = 16.4$ in.

Determine Design Shear Strength:

Unfactored uniform load based on P:

$w_{pv} = P/b_e = 1{,}000$ lbs/16.4 in./12 in./ft = 732 plf

Total factored shear:

$V_u = (1.2w_{DL} + 1.6w_L)L/2 + 1.2w_{pv}(120$ in. – 6.25 in.)/120 in.

$w_{DL} = w_{concrete} + w_{deck} = 48$ psf (based on Vulcraft Unshored Span Calculator of 47.615 psf).

$V_u = [1.2(48$ plf$) + 1.6(80$ plf$)](10$ ft$)/2 + 1.2(732$ plf$)(120$ in. – 6.62 in.)/120 in. = 1,760 plf

$\phi V_{nt} = 5664$ plf
SDI Manual Table 8C

5,664 plf ≥ 1,760 plf **o.k.**

Determine the Design Bending Strength:

Unfactored Uniform Load based on $P$:

$W_{pb} = P/b_e$ = 1,000 lbs/(55.5 in./12 in./ft) = 216 plf

Total factored moment near mid-span (based on a simple beam)

$M_{pos(P)}$ = 0.125(1.2 $w_{DL}$ + 1.6$w_{LL}$)($L^2$) + 0.25(1.2$W_{pb}$)($L$)

= 0.125[(1.2)(48 plf) +1.6(80 plf)](10 ft)$^2$ + 0.25(1.2)(216 plf)(10 ft)

= 2,970 lbs-ft/ft.

= 35.6 kip-in./ft

$\phi M_y$ = 62.36 kip-in. = 5,200 lb-ft

SDI Manual Table 4D

5,200 ft-lbs ≥ 2,970 ft-lbs/ft **o.k**.

Determine required distributed steel:

Weak axis moment to be resisted at mid-span:

$w = L/2 + b_3 \leq L$ = 120 in./2 + 4.0 in. = 64.0 in. ≤ 120 in.

$b_e$ = 70.5 in.

$M_{weak\ axis} = P_u b_e/15\ w$ = 1.2(1,000 lbs)(70.5 in.)/[(15)(64 in.)] = 88.1 lb-in./ft = 1,060 lb-in./ft

Try 6x6xW2.1xW2.1 welded wire reinforcement.

$A_s$ = 0.042 in.$^2$/ft

$b$ = 12.0 in.

$d = t_c/2$ = 3.25 in./2 = 1.625 in.

$$a = \frac{f_y A_s}{\phi(b)\left(f_c'\right)} = \frac{(60\text{ ksi})\left(0.042\text{ in}^2/\text{ft}\right)}{0.85(12\text{ in.})(3\text{ ksi})} = 0.082\text{ in.}$$

$$\phi M_n = \phi f_y A_s\left(d - \frac{a}{2}\right) = 0.9(60\text{ ksi})\left(0.042\text{ in.}^2/\text{ft}\right)\left(1.625\text{ in.} - \frac{0.082\text{ in.}}{2}\right) = 3.59\text{ kip-in./ft}$$

= 3,590 in.-lbs/ft

1,060 lb-in./ft ≤ 3,590 lb-in./ft **o.k**.

Determine deflection:

$I_d$ = 11.85 in.$^4$/ft
SDI Table (3H)

$$\Delta = \frac{w_{pb}L^3}{48E_s I_d} = \frac{(216\text{ plf})(120\text{ in.})^3}{48(29{,}500{,}000\text{ psi})\left(11.85 \leq \text{in.}^4/\text{ft}\right)} = 0.022\text{ in.}$$

$$\frac{L}{\Delta} = \frac{120}{0.022} = 5{,}450\text{ o.k.}$$

Check Punching Shear:

$b_2 = b_3$ = 4.0 in.

$t_c = h_c$ = 3.25 in.

$f_c$' = 3000 psi

$$V_{pr} = \left(2 + \frac{4}{\beta_c}\right)\phi_v \sqrt{f_c^{'}} b_o h_c \leq 4\phi_v \sqrt{f_c^{'}} b_o h_c \qquad \text{SDI (2017b) Eq. (2.4.9a)}$$

$ß_c$ = ratio of long side to short side of concentrated load = 4.0 in./4.0 in. =1.0

ACI 318 indicates the effective shear perimeter need not approach closer than $t_c/2$ to the edge of the applied load.

$b_o = 2(b_2 + t_c) + 2(b_3 + t_c)$ = 2(4.0 in. + 3.25 in.) + 2(4.0 in. + 3.25 in.) = 29.0 in.

The available shear equals:

$$V_{pr} = \left(2 + \frac{4}{1.0}\right)0.75\sqrt{3000\text{ psi}}\,(29\text{ in.})(3.25\text{ in.}) \leq 4(0.75)\sqrt{3000\text{ psi}}\,(29\text{ in.})(3.25\text{ in.})$$

$$17{,}400\text{ lbs} \leq 11{,}600\text{ lbs}$$

$$V_{pr} = 11{,}600\text{ lbs}$$

1.2$P$ = 1.2(1000 lbs) = 1,600 lbs ≤ 11,600 lbs **o.k.**

The SDI Design Standard for Composite Steel Floor Deck-Slabs requires that bearing lengths be determined in accordance with the AISI Specification. The SDI Design Standard requires a uniform load of wet concrete plus dead weight plus 20 psf construction load be used for this calculation.

### Concrete for Slabs on Steel Deck

The concrete used on steel deck is structural concrete. The minimum 28-day compressive strength required by the SDI Specification is 3000 psi. This concrete is available in a range of densities depending on the aggregate used. The range is from 145 pcf (normal weight) to 110 pcf (structural lightweight concrete). Normal weight concrete is most commonly used. However, the lesser density of structural lightweight concrete is often used for an advantage in fire rated assemblies because in rated assemblies it can result in lighter overall slabs. Also, the elimination of the need for fireproofing on the under-side of deck often justifies the greater unit cost for structural light weight concrete. The minimum thickness of concrete over the deck flutes is given as 2 in. by SDI. A greater thickness may be required as part of a fire rated assembly or may be required to increase the mass and transverse stiffness of a floor system to control vibration.

Concrete on steel deck requires a complete system of concrete stops, flute closures, trim pieces and sheet material around columns. The drawings and specifications should indicate if this material is part of the deck work, concrete work or sheet metal work.

Cast-in-place slabs on steel deck form excellent floor diaphragms. Diaphragm values for both concrete on form deck and composite slabs in various combinations of concrete properties and steel deck profiles and gages are given in the Vulcraft Manual “Steel Floor and Roof Deck.”

### Precast Slabs

The precast slabs discussed in Section 2.3 of Chapter 2 can also be used in floor construction. These slabs are available in three configurations. Channel slabs, hollow core slabs and solid tongue and groove edge planks. Manufacturer’s load tables should be consulted for spans and load capacities. The attachment of these decks to the joists and the diaphragm capabilities are discussed in Chapter 2.

### Wood Decks

Plywood and wood plank decks were also discussed in Section 2.3 of Chapter 2. These decks are also used in floor construction. As cited in Chapter 2 the plywood identification for floors has associated with it a super imposed load of 160 psf for floors. Common spans for plywood decking are 16 to 24 inches. Wood planks can be used at greater spans in the range of 36 to 48 inches. These decks are either attached directly to the joists or by means of continuous nailers as was discussed in the section on roof decks. Their capability as diaphragms was also discussed there.

## 3.3 FLOOR LOADING

Floor loadings can be divided into three principal categories: self-weight, collateral dead loads and live loads.

### Self-weight

Self-weight of the floor system consists of the weight of concrete, steel deck and framing. The weight of the cured concrete slabs and steel deck combinations are given in the Vulcraft Manuals. The self-weight of framing must be computed on a job by job basis.

### Collateral Dead Loads

Suspended collateral loads are the same as was presented in the chapter on roofs (see Chapter 2 Section 2.4). There is however, one additional superimposed load on floors: the weight of partitions. The loading for partitions is prescribed by the Building Code and by the engineer's judgment for the intended use of the building.

Partitions running parallel with joists create a continuous line load on the deck. Alternately, partitions running perpendicular to the joists create a concentrated load on the joists. Codes used to specify that the joists be doubled under partitions. Such requirements have been dropped due to the need for flexibility in partition arrangement and rearrangement. The provision of a blanket uniform load to account for partitions is generally all that must be done for standard drywall partitions of normal height (eight to twelve feet). Masonry walls for example at stairs and elevators should be specifically accounted for in the design of their supporting members.

### Live Loads

Minimum design live loads are specified by building codes. These loads are given as blanket loads in pounds per square foot. Some codes also give concentrated loads along with an area of application. Code specified loads vary with the classification of use of the structure. Building owners may at times require design live loads in excess of the code specified minimums. Alternately the owner may have in mind special uses not anticipated by the code. It is thus important for the designer to review the live load requirements before embarking on the design. The design live loads should be tabulated on the drawing for future reference. Special treatment is usually required for loads resulting from equipment, storage racks, files, libraries, safes and moving concentrated loads from pallet lifts and fork trucks. These loads may not only require greater than normal flexural strength but also shear strength. In the case of concentrated loads shear may govern the design when the load is placed near the support. The shear strength of composite slabs with steel deck should be carefully verified because the load in question may differ from those anticipated in the calculations and load tests which were used in the development of the deck load tables.

### Live Load Reductions

The IBC provides for the probabilistic expectations of full live load by means of live load reductions. The amount of reduction depends on the nature of the load and the tributary area of the element under consideration. The applicable building code procedures should be followed as appropriate.

## 3.4 SERVICEABILITY CONSIDERATIONS

Serviceability considerations are related to the function of the building and its components. It is generally a function of stiffness rather than strength. In floor design the primary issues are control of deflection and control of vibrations.

### Control of Deflections

What follows is a brief review of deflection limits from various sources for steel deck and joists.

Steel Deck Institute:

a. Span over 180, not more than 3/4", uniformly distributed weight of wet concrete and weight of metal form deck

b. Span over 180, not more than 3/4", uniformly distributed weight of wet concrete and weight of composite steel deck as a form

c. Span over 360 for superimposed load on composite steel deck

Steel Joist Institute:

a. Span over 360, live load for floors

b. Span over 360, where a plastered ceiling is attached or suspended

c. Span over 240, for all other cases

### Other Deflection Considerations

Partitions and ceilings require detailing consistent with the deflections which occur after their installation. Partitions are supported by the floor and must be able to follow the floor's deflected curve without distress. This distress would most likely occur as a diagonal crack at the upper corners of doorway openings. The potential for distress is most directly addressed with control joints at openings and at intervals of long uninterrupted walls. The spacing of such joints is suggested to be 30 feet or closer. Other studies have suggested panel height to length ratios of 1:2 or 1:3.

The tops of partitions, when run to the underside of the next floor or to the underside of roof, should be slip jointed to provide lateral stability and to prevent inadvertent transfer of load from one level to another. Details of this sort usually provide a range of movement between 3/8 to 1 inch. This range of movement should be matched to the expected deflections.

The deflection limit of span over 360 is a well-established criterion for the performance of plaster and other ceilings. It should be noted however, that this limit may allow greater deflection than can be accommodated in a rigid joint between wall and ceiling, especially when there are ceilings supported by long bays with abutting partitions at mid-span. The relative movement between walls and ceilings consisting of acoustical panels in a metal grid is most easily accommodated with this construction. The relative movement between wall and ceiling which can be accommodated is in the range of ¼ to ½ inch.

The deflection of supporting members during concreting operations is of concern because it affects the performance of the concreting crew and may also result in unanticipated dead loads. American Concrete Institute (ACI, 2014) requirements for form deflection are not written with steel deck and joists in mind. The deflections given are stricter than is usually the practice in buildings framed with steel. It is recommended in AISC Design Guide No. 3 (Fisher J.M., West M.A., 2019), that framing members be held to a maximum deflection of span over 360 (1 in. max.) for the weight of wet concrete and framing. This should be the maximum accumulated deflection in the bay. The concrete contractor must be prepared for this deflection and must anticipate the need for the proper volume requirement to fill the deflected curve.

### Vibrations

The control of vibrations is a special topic and covered in Chapter 5.

### Expansion Joints

Cast-in-place concrete slabs on steel deck should have expansion joints at intervals of 200 to 250 feet on center. This range is stricter than would be recommended by the Federal Construction Council as cited Chapter 2, Section 2.5, but recognizes the fact that the slab is thinner and less heavily reinforced than the cast-in-place concrete structures upon which the Federal Construction Council recommendations are based. It is also based on practical experience.

## 3.5 FRAMING CONSIDERATIONS

### Bay Size

For most multistory buildings little if any options exist for the engineer to select the optimum bay size. Architectural requirements and building footprint usually dictate the bay size. Certainly, larger bay sizes are more favorable for steel systems as compared to poured in place concrete systems. The 30'x30' bay size is very common for multistory structures. It is an economical bay for joist and joist girder framing. The 30' bay is also economical relative to spandrel systems. If the building cladding system is to be supported from a spandrel member, deflection requirements will generally dictate the size of the spandrel, thus the spandrels become increasingly more expensive with span length. If perimeter bays larger than 30 feet are used, it is often economical to add intermediate columns around the perimeter of the building to save costs. In bays with composite girders the bays should be set at even foot increments so that the deck flutes can be laid out with a flute over the girder which will allow the installation of shear connections without special cutting of the deck.

### SJI Floor Bay Design Tool

The SJI provides a "Design Tool" to assist the specifying professional on optimizing floor bay size. The tool is entitled, "Floor Bay Analysis Tool." The tool can be downloaded free of charge from the SJI Website, www.steeljoist.org, under the tab "Design Tools." The user can input various scenarios to arrive at the least weight or the least cost bay size. Cost data can be input by the user along with other design data. Bays can be evaluated using either ASD or LRFD. Pull down menus allow for easy selection of steel deck, joist type (K, LH, DLH and CJ-Series) and Joist Girder selections.

### Joist Span Direction

For floor systems, it is almost always more economical to span the joists in the long direction of framing. Since the joists sit on top of the girder, they can be made deeper than the Joist Girder (by the amount of the seat depth) without infringing upon the clear height requirements.

### Joist Spacing

Experience has shown that wide joist spacing provides very economical floor systems. In fact, the widest spacing for a given deck profile and slab thickness should always be used. The wider joist spacing provides several advantages over joists spaced 2'-0″ o.c. Typically erection costs are less and the wider joist spacing provides a floor system with better vibration characteristics. The joists are deeper thus allowing larger penetrations through their web openings.

### Seat Depths

Per SJI Table 5.4-3, standard bearing seat depths range from 2-½" for K-series to 5" for LH- and light DLH-series to 7-½" for very-heavy LH- and heavy DLH-series and Joist Girders. However, please note that it is possible to specify a special minimum bearing seat depth (BSD) in order to reduce the impact of this depth on the height of the structure. Per SJI Table 5.4-3, the BSD may also be determined as BSD = 0.6 x (RP + D). D is defined as 2.5 (K-series) or 4 (LH-series). RP is defined as the horizontal distance from the joist end reaction point to the face of the wall or edge of support member (see Figure 3.5.1). Please refer to the Economical Joist Guide in the Vulcraft Manual (Vulcraft 2017c) using the specific span and loading situation to determine which series is likely the most economical selection.

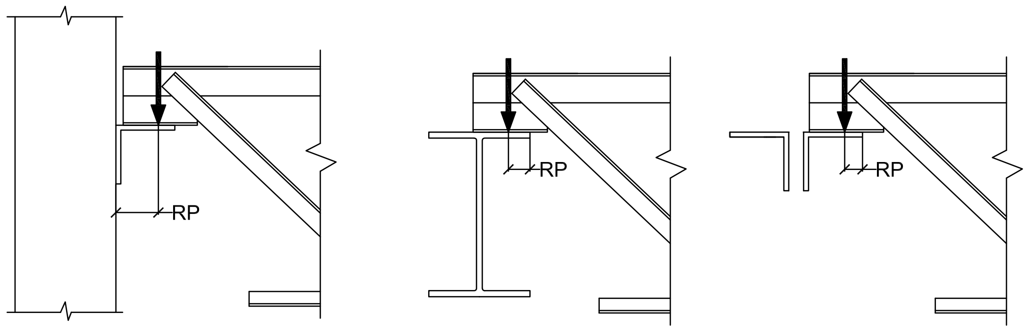



**Fig. 3.5.1 Definition of 'RP' dimension from SJI Table 5.4-3**

If a joist requires a sloped seat (i.e., a seat whose bearing surface is not parallel with the top chord), standard bearing seat depths will likely not be sufficient. Also, the seat depth will differ from one end of the joist to the other. In addition, if top-chord extensions are also required, this will further affect the seat depth. Guidance as to how to determine the bearing seat depths for a sloped joist can be found in the Vulcraft Manual in the section entitled 'Sloped Seat Depths'.

Vulcraft can provide a flush-framed connection at the ends of the most floor joist series (except for K-series) and Joist Girders using flush-shear plates which attach to shear tabs on the girder as shown in Figures 3.5.2a thru 3.5.2c. Having the top of the floor joist be flush with the top of the girder reduces overall floor-to-floor height, facilitates the easy installation of welded shear studs on the girder top flange, and permits the girder to be designed with composite shear studs, if desired.

The **joist** flush-shear plates are designed and detailed by Vulcraft using 1 in. thick plates with standard short slot holes (1-1⁄16" x 1-5⁄16") for 1 in. diameter A325 bolts. Vulcraft will design and resolve within the joists, the effects of the end moments created by the eccentricities shown in Figure 3.5.2. The angle braces shown in Figure 3.5.2a are to brace the wide flange girder for torsion due to unbalanced loading or eccentricity of the load. The angle braces can be omitted from the detail if the EOR determines the girder does not have issues due to torsion. The EOR is responsible for the design of the **beam** shear plates and the required number of 1 in. diameter bolts shown in Options A, B, and C in Figures 3.5.2a thru 3.5.2c. The bolts must be designed by the EOR to resist the joist end shear and bending from the end shear times the eccentricities shown.

The connection shown in Fig.3.5.2d will be designed by Vulcraft. The bottom chord braces will be supplied loose.

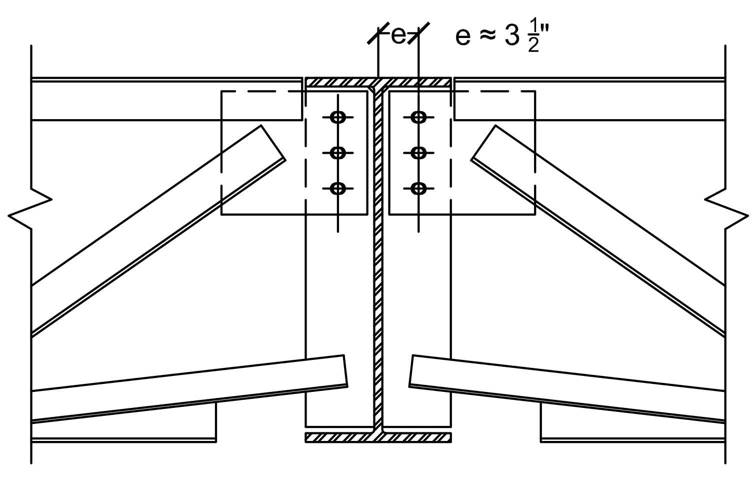



**Fig. 3.5.2a Option A- Partial Depth Joist Flush Shear Plate**

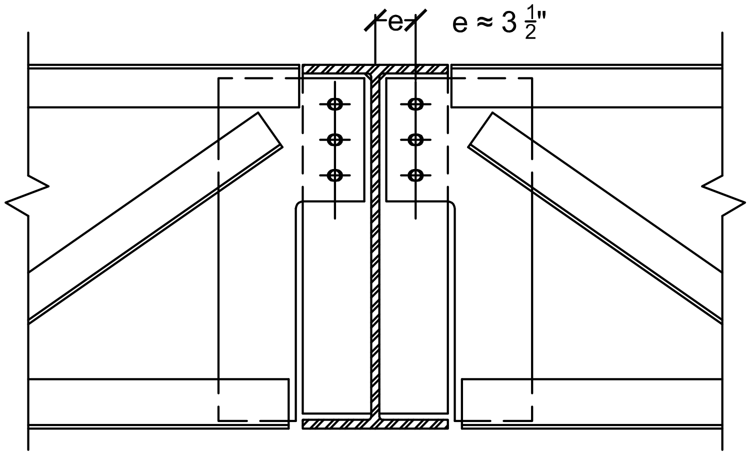



Fig. 3.5.2b Option B- Full Depth Coped Joist Flush Shear Plate

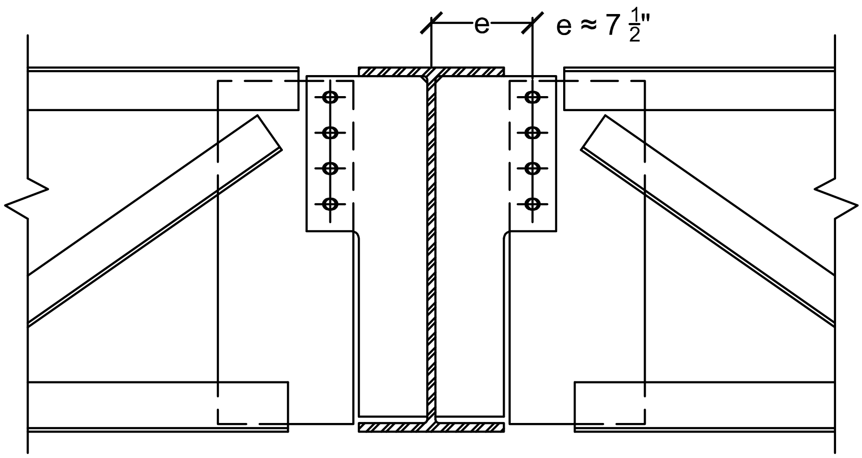



Fig. 3.5.2c Option C- Full Depth Rectangular Joist Flush Shear Plate

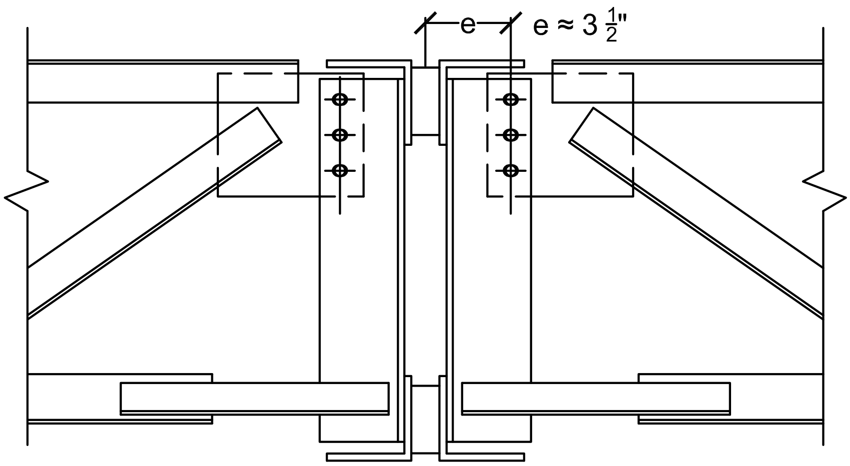



Fig. 3.5.2d Option D- Partial Depth Joist Flush Shear Plate

### Framing System Depth and Story Height

Provided in Table 3.5.1 are estimated depths and weights of framing for various bay sizes and supported loads for planning purposes. The total floor to floor distance can sometimes be reduced when mechanicals are run in the joist spaces.

**WEIGHT TABLE**

| Joist and Girder Information | | | | | Total Weight of Joists, Girders and Bridging (psf) for Loads Shown | | | |
|---|---|---|---|---|---|---|---|---|
| Joist Span (ft) | Girder Span (ft) | Joist Spacing (in.) | Joist Depth (in.) | Girder Depth (in.) | 120 (psf) | 130 (psf) | 140 (psf) | 150 (psf) |
| 20 | 20 | 6′–8″ | 24 | 20 | 3.0 | 3.1 | 3.3 | 3.5 |
| 30 | 20 | 6′–8″ | 24 | 20 | 3.6 | 4.0 | 4.2 | 4.5 |
| 30 | 20 | 10′–0″ | 24 | 20 | 3.6 | 3.9 | 4.1 | 4.5 |
| 30 | 30 | 7′–6″ | 28 | 24 | 4.5 | 5.0 | 5.1 | 5.7 |
| 30 | 30 | 10′–0″ | 28 | 24 | 4.5 | 4.7 | 5.1 | 5.3 |
| 35 | 30 | 7′–6″ | 32 | 28 | 4.6 | 4.7 | 5.2 | 5.6 |
| 35 | 30 | 10′–0″ | 32 | 28 | 4.6 | 4.8 | 5.1 | 5.3 |
| 35 | 35 | 7′–0″ | 36 | 32 | 4.9 | 5.2 | 5.4 | 5.9 |
| 35 | 35 | 11′–0″ | 36 | 32 | 4.8 | 5.1 | 5.4 | 5.8 |
| 40 | 30 | 7′–6, | 32 | 28 | 5.1 | 5.3 | 5.9 | 6.4 |
| 40 | 30 | 10′–0″ | 32 | 28 | 4.9 | 5.3 | 5.7 | 6.3 |
| 40 | 35 | 7′–0″ | 36 | 32 | 5.4 | 5.8 | 6.0 | 6.6 |
| 40 | 35 | 11′–8″ | 36 | 32 | 5.3 | 5.7 | 6.1 | 6.5 |
| 40 | 40 | 8′–0″ | 40 | 36 | 5.8 | 6.1 | 6.6 | 7.3 |
| 40 | 40 | 10′–0″ | 40 | 36 | 5.6 | 5.9 | 6.2 | 7.2 |
| 40 | 40 | 13′–4″ | 40 | 36 | 5.6 | 6.2 | 6.5 | 7.2 |

The above table can be utilized as a guide to estimate the dead load of the joists, Joist Girders, and bridging in the initial design phase

**Table 3.5.1 Framing Depths and Weights**

## ALLOWABLE DUCTWORK

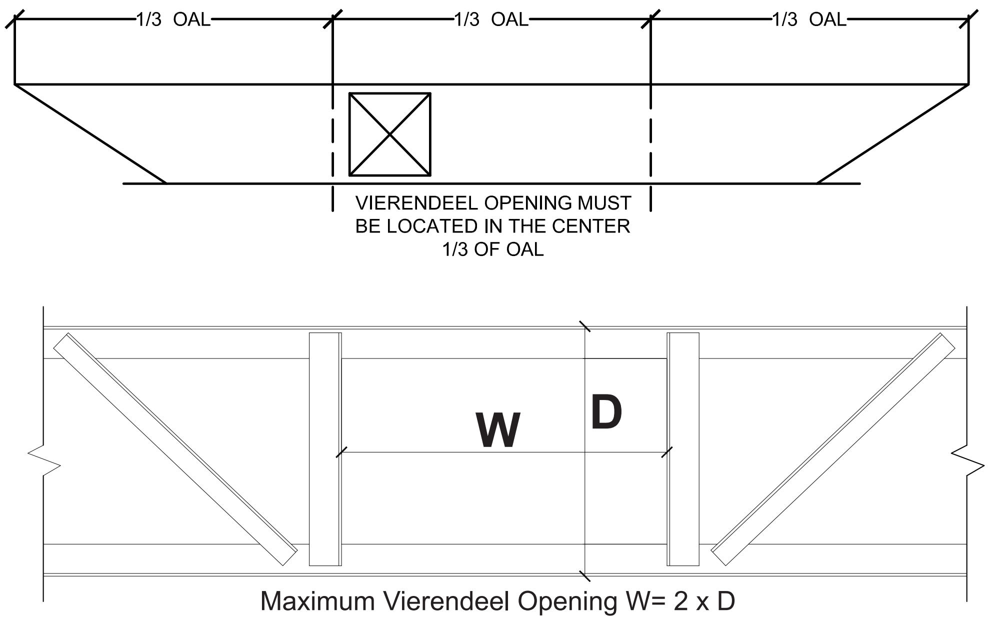
| Joist Depth (in.) | Panel Length (in.) | Maximum Span (ft.) | Round (in.) | Square (in.) | Rectangular (in. x in). | Flat Oval (in. x in) |
|---|---|---|---|---|---|---|
| 18 | 48 | 22 | 11.0 | 9.25 | 6.0 x 18.25 | 20.50x 7.50 |
| 20 | 48 | 25 | 12.5 | 10.25 | 7.0 x 18.75 | 21.25x 8.75 |
| 22 | 48 | 26 | 14.0 | 11.25 | 8.0 x 19.25 | 21.75x10.00 |
| 24 | 48 | 32 | 14.5 | 12.0 | 8.75 x 19.0 | 22.00x10.75 |
| 26 | 56 | 38 | 16.0 | 12.75 | 9.5 x 19.25 | 25.50x11.75 |
| 28 | 56 | 45 | 15.5 | 12.75 | 9.75 x 18.5 | 25.00x12.25 |
| 30 | 64 | 45 | 17.5 | 14.25 | 11.0 x 19.5 | 30.00x14.00 |
| 32 | 64 | 50 | 19.5 | 15.75 | 11.5 x 25.25 | 29.50x14.50 |
| 34 | 78 | 52 | 21.5 | 17.5 | 12.75 x 28.0 | 36.00x15.75 |
| 36 | 78 | 56 | 22.5 | 18.25 | 13.25 x 29.25 | 36.75x17.00 |
| 38 | 86 | 60 | 23.5 | 19.0 | 13.75 x 30.75 | 40.75x18.00 |
| 40 | 86 | 60 | 25.0 | 20.25 | 14.75 x 32.5 | 41.25x19.25 |
| 42 | 96 | 60 | 27.5 | 22.25 | 16.25 x 35.5 | 45.50x20.25 |
| 44 | 96 | 60 | 29.0 | 23.75 | 17.75 x 37.5 | 46.25x21.50 |
| 46 | 82 | 60 | 31.0 | 25.0 | 18.25 x 39.5 | 40.50x23.00 |
| 48 | 82 | 60 | 32.5 | 26.5 | 19.5 x 41.5 | 40.75x24.25 |
| 50 | 100 | 60 | 35.0 | 28.5 | 21.0 x 44.45 | 50.50x26.00 |

**Table 3.5.2 Maximum Allowable Ductwork Size for Joist Without Fireproofing or Insulation**

Footnotes to Table 3.5.2

1. The ductwork table represents the largest ductwork shapes that can be accommodated by floor joists for each given depth based on a 2000 plf loading condition and the maximum span shown. If the span is less than the maximum shown, or the loading is less than 2000 plf, it may be possible to accommodate larger ductwork. Contact Vulcraft with your condition if the dimensions given in the table are inadequate.
2. The sizes in the above table represent the maximum duct sizes that will pass through an interior panel in the center one-third of the joist. If the duct falls at either end of the joist, allowable duct sizes may be reduced due to larger web sizes. If the duct is wrapped in insulation, dimension from outside to outside of insulation should be compared to the table.
3. No allowance has been made for fireproofing and/or insulation on the web members. If either is present, the size of the ductwork must be decreased accordingly.
4. To ensure the ductwork will satisfactorily pass through the joists, be sure to specify the minimum panel size listed in Table 3.5.2 on the structural drawings if the actual duct sizes approach those given. Note that Vulcraft reserves the right to furnish panels larger than those shown in Table 3.5.2.
5. Table 3.5.2 does not apply to Joist Girders because of the wide variability of panel lengths possible due to the flexibility of Joist Girders to meet project requirements.

If large HVAC ducts are being run perpendicular to the joists, Vulcraft can provide a Vierendeel opening which must occur within the center third of the overall length (OAL) of the joist. These Vierendeel openings permit larger ducts to be run through the steel joist, thereby reducing the required floor to floor height. It is typical for the maximum width of the Vierendeel opening to be no greater than two times the steel joist depth. See Figure 3.5.3.




Maximum Vierendeel Opening W= 2 x D

**Figure 3.5.3 Vierendeel Openings**

### Composite Joists (CJ-Series)

It is possible to use joists in floor systems which are designed to act compositely with the floor slab. The employment of composite behavior has all the advantages of composite construction using wide flange members plus the advantages of joist construction.

The advantages of joist construction are:

1. Simplified fabrication and erection due to end bearing seats
2. Lighter overall structure weight
3. Open webs for mechanicals

Additional advantages when using composite joist construction:

1. Shallower depth joists, which allow shorter story heights
2. Lighter joists due to reduced top chord size
3. Fewer pieces due to wider spacing of joists with thicker slabs
4. Longer spans than comparable depth non-composite construction

The decision as to whether or not to use composite joists should include consideration of:

1. The potential for dead load deflection of the un-shored non-composite section
2. Floor vibration considerations
3. The added cost of the shear connectors required for composite action

Because the use of composite joists requires custom design of composite joists, it is important for the specifying professional to contact Vulcraft during the planning stages to establish the design limitations appropriate to the efficient use of composite joists. It will be necessary for the designer and Vulcraft to develop a proper means for specifying the composite joists for subsequent design and fabrication. The SJI "Standard Specification Composite Steel Joists Catalog" (SJI, 2019) contains a section on the "Responsibility of the specifying professional." Since Vulcraft is designing the CJ-Series joists, communication between the specifying professional and Vulcraft is extremely important. To do this, basic design information must be provided to Vulcraft. The following list summarizes the needed information:

1. Joist Depth
2. Joist Span
3. Adjacent Member Spacing
4. Type of Floor Deck
5. Concrete Unit Weight
6. Concrete Compressive Strength
7. Slab Thickness above Floor Deck
8. Composite Design Loads
   a) Noncomposite DL
   b) Construction LL
   c) Composite LL
   d) Composite DL
9. Camber

A "Composite Joist Floor Design Parameters Checklist" can be found in the "Code of Standard Practice for Composite Steel Joists." A form for filling in the above information is provided.

Prior to contacting Vulcraft the specifying professional is encouraged to conduct a preliminary design for the composite joists. As mentioned earlier this can be done by downloading and running the SJI Tool entitled, "Floor Bay Analysis Tool," at www.steeljoist.org. The tool can provide excellent information as to the least weight and most cost effective floor system by iterating several parameters such as joist depth, joist spacing, bay size and spans, etc.

## ECOSPAN® Composite Floor System

The ECOSPAN Composite Floor System is a lightweight floor system comprised of Vulcraft's open web steel composite joists, form deck or 1.5 inch composite deck. The unique feature of the floor system is the self-drilling and self-taping Shearflex® Screws which fasten the deck to the joists. The Shearflex® screws act as shear studs providing the composite action between the concrete and the joists. A special Shearset® Tool is used to install the screws. The concrete slab design is the responsibility of the specifying professional. Slab reinforcement may consist of rebar, welded wire fabric. The reader is referred to the ECOSPAN® "Design Manual" (Vulcraft, 2016a) for further information on the system.

## 3.6 SHEAR CONNECTORS

Composite behavior between steel framing members (such as composite joists) and the supported concrete slab on steel deck is created by the attachment of shear connectors to the framing members. These shear connectors project into the slab and are anchored into it when the concrete hardens. The requirements for composite construction are given in Chapter I of the AISC Specification. Studs are attached using proprietary methods which are designed to burn through the decking to weld themselves on the framing member below. One problem in the installation of studs results from snow/water on the top surface of the steel deck or water accumulating between the deck and the framing members. It is desirable to avoid welding shear connectors through two steel deck thicknesses where possible. AWS D1.1, Section 7 Shear Studs, outlines stud qualification testing required by the shear stud manufacturer and pre-production qualification testing required on the jobsite at the start of each shift. The attachment of the concrete slabs to the framing members with shear connectors can be a substitute for welding the deck to the framing. However, welds must still be supplied to anchor the deck panels prior to the installation of the shear connectors.

The specification of the size, number and arrangement of shear connectors is done as part of the building structural design. The arrangement of shear connectors must in general reflect the shear diagram of the composite member. Thus, each side of the point of zero shear receives shear connectors and the spacing is uniform except as governed by the AISC Specification equations which account for concentrated loads.

It is possible to design for either full or partial composite action. For full composite action, the size and number of studs is selected to resist a horizontal shear based on the lesser force of either the fully stressed concrete slab or the fully stressed steel section. For partial composite action, only the horizontal shear required to develop the portion of slab needed to resist loads is provided. Once the magnitude of shear force is determined, the size and number of shear connectors can be selected using the AISC Specification. The shear connector values must be calculated based on the specified concrete compressive strength. The shear connector values must also account for the deck profile and number of studs per flute. The requirements regarding this condition are set forth in the AISC Specification.

The required deck dimensions are provided in figures in the Vulcraft deck manual. In the manual the maximum and minimum flute widths are given so that the average may be computed. The selection of the type of shear connectors is the responsibility of the specifying professional. Vulcraft will prepare composite joist designs outlining the size, spacing and quantity of shear studs for each CJ-Series joist. If requested by the specifying professional, Vulcraft will provide design calculations for the CJ-Series joists with a cover letter bearing the seal and signature of Vulcraft's registered specifying professional.

The construction documents should also state that a revision of stud selection must be made if the final deck profile differs from the one anticipated in the design. Changes in deck gauge for a given deck profile have negligible effect on the shear capacity of the shear connector.

## 3.7 CONNECTIONS

Two connection conditions are unique to floors as compared to roofs. First, because columns are continuous to the roof joist, the floor Joist Girders must frame into both sides of the columns. This is accomplished with standard bracketed seats such as illustrated in Part 10 of AISC Steel Construction Manual. When joists frame into the web of the wide flange columns, care must be taken to select a deep enough column so that the seat width can be accommodated in the inter-flange dimension. Also, the deck must be supported over the Joist Girder seat and around the column.

Another connection concern involves headers around large openings such as for stairs. Often a header must be provided to support joists with this header in turn supported by a joist or girder. This situation frequently requires the use of wide flange headers which must be supported by Joist Girders. This requires a specially designed seat connection so that the header can be supported on the girders as if it were a joist, i.e. a shallow bearing seat be used. The design of this seat is discussed in Chapter 5.

The SJI provides "Design Tools" to assist the specifying professional for designing Joist Girder floor connections. These tools can assist the specifying professional by making the design process more timely and complete. Six different spreadsheets to assist in the design of moment connections are provided for free download from the SJI Website, www.steeljoist.org under the tab "Design Tools." Each can be used to calculate connection strength based on the necessary limit states. A reference manual is provided with each spreadsheet, explaining the calculations. Each spreadsheet provides for the design of a Joist Girder framing into one side or both sides of the column. The six connection Spreadsheets are:

1. Connection to the Strong Axis of Wide Flange Columns
2. Connection to the Strong Axis of Wide Flange Columns- Intermediate Levels
3. Connection to the Weak Axis of Wide Flange Columns
4. Connection to HSS Columns- Top Plate
5. Connection to HSS Columns- Knife Plate
6. Connection to Wide Flange Columns - Knife Plates

Although the Spreadsheets are specifically written for the design of moment connections, they can also be used for cases where Joist Girder additional top and bottom chord axial load transfer is required, or for seat design. The tools can be used for either ASD or LRFD.

# Chapter 4

# Lateral Load Systems

## 4.1 INTRODUCTION

In this chapter the various means of providing lateral force resistance systems for single story and multistory joist and Joist Girder buildings are presented. There are several lateral force resisting systems available to the structural engineer for both horizontal and vertical loading. The best combination of systems to use for a given project is dependent upon several variables. These include building geometry, number of stories, end use, roofing types, vertical loading requirements, lateral loading type and magnitude of the lateral loading. Wind, seismic and earth pressures are the most common lateral loading that affect the design. Depending on the building's height, weight and location, the IBC and ASCE7 may place restrictions on the type of vertical lateral force resisting system that may be used for the building.

## 4.2 DIAPHRAGMS

### Introduction

Roof and floor diaphragms have been utilized in the design of structures for many years. The primary purpose of the diaphragm is to transfer horizontal shear forces to the vertical force resisting system in the structure. A common analogy is that the diaphragm is a deep horizontal plate girder. The decking materials are similar to the web of the plate girder in that they carry the shear forces. Just as the stiffeners in a plate girder prevent the web from buckling, the major deck corrugations, joists, etc. provide the buckling resistance for the diaphragm, the flanges of the plate girder carry the flexural stresses. Similarly, diaphragms must also have continuous flanges at their perimeters to resist the flexural forces.

### Diaphragm Types

Diaphragms can be classified based upon the type of materials used to comprise them. Materials commonly used with joist and Joist Girder buildings include:

1. Steel deck
2. Steel decks in combination with insulating fills
3. Concrete slabs on steel form deck
4. Composite steel decks with lightweight concrete
5. Composite steel decks with normal weight concrete
6. Wood diaphragms

The strength and stiffness of a diaphragm system is controlled predominantly by:

1. The deck configuration, i.e. the height of the major corrugations and spacing of corrugations within the panel
2. The span of the individual deck panels
3. The material thickness and strength
4. The type and arrangement of fasteners (specifically, the spacing of fasteners to the structural elements and the side lap connectors)
5. The type and amount of concrete fill, if any

## Fastening of Steel Decks

A variety of fastener types and patterns are available for connection of the deck to the structural members, and for the deck to deck sidelaps. The choice of the fastener type not only depends upon the shear requirements, but also on the project requirements and the preferences of the construction team.

The fastening of the deck to the joists must as a minimum meet the SJI requirement for the joist chord lateral stability. This is given in the SJI Specifications (SJI, 2015a) and is 100 lbs. per foot for K-Series joists and ranges from 120 lbs. per foot to 520 lbs. per foot based on chord size for LH and DLH joists.

Welding is often used to connect the deck to the structural members. The Steel Deck Institute (SDI, 2017d) requires 5/8-inch arc spot welds (puddle welds) or a 3/8 x 3/4 in. elongated weld. The 3/8 x 3/4 in. elongated welds are required for A and F decks because the 5/8 in. arc spot welds cannot be made in the narrow rib of these decks. In the International Association of Plumbing and Mechanical Officials (IAPMO, 2018) Evaluation Reports, the arc spot weld size requirements vary somewhat between various manufacturers. Weld patterns vary depending upon shear requirements. For 36-inch-wide roof deck a minimum of three arc spot welds per sheet width are required for attachment to the structural members. This is equivalent to an 18-in. spacing. The maximum number of welds per 36-in. sheet width is 7, i.e. one every 6-inches. These patterns are commonly designated as 36/3 and 36/7. Other patterns are 36/4 and 36/5. The reader is referred to the Vulcraft Steel Roof and Floor Deck Manual (Vulcraft, 2018) and to Punchlok II® Roof Deck, Weld and Screw Support Connections (Vulcraft, 2016b) for a full description of fastener patterns.

A comment should be made regarding the welding of the deck to structural members using welding washers. The SDI (SDI, 2017d) AISI (AISI, 2016) and the AWS (AWS, 2015) do not require the use of welding washers for decks equal to or greater than 0.028-inches in thickness. Since 22 gage decks are approximately 0.0295-inches in thickness welding washers are not required. The SDI Specifications go further recommending against using welding washers for decks greater than 0.028-inches in thickness. Their research indicates a decrease in shear resistance if welding washers are used for these decks.

Mechanical fasteners such as power-driven fasteners and self-drilling screws are also used for the deck-to-structural attachment. The use of these fasteners is growing in popularity. Although these fasteners provide less shear capacity than welds they have several advantages. These include:

1. They are easy to install with little skill required
2. They provide a clean neat appearance since deck burn thru is eliminated
3. Quality control questions are basically eliminated since their strength is very dependable once they are correctly installed
4. Inspection costs are generally less than welded systems

The disadvantages of power-driven fasteners and self-drilling screws are:

1. Their shear capacities are less than those for welded deck
2. The installed cost can be higher compared to welding the deck (depending on building size)

Power driven fasteners and self-drilling screws have more attachment patterns that can be used as compared to those for welding. Heavier attachment patterns, depending on span and gage, can sometimes match the capacity of welds.

Sidelap connections are made by welding, Punchlok II system, button punching, mechanical or self-drilling screws. Vulcraft/Verco, and other manufacturers of steel deck, do not recommend the welding of sidelaps of nestable decks of 22 gage or less. Sidelap welds can be made on 22 gage interlocking decks. However, extreme care must be exercised even with proper welding equipment. The Punchlok II Tool provides a connection where the sidelap material is sheared and offset so the sheared surface of the male leg is visible in the cut. This provides a strong reliable connection for shear transfer. Button punching of sidelaps is probably the most unreliable method used. Extreme care must be taken to make sure that the upstanding leg of the deck is fully inserted into the upper portion of the deck. If it is not fully inserted, then the button punching operation will not connect the lap together properly. This is also a problem if the interlocking deck is to have sidelap welds. If the vertical leg is not inserted properly then the weld on the sidelap will not engage both pieces of deck.

## Fastening of Wood Systems

Wood deck diaphragms are attached to steel joists either directly by special self-drilling screws or by using wood nailers, which are generally attached to the joist with wood screws. The method of attachment should be specified on the plans so that the required holes in the top of the joist can be accounted for in the design of the joist. It is sometimes possible to attach wood nailers by bolting through the gap between the top chord angles. The use of this sort of detail requires coordination between the specifying professional and Vulcraft so that the bolt diameter and bolt spacing can be chosen to match the geometry of the joist. The bolt will be counter-sunk, and a washer will be placed under the top chord. The specifying professional must check for uplift pull-through and for bearing of the reduced wood thickness against the bolt shank. If the perimeter joist is to be used as a diaphragm chord, provision must be made for chord force transfer from the nailer to the joist. Bolting between the top chord angles is typically not preferred.

In general, the requirements for the inter-connection of wood diaphragm elements have been developed as wood to wood connections. The most common fastener is the nail. The use of wood nailers on joists allows the wood deck diaphragm to be constructed without variation from standard and codified procedures. If the sheathing is connected directly to the joist, the load path must be considered. The gap between the joist top chords is not designed to transfer horizontal forces between pieces of sheathing. The edges of the sheathing need to be detailed and installed so they both occur on the same joist top chord angle. If this is not practical, another means of transferring the load needs to be detailed and provided, for instance a light gage metal strap across the sheathing edges. When attaching wood decking directly to joists, care must be taken to ensure that the fasteners are equivalent to those used in the standard procedures and that they are acceptable to the building official. Where diaphragm strengths are taken from standard references, care must be given to maintain the patterns of support associated with the given capacity. In many cases, a series of relatively closely spaced joists is expected. Also, many capacities are dependent on blocked edges which requires a wood member on the sheathing panel edge which runs perpendicular to the joists.

## Load Tables

*Metal Systems:*

In the United States, considerable research on light gage steel deck diaphragm systems was conducted in the early 1960's. In 1967 the American Iron and Steel Institute published its first edition of design criteria for light gage steel shear diaphragms for use in roofs and walls. The criteria included information on steel and concrete diaphragms. The Steel Deck Institute published its first "Diaphragm Design Manual and Load Tables" in 1981. A 4th edition and expanded diaphragm manual (SDI, 2015) was published by the SDI in 2015. In addition to these organizations, various manufacturers of steel deck have conducted their own research and published diaphragm strength and stiffness values of their own.

In its 2016 North American Specification for the Design of Cold- Formed Steel Structural Members (AISI, 2016a), the American Iron and Steel Institute included recommended safety factors for light gage diaphragms. Basically, these safety factors agree with the International Association of Plumbing and Mechanical Officials (IAPMO, 2018) Evaluation Reports for welded diaphragms, and the Steel Deck Institute for mechanically fastened systems.

Currently, specifying professionals of steel deck systems rely on two sources for diaphragm values. These are the Steel Deck Institute and IAPMO. Both sources provide load tables with strength and stiffness/flexibility criteria. IAPMO diaphragm strength and flexibility values are used predominantly by specifying professionals on the West Coast. The Steel Deck Institute load tables are used throughout the remainder of the country. Some differences in the allowable strength and stiffness values will be apparent to the specifying professional when comparing the two different sets of load tables. This is since different researchers compiled the data and they used different empirical equations to establish diaphragm tables. The SDI Diaphragm Design Manual (SDI, 2015) contains detailed information regarding diaphragm behavior and presents examples illustrating the analysis and design diaphragm systems. The reader is encouraged to study this manual for a comprehensive treatment of steel deck diaphragms.

*Wood Systems:*

Wood diaphragms can be formed from plywood, Oriented Strand Board (OSB), timber decking, laminated timber decking and other board sheathing. Each of these types poses special design and detailing concerns which are beyond the scope of this document. The reader is directed to the following references for a detailed treatment of the design and detailing of wood diaphragms.

1. Timber Diaphragms: American Institute of Timber Construction, "Timber Construction Manual," Tigard, Oregon. (AITC, 2012)
2. "Diaphragms and Shear Walls, Design/Construction Guide": American Plywood Association, "Plywood Diaphragm Construction," V310, Tacoma, Washington. (APA, 2007)
3. "Special Design Provisions for Wind & Seismic (SDPWS)," (AWC, 2015)

The remainder of this chapter is devoted solely to diaphragm design using steel decks.

## Diaphragm Design Procedure

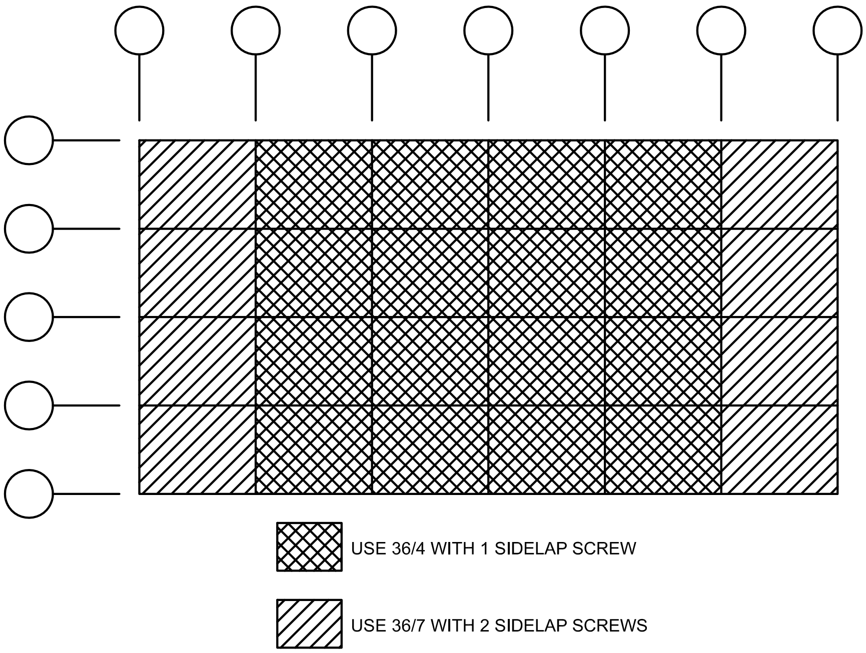
The specifying professional can control the strength and stiffness of the diaphragm by the selection of:

1. The deck thicknesses
2. The type of fastening to the structure
3. The frequency of fastening to the structure
4. The type of fastener used to connect the sidelaps of the deck together
5. The frequency of the sidelap fasteners
6. The spacing of structural members

In addition to the above items the stiffness of deck systems without concrete infill are controlled by the deck type, i.e. A, F, N, 2D, 3.5D, N32 or B.

For most situations, the thickness of the deck and the spacing of the structural members are determined by the gravity load design. The thickness can be increased over gravity load requirements if the diaphragm shears are such that providing a thicker deck is more economical than providing additional sidelap fasteners or additional deck-to-structural connections. However, as a rule the authors have found that the maximum number of structural connections and increasing sidelap connections should be used prior to increasing the deck thickness. It is not generally economical to change the spacing of the structural members in order to provide greater diaphragm resistance.

After selecting the type of deck for gravity loads, the next step is to design the diaphragm for horizontal lateral forces. Once the horizontal loads and diaphragm shears have been determined, the specifying professional uses the load tables to determine the type and number of connectors to achieve the required strength. To optimize the design, it is common for the connection patterns and or the deck thickness to vary across the diaphragm. This is done so the strength can be matched close to the actual shear in the diaphragm, i.e. as the shear decreases the attachment requirements are decreased. When this procedure is used it is necessary for the specifying professional to provide the locations of the deck thicknesses and fastener spacing. Shown in Figure 4.2.1 is an illustration of weld and screw patterns used to meet the strength requirements of the diaphragm system.




Use 36/4 with1 Side Lap Screw Use 36/7 with2 Side Lap Screws

**Fig. 4.2.1 Roof Diaphragm Key Plan**

In addition to strength considerations the deflection of the diaphragm must be considered relative to its effect on the building. The deflection of a given diaphragm can be calculated based on the equations provided in the SDI Diaphragm Manual or in various Evaluation Reports. The diaphragm deflections can then be compared to serviceability requirements for the structure in question. Serviceability guidelines for low rise steel buildings can be found in AISC's "Serviceability Design Considerations for Low- Rise Buildings, Steel Design Guide Series 3," (Fisher, J.M. and West, M.A., 2019). Generally, serviceability limits are not contained in the building codes since they are not life safety issues. The specifying professional should discuss these limits with the client because they may have a significant impact on the quality of the structure as well as its cost. The Evaluation Reports on diaphragms contain tables of diaphragm flexibility limitations. The reports also indicate that when diaphragms are supporting masonry or concrete walls, the maximum deflection of the diaphragm should be computed using the code prescribed lateral forces, and should be limited by the equation below:

$$\Delta_{wall} = 100 h_w^2 f_c^{'} E_w t_w$$

$h_w$ = Unsupported height of the wall in feet.

$t_w$ =Thickness of the wall in inches.

$E_w$ = Modulus of elasticity of wall material for deflection determination in pounds per inch.

Where a reduced $E_w$ is given in the code for uninspected masonry, the reduction is not recommended in this equation.

$f'_c$ =Allowable compressive strength of wall material in flexure in pounds per square inch.

This equation cannot be derived and is empirical in nature. It is not intended as a strict limitation, but rather a guideline for the specifying professional in giving thought to the diaphragm deflection.

## Diaphragm Connections

Once the shears and chord forces are determined and the fastening system selected the connections must be designed to transfer the forces into and out of the diaphragm. These connections are extremely critical and often overlooked. They are not discussed in any detail in the referenced documents. The connections fall into three basic categories:

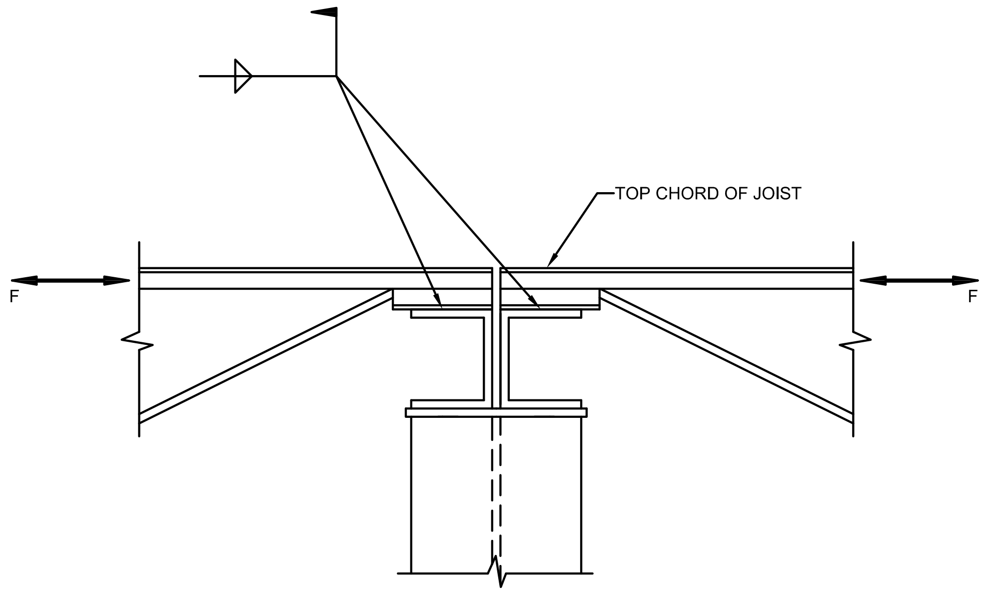
1. Chord Force Connections
2. Shear Wall Attachments
3. Vertical Bracing Attachments

## Chord Force Connections

*Chord Connection Parallel to Joist*

As mentioned previously, the perimeter members of the diaphragm serve to carry the flexural forces. The chord force is determined using beam theory, i.e. the maximum bending moment in the diaphragm is calculated and the chord force is determined by dividing the moment by the depth of the diaphragm. The perimeter member of the diaphragm must have the strength to resist the chord force along with any other imposed loads. If the perimeter member is a joist or Joist Girder, the forces resulting from diaphragm action must be provided to Vulcraft, unless it can be determined that the perimeter member will not be overstressed by the chord force. Forces from all load cases in combination with the diaphragm chord force must be specified. It is not enough to simply provide Vulcraft with the chord force because of the need to check specific code load combinations which include the diaphragm force. The specifying professional should note if the chord force is a wind or seismic load and should indicate if ASD design loads or LRFD design loads are to be used.

Suitable connections are also required between members acting as the diaphragm chord. Specifically, force continuity must be provided between adjacent chord members just as would be provided in plate girder flange splices. This is illustrated in the detail shown in Figure 4.2.2, for small chord forces.




Note: The girder must be designed to transfer the force F across the gap between girder top chord angles.

**Fig. 4.2.2 Diaphragm Chord**

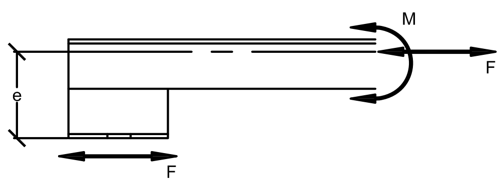



**Fig. 4.2.3 Joist Chord Bending**

Represented in the Figure 4.2.2 are the top chords of a perimeter joist in a diaphragm system. Force, *F,* is the chord force determined using the beam theory described above. The force, *F* to be transferred from one joist to the next must pass through the joist seat, the welds connecting the seat to the Joist Girder, across the gap between top chord angles of the Joist Girder, to the adjacent joist. This is a legitimate force path, but each component must be designed to resist the force. The capacity of this connection is limited. The strength may be controlled by the strength of the joist top chord. The chord is subjected to the axial force, the bending from uniform loads and the additional bending moment as shown in Figure 4.2.3, where $M = F(e)$. This bending moment could severely overstress a joist chord if no consideration were given to it in the design. Reinforcement of the chord may be required. Vulcraft can design the joist to accommodate the axial force and bending moment, if the proper information is provided; however, the result would probably be a substantial increase in the size and cost of the perimeter joists.

A better force path would be created if a top plate or tie angles were added to transfer the axial force directly from the top chord of one joist to the top chord of the adjacent joist. For roofs, this is illustrated in the details shown in Figures 4.2.4 and 4.2.5.

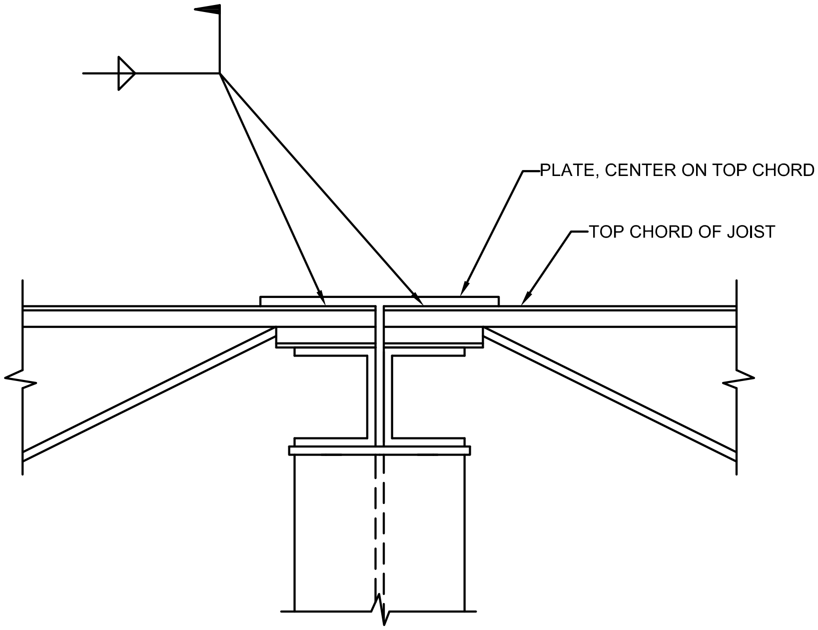



**Fig. 4.2.4 Joist Tie Plate**

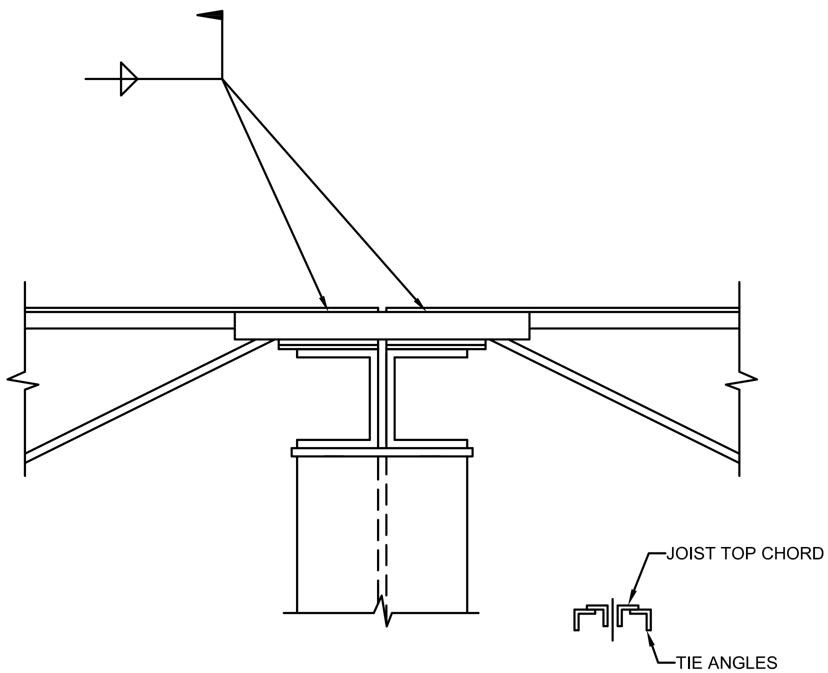



**Fig. 4.2.5 Joist Tie Angles**

The tie connection and weld sizes are based on the calculated chord force. As mentioned, the specifying professional needs to specify the required axial load on the plans so Vulcraft can design the joist accordingly. The design professional does have the option to do additional calculations to check if a standard SJI chord number designation joist is adequate for the vertical loads plus the chord force. It will be faster and give a more efficient joist design to specify the axial load on the plans along with the vertical load requirements.

The following example illustrates the design of the continuity tie and a procedure to check the joist chord for the diaphragm chord force.

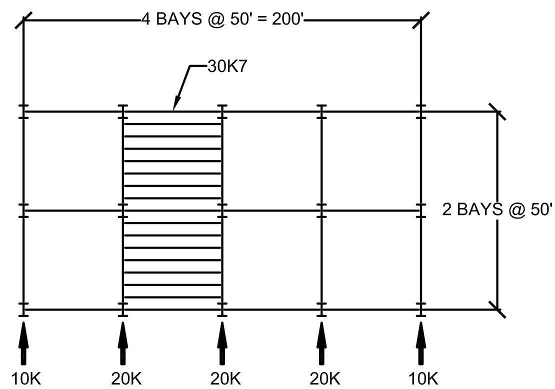



**Fig. 4.2.6 Example 4.2.1 and 4.2.2**

### Example 4.2.1 Diaphragm Chords (ASD)

Determine whether the standard 30K7 perimeter joist shown in Figure 4.2.6 is adequate for the diaphragm chord forces due to ASD design wind loads.

**Given:**

The perimeter joists have a "net" uplift load of 150 plf acting simultaneously with the diaphragm forces (ASD). Perimeter joists have a dead load = 14 psf, a roof live load = 20 psf and a downward wind load 0.6W=10 psf. The joists are at 6'-3" on center

**Solution:**

*Determine the diaphragm chord force from the lateral loads:*

$M_{a\ chord}$ = (30 kips)(100 ft.) - (20 kips)(50 ft.) = 2000 kip-ft.

$P_{a\ chord}$ = 2000 kip-ft./100 ft. = 20.0 kips (ASD)

*Uniform loads:*

Tributary width = 6.25 ft./2 = 3.125ft.

Dead Load $w_{DL}$ = (3.125 ft.)(14 psf) = 43.75 plf, say $w_{DL}$ = 44 plf

Roof Live Load $w_{Lr}$ = (3.125 ft.)(20 psf) = 62.5 plf, say $w_{Lr}$ = 63 plf

Downward Wind Load $0.6w_W$ = (3.125 ft.)(10 psf) = 31.25 plf, say $0.6w_W$ = 32 plf

*Downward Load:*

$D+L_r$ $w$ = 44 plf + 63 plf = 107 plf

$D+0.75(0.6W) + 0.75L_r w$ = 44 plf +0.75(32 plf) +0.75(63 plf) = 115.25 plf < Net Uplift. Load

Net Uplift Load is greater than downward load, so it will control the bending moment in the joist.

*Determine the joist chord force from uplift:*

$M_a = wL^2/8 = (0.150 \text{ k/ft.})(50 \text{ ft.})^2/8 = 46.9$ kip-ft.

$P_a = 46.9$ kip-ft.)(12 in./ft.)/[(30 in.- 2(0.5 in.)] ≅ 19.5 kips (tension)

Where 0.5 in. is the estimated centroid distance for the chord angles.

The maximum required axial chord force, $P_a$ = 19.5 kips+ 20 kips = 39.5 kips (tension).

*Determine the allowable tension chord forces:*

Based on the joist load tables a 30K7 can support 203 lbs/ft.

The allowable moment $M_n/\Omega$ = 63.4 kip-ft.

The allowable tension chord force = $M_n/\Omega d$

= (12 in./ft.)(63.4 kip-ft.)/[(30 in.- 2(0.5 in.)] = ± 26.2 kips.

Allowable chord force = 26.2 kips < 39.5 kips needed. **n.g.**

**Result**: standard 30K7 joist is not adequate to act as the diaphragm chord member.

See the following alternate solutions for the diaphragm chord requirements.

**Alternate Solution 1**:

Since the required axial chord force is greater than the available force, the axial chord forces should be provided to Vulcraft for a modified 30K7 design. Due to the load combinations, Vulcraft will combine the uniform load with the axial load.

**The joist can be specified as shown in Table 4.1.**

| JOIST SCHEDULE | | | | | | |
|---|---|---|---|---|---|---|
| Joist Mark Number | Designation (1) | Loads for Combined Bending and Axial Check (2) | | | | |
| | | Wind Top Chord Axial Load 0.6W | Dead Load | Roof Live Load $L_r$ | Downward Wind load 0.6W | Net Wind Uplift load (3) |
| J1 | 30K7 | 20.0 kips | 44 plf | 63 plf | 32 plf | 150 plf |

(1) Standard designation is minimum requirement. Joist Manufacturer to modify joist design as required for combined loading requirements.
(2) Joist manufacturer to use these load in the applicable code load combinations to design the joist for combined bending and axial.
(3) Net Wind Uplift is the result of the 0.6D+0.6W load combination.

**Table 4.1 Joist Schedule for Perimeter Joist**

Note: This solution has the benefit that the specifying professional could skip checking the 30K7 to see if it is adequate for the diaphragm chord force. By specifying all the loading, Vulcraft will check the joist for the standard requirements and check the joist for the axial load requirements and modify the joist as needed.

The specifying professional could take it one step further and specify the joist using the Load/Load designation (30K107/63) along with the wind load, thus bypassing having to determine the standard designation.

**Alternate Solution 2:**

Since the required axial chord force is greater than the 30K7's capacity a larger edge joist can be specified.

The capacity of the 30K7 can be scaled in order to determine the uniform load that would be required for a 30 in. deep joist to work for the combined loading.

The 30K7 capacity is 203 plf, which translated to 26.2 kip axial capacity. The required axial force is 39.5 kips.

Required uniform load = (39.5 kips/26.2 kips)(203 plf) = 306 plf

Based on the standard load tables for 50ft. span, a 30K11 has a capacity of 333 plf > 306 plf **o.k.**

**Specify: 30K11 for the perimeter joist**.

Note: The 30K11 may weigh more and be more expensive than 30K7 with axial load. The 30K7 with axial load will only require the top chords to be modified. The 30K11 will have a larger top chord than the 30K7. The webs and bottom chord of the 30K11 will also have to be upgraded from those required for a 30K7 due to the additional vertical load required for a 30K11 joist. This solution also requires additional calculations by the specifying professional compared to the method in Alternate Solution 1.

### Example 4.2.2 – Tie Plate and Connection for Diaphragm Chord Force (ASD)

Design a tie plate and the required weld to transfer the chord axial force $0.6W = \pm 20$ kips (ASD) to the 30K perimeter joists from Example 4.2.1.

Note: These calculations should be made by the specifying professional. Any assumptions made by the specifying professional in the calculations need to be specified on the plans, for example, the top chord horizontal leg size and top chord angle minimum thickness.

**Solution:**

The limit states of: tension yielding, tensile rupture, block shear and base material strength are required for the design of the tie plate and the attachment to the top chord of the joist. The compressions buckling strength of the tie plate must also be determined.

*Tie Plate Design: (use A36 steel)*

*Tension yielding:*

Try a tie plate PL 4x1/4.

$A_g = 1.0 \text{ in.}^2$
$P_n = F_y A_g = (36 \text{ ksi})(1.0 \text{ in.}^2) = 36 \text{ kips}$ AISC Eq. (D2-1)
$P_n/\Omega = 36 \text{ kips}/ 1.67$
$= 21.6 \text{ kips}$
21.6 kips > 20 kips **o.k.**

*Compression Buckling of the tie plate:*

Slenderness ratio = $L_c/r_x$

With a one-inch gap between the joist ends and the weld will extend to the end of the joists, $L_c = 1$ in.

$r_x = 0.288675d$ AISC Manual Table (17-27)
$r_x = (0.288675)(0.25 \text{ in.}) = 0.0722 \text{ in.}$
$L_c/r_x = 1 \text{ in.}/0.0722 \text{ in.} = 13.9$

Since $L_c/r_x \leq 25$ the compressive strength equals the tension yield strength
AISC Section (J4.4)

**Note:** The size of the tie plate is influenced by and affects several parameters of the design:

1. For projects with steel deck: the tie plate thickness should not be greater than 3/8 in. for the steel deck to be installed over the plate

2. The width of the plate must be coordinated with the width of the joist top chord. For example, the width of the plate plus the weld shelf dimension each side, should be the same or less than the width of top chord angles plus the gap between the angles (top chord width). Having the tie plate and shelf dimension be less than the top

chord width will allow for down hand welds. The gap between top chord angles is often 1 in. (it may be less on small rod web joists). Figure 8-13 in the AISC Steel Construction Manual (AISC, 2017) provides minimum shelf dimensions for various weld sizes

3. The thickness of the plate dictates maximum fillet weld leg size

4. The capacity of the weld is the lesser value of the weld metal strength and the base metal strength, per AISC Specification Section J2.4. Due to the custom angle sizes used by Vulcraft, the base metal thickness should be checked

Tie Plate width and top chord width compatibility:

Assume 3⁄16 fillet weld, shelf width = 7⁄16 in.

Joist chord width ≥ Plate width + 2(shelf width)
Joist chord width ≥ 4.0 in. + (2)(7/16 in.) = 4.88 in.
Joist horizontal leg length = (4.88 in.-1.0 in gap)/2 = 1.94 in. < 2.0 in.
**Specify Minimum top chord angle horizontal leg = 2 in.** (2 in. is a common angle size for joists).

*Weld design-tie plate to joists: (E70 electrodes):*

Continue check with 3/16 fillet weld:

*Weld Metal Check:*

$R_n = (0.6F_{EXX})(0.707)(D/16) = (0.6)(70\text{ ksi})(0.707)(3/16) = 5.57\text{ kips/in.}$

$R_n/\Omega = (5.57\text{ kips/in})/2.0 = 2.78\text{ kips/in.}$

Total required weld length = (20.0 kips)/(2.78 kips/in) = 7.18 in.

Try 4 in weld each side of plate to joist top chord: Total weld length = 2(4 in) = 8 in. **o.k.**

Per AISC Table D3.1 (Case 4) the weld lengths on each side of the plate ($l_1$ and $l_2$) shall not be less than 4 times the weld size.

Minimum length =(4)(3/16 in.) = 0.75 in. < 4.0 in. **o.k.**

*Minimum base material check of the top chord angles (shear rupture controls).*

Required base metal strength = 20 kips/(4 in.+4 in.) = 2.5 kips/in.

$R_{n\ base\ metal} = (0.6)(F_u\text{ ksi})(t\text{ in.})\text{ kips/in.} = (0.6)(65\text{ ksi})(t\text{ in}) = 39t\text{ kips/in.}$

$R_{n\ base\ metal}/\Omega = (39t\text{ kips/in})/2.0 = 19.5t\text{ kips/in.}$

$R_n/\Omega$ = Required strength: $19.5t$ kips/in. = 2.50 kips/in.

The minimum chord thickness, $t$ = (2.50 kips/in)/(19.5 kips/in.) = 0.128 in.

**Specify: minimum top chord thickness = 0.128 in.**

(Vulcraft has multiple angle thicknesses between 1/8 in. and 3/16 in., as well as between 3/16 in. and ¼ in. As a result, the minimum thickness does not have to be rounded to a 1/16 of an inch).

*Tensile rupture check for the* 4x1/4 *plate with 4 in. long weld each side:*

Determine the shear lag factor for the plate from AISC Specification Table D3.1, Case 4.

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right) = \frac{3(4.0 \text{ in.})^2}{3(4.0 \text{ in.})^2 + (4.0 \text{ in.})^2}\left(1 - \frac{0.125 \text{ in.}}{4.0 \text{ in.}}\right) = 0.73$$

where
$l$ = (4 in. + 4.0 in.)/2 = 4.0 in.
$w$ = 4 in. wide plate.
$\bar{x}$ = 0.25 in./2=0.125 in.
$P_n = F_u A_e$ AISC Eq. (D2-2)
$A_e = A_n U$ AISC Eq. (D3-1)
$A_n$ = (4.0 in.)(0.25 in.) = 1.0 in.$^2$
$A_e$ = (0.73)(1.0 in.$^2$) = 0.73 in.$^2$
$P_n$ = (65 ksi)(0.73 in.$^2$) = 47.4 kips
$P_n/\Omega$ = 47.4 kips/2.0 = 23.7 kips ≥ 20 kips **o.k.**

*Check Joist top chord angles at tie plate:*

From earlier calculations, the angles require a 2 in. horizontal leg. Typical top chord angles are equal leg angles. From earlier calculations, the minimum thickness required was 0.128 in. Thus, assume L2x2x0.128 top chord angles. Angles have $F_y$ = 50 ksi, $F_u$ = 65 ksi.

$A_g$ = 0.99 in.$^2$ (for the two L2x2x0.128 angles)

*Check block shear of joist top chord angles at the tie plate:* AISC Section (J4.3)

Shear area equals the thickness of the two top chord angle legs for the total length of the weld:

$A_{gv}$ = (2 angles)(0.128 in.)(4.0 in.) = 1.02 in.$^2$

Tension area is the area of the top chord angles beyond the tie plate. With L2x2 top chord angles and a 1.0 in. gap:

Top chord width = 2.0 in + 1.0 in. + 2.0 in. = 5.0 in.

Total length beyond plate = 5.0 in. - 4.0 in. = 1.0 in.

$A_{nt}$ = (1.0 in.)(0.128 in.) = 0.128 in.$^2$

$R_n = 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$ = (0.6)(50 ksi)(1.02 in.$^2$) + (0.5)(65 ksi) (0.128 in.$^2$) = 34.8 kips

$R_n/\Omega$ = 34.8 kips/2.0 = 17.4 kips < 20.0 kips **n.g.** (need to increase weld length)

**Increase weld length to 5.0 in.**:

$A_{gv}$ = (2 angles)(0.128 in.)(5 in.) = 1.28 in.$^2$

$R_n$ = (0.6)(50 ksi)(1.28 in.$^2$) + (0.5)(65 ksi) (0.128 in.$^2$) = 42.6 kips

$R_n/\Omega$ = 42.6 kips/2.0 = 21.3 kips > 20.0 kips - **5.0 in. long weld o.k.**

*The top chord tension yield check will be done by Vulcraft based on the provided top chord.*

*Check tensile rupture of the joist chord angles:*

Determine the shear lag factor for top chord from AISC Specification Table D3.1, Case 4.

Treat the two angles as a Tee Section (since joist construction will connect the angles):

$$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right) = \frac{3(5.0 \text{ in.})^2}{3(5.0 \text{ in.})^2 + (4.0 \text{ in.})^2}\left(1 - \frac{0.534 \text{ in.}}{5.0 \text{ in.}}\right) = 0.74$$

where
$l$ = (5.0 in. + 5.0 in.)/2 = 5.0 in.
$w$ = the plate width = 4.0 in.
$\bar{x}$ = 0.534 in. (centroid of angles)

$P_n = F_u A_e$ AISC Eq. (D2-2)
$A_e = A_n U$ AISC Eq. (D3-1)
$A_e$ = (0.99 in.)(0.74) = 0.73 in.$^2$
$P_n$ = (65 ksi)(0.73 in.$^2$) = 47.4 kips
$P_n/\Omega$ = 47.4 kips/2.0 =23.7 kips $\geq$ 20 kips **o.k.**

Plate Length: $L$ = 5.0 in. + 1.0 in. gap between joists + 5.0 in. = 11.0 in.

**Use: Tie Plate 4"x ¼"x 0'-11" with 3/16" fillet welds 5.0 in each side of plate to each joist.**

**Specify in detail on plans: Minimum top chord thickness = 0.128 in.; Minimum top chord angle horizontal leg = 2 in.**

Note: Because the weld had to be lengthened after the top chord base metal check was run, it is possible to decrease the minimum required thickness with the 5 in. weld each side of the plate. Since the required thickness is so close to 1/8 in. it is not necessary for this connection.

If the joists shown in Figure 4.2.6 had been framed in the opposite direction, then the diaphragm chord force would be resisted by some other structural element.

The detail shown in Figure 4.2.7 represents one example at a building end wall where the joist is supported by a Joist Girder.

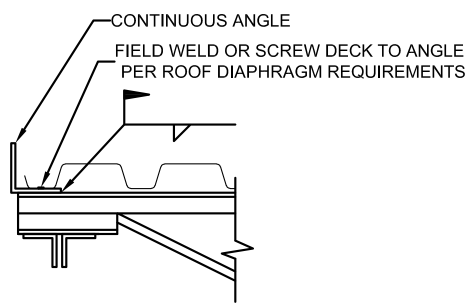



**Fig. 4.2.7 Deck Support Angle**

In many buildings the angle is provided to support the deck and attached roofing materials from tearing due to construction and foot traffic. The need for the angle as a deck support member should be determined first. If the angle is to be provided for this purpose it is possible to also size the angle for the diaphragm chord requirements. A word of caution is appropriate here. Typically, the edge angles cannot be installed as one continuous angle, whether due to building length or practicality of installation. If the edge angle is used as the diaphragm chord member, the splice connection must be detailed on the construction plans in order to provide the continuous load path. If this detail is not shown it is likely the angles will simply be butted together and not connected.

If the girder shown in Figure 4.2.7 is to be used as the chord member for the diaphragm, the specifying professional will need to consider the load path for the chord forces. There will need to be a tie connection between girders to form the continuous member, typically a knife plate or tie plate. In addition, the chord forces must be transferred down to the girder to activate it. To form this load path, the angles collect the load from the deck diaphragm. The angle must transfer that load to the joists. The joist then transfers the load to the girder. This is done by rollover on the joist seat. The rollover strength of typical joist seats is 1,820 pounds. This capacity is discussed in Chapter 7. If additional strength is required, some type of shear transfer member is installed between the joists to directly connect the deck to the girder. For diaphragms that resist seismic loads, especially in high seismic areas, it is best to provide shear transfer members (blocking) for a positive connection between deck and girder. This avoids the joist seat experiencing cyclical rollover loading from the seismic loads.

If the girder shown in Figure 4.2.7 is in line with or part of the Lateral Force Resisting System, the angle may also need to be used to transfer the diaphragm shear parallel to the deck span down to the girder. The deck is attached to the angle with welds, power driven fasteners or self-drilling screws. Without these fasteners it is not possible to transfer the diaphragm shear from the deck directly to the joist. The load is transferred from the angle to the joist. Then from the joist to the girder by means of joist seat rollover. As discussed in the previous paragraph, the joist seat has a relatively small capacity for rollover. If additional capacity is needed for the diaphragm load transfer, some type of shear transfer member will need to be installed. Just like with seismic chord force, described in preceding paragraph, it is best to use blocking to transfer seismic loads from the diaphragm to the girder. Once the load is in the Joist Girder top chord a proper force path to connect girder to girder must be provided. This can be done with a knife plate or with a tie plate or tie angles, like that shown in Figure 4.2.4 and Figure 4.2.5.

### Shear Transfer Members

It is required to provide a positive load path to transfer the diaphragm shears into the vertical lateral force resisting system. A variety of details have been used for this purpose. In the preceding paragraphs the transfer of deck shears to Joist Girder top chords was briefly discussed. For small shears it was pointed out that the joist seat could be used for this transfer. For larger shear loads, or seismic loads, a shear transfer member, would likely be required between the deck and the Joist Girder top chord, or edge beam. There are multiple options for the shear collector members including HSS tubes, channels, joist substitutes (2.5K or VS joist), angles and bent plates. The details shown in Figures. 4.2.8 and 4.2.9 provide two possible examples. The height of the shear transfer member should match the height of the joist seat. This means different size members will need to be used for K-Series and LH-Series joists. When choosing the shear transfer member, the specifying professional must consider the width of the horizontal top surface for making the connection from the deck to the shear transfer member. The connection of shear transfer to the Joist Girder or wide flange beam can be done with a continuous weld or a stitch weld depending on the loading. On Joist Girders, the shear transfer members should either be attached to both top chords, or attached to alternating girder top chord angles. This way the load is equally distributed to the girder top chord angles. The contractor must take care when installing the shear transfer member and the deck. The low flute of the deck should be flat against the shear transfer member in order to properly fasten the deck. Provided in the AISI Specification are equations for calculating allowable arc spot weld shear forces. The manufacturers of mechanical fasteners also provide strengths for determining the required spacing of the fasteners for the required loading.

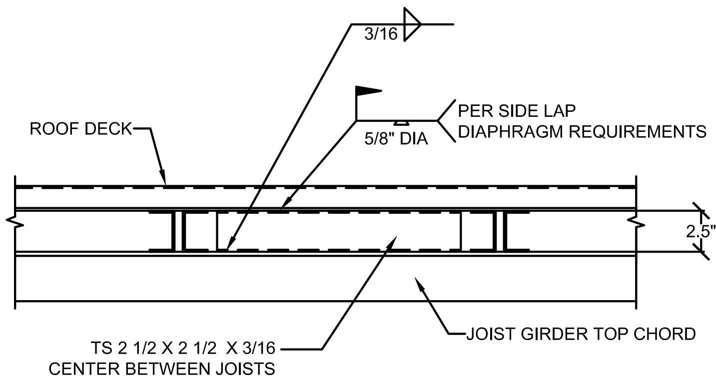



**Fig. 4.2.8 Shear Transfer Member with K Joist**

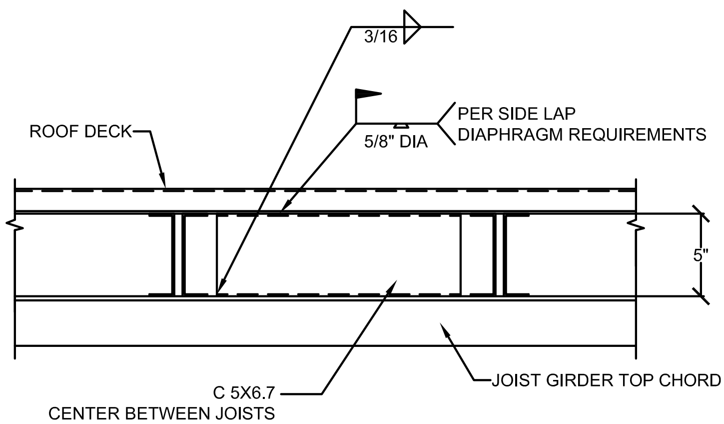



**Fig. 4.2.9 Shear Transfer Member with LH Joist**

## Attachment to Shear Walls

Shear walls are often used to transfer diaphragm forces from floors and roofs to the foundation system. The edge attachment of the diaphragm to the shear wall can be accomplished in several ways. It is common to use ledger angles, channels or bent plates. The deck is attached to the ledger and the ledger is connected to the shear wall. The connection to the shear wall can be done with embeds, headed weld studs, bolts or other means. In addition to the shear transfer, the effects of gravity load and wind uplift must be considered on these connections. When a joist or Joist Girder is positioned directly next to the vertical wall, allowance must be made for the vertical movement of the joist or girder relative to the "non-deflecting" vertical wall. If the shear transfer device loads the wall vertically as well as horizontally then the wall must be designed for the vertical load as well as the shear force. In Figures. 4.2.10 and 4.2.11 shear transfer details are illustrated which permit vertical movement. The plate size, welding and wall attachments must be designed for the shear forces. The detail shown in Figure 4.2.12 does not have "built in" slip, thus the connecting plate must be designed to flex vertically under the action of gravity or uplift wind loading or it must be designed to transfer the vertical loading to the wall. Particular attention must be paid to the weld design for this detail. When the joist deflects vertically considerable prying is placed on the arc spot welds.

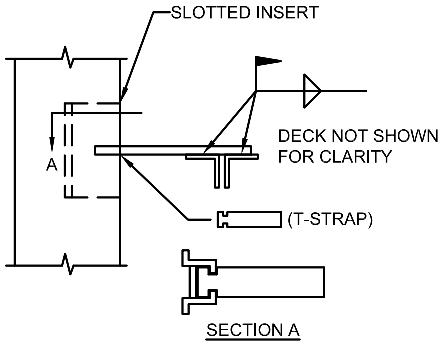



**Fig. 4.2.10 Shear Transfer to Precast (T-Strap)**

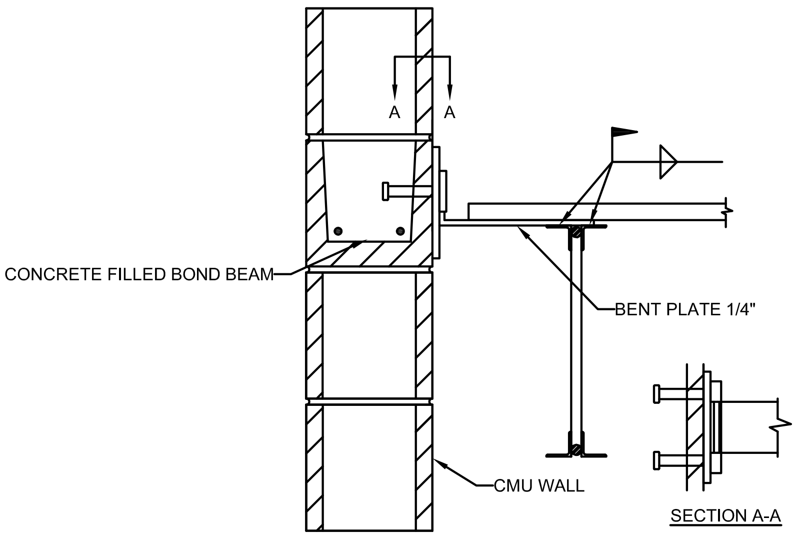



**Fig. 4.2.11 Shear Transfer to Masonry**

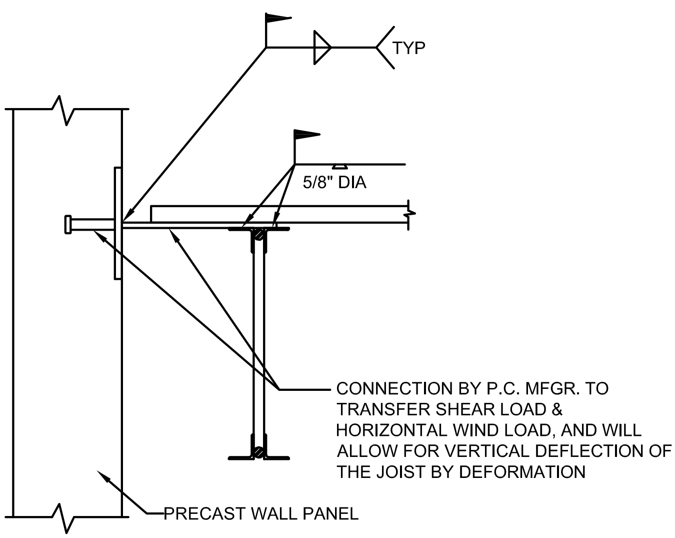



**Fig. 4.2.12 Shear Transfer to Precast**

The detail shown in Figure 4.2.13 represents one way that the shear forces can be transferred to a CMU wall when the joists frame into the wall. The continuous angle provides the multipurpose function of deck support, shear transfer and diaphragm chord.

When the first joist parallel to the shear wall is not installed immediately next to the wall, i.e. it is located a typical joist space away from the wall, the design considerations change. The detail shown in Figure 4.2.14 is an example of this condition. The continuous ledger angle can be installed conforming to the roof slope. With the additional distance, the deck can flex to accommodate the vertical deflection of the joist. The detail also has the advantage of accommodating joist camber. The deck will flex to accommodate most camber conditions for K-Series joists. The accommodation of camber is discussed further in section 5.10. In these details the angle ledger also serves as the structural support for the gravity loads on the deck. This must be included in the design of the attachment. The design of the ledger to the wall should take into consideration the construction sequencing in order to come up with the most cost-effective connection.

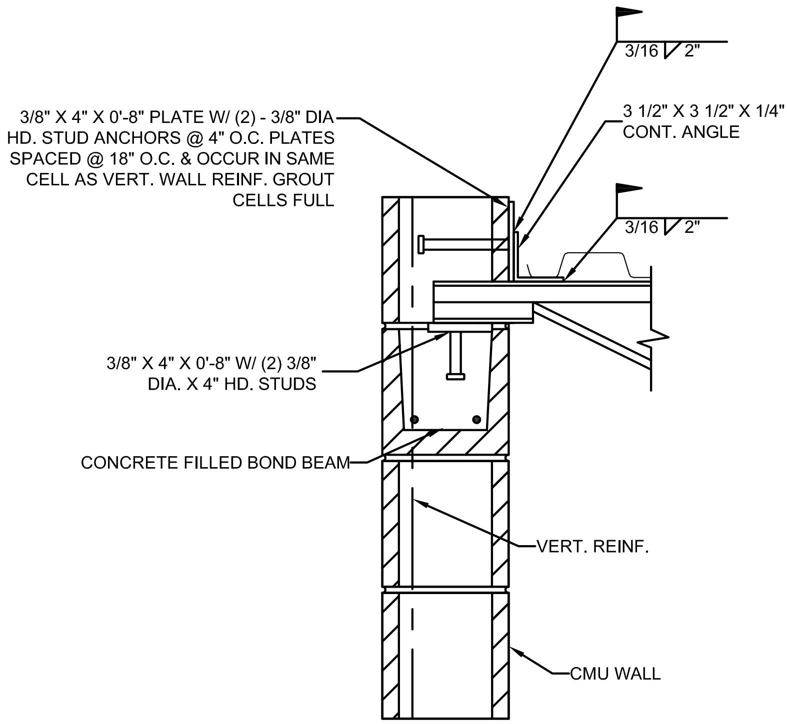



**Fig. 4.2.13 Gravity and Shear Load Transfer to Masonry**

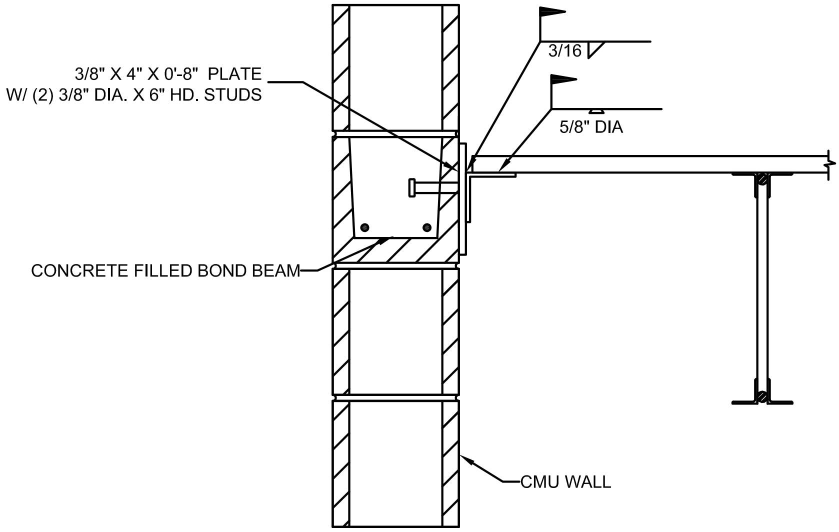



**Fig. 4.2.14 Gravity and Shear Load Transfer to Masonry**

In seismic areas, projects with concrete or masonry, shear walls may have different requirements and loading considerations for the connection to the wall. One such consideration is the out of plane wall anchorage forces to prevent the wall from pulling away from the building.

A few additional comments are pertinent to deck attachments to hard wall systems. These relate to building expansion and contraction. Some specifying professionals use masonry wall bond beams as the chord members for the diaphragm. The bond beam reinforcement must be continuous, which can cause expansion and contraction problems with the wall.

For buildings with long walls, diaphragm force attachments should take into consideration the expansion and contraction of the building. This may require the strategic use of horizontal and vertical slip joints at certain locations.

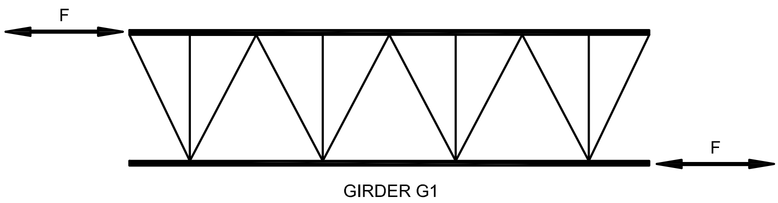
The specifying professional should be aware that the construction sequence may dictate the type of connection used to the wall system. In some cases, the walls may be present prior to the steel erection and in other cases the steel may be erected first. Where the steel is erected first, it may be necessary to hold the deck back from the wall in order to permit the connections to the wall to be made. This procedure may cause the erector to provide additional temporary bracing until the diaphragm is attached to the shear walls. This condition is discussed further in Chapter 8.

### Attachment to Vertical Bracing (Braced Frames)

When vertical steel bracing is located below the perimeter member, the perimeter member must transfer the forces into the vertical bracing. When the perimeter member is a Joist Girder, the bottom chord of the Joist Girder will interfere with the steel bracing unless the steel bracing is attached to the column below the bottom chord. The bottom chord can serve as the compression strut if it is connected to the stabilizer plate, otherwise the column must be designed to transfer the force from the column top to the vertical bracing. For proper force transfer the top chord force must be transferred to the column top by the girder seat or by some other means. The girder seat capacity is limited to 8 to 16 kips (ASD). See Chapter 7 for further discussion. For larger forces, the load must be transferred from the top chord by using a tie plate, knife plate or a similar attachment.

When the bottom chord is used as a strut the top chord force must be transferred to the bottom chord through the web members. The specifying professional must provide the force information so that Vulcraft can check the web members and can also determine lateral bracing requirements for the bottom chord of the Joist Girder. The force diagram shown in Figure 4.2.15 can be used to convey this type of information.

The specifying professional must schedule the required forces. The force, F, may be due to wind, seismic or other lateral loads.




Note: Design the web system to transfer the force F from the top chord to the bottom chord.

**Fig. 4.2.15 Joist Girder Note**

In addition to the information shown in Figure 4.2.15, Vulcraft must be informed as to how to combine the force in the load combinations. Sample load schedules are discussed in Chapter 6.

This same procedure could be used to transmit forces through the joists to the vertical bracing system. In most cases special edge joists will be required. When the bracing forces become relatively large (greater than 20 kips), it may be better to substitute a wide flange beam for the perimeter member. The size of the beam and its lateral bracing must be determined. Special bridging can be specified for the bracing of the beam. The beam also requires detailing at the column location to transfer shear and axial forces.

## Expansion Joints

At times it is necessary to transfer diaphragm shears across a roof or floor expansion joint. Any detail that allows the expansion joint to perform its intended function and yet is capable of shear transfer will work. The details shown in Figures 4.2.16 and 4.2.17 have been used. In Figure 4.2.16 the strap plates offer negligible resistance perpendicular to the joist direction, allowing the expansion joint to move, yet since they are axially stiff, they can transfer shear across the joint via tension in the straps. The joist seats must have the rollover capacity to resist the strap component of force in the direction of the expansion joint. If insufficient capacity exists, a shear collector member can be used on the Joist Girder lines to carry the force.

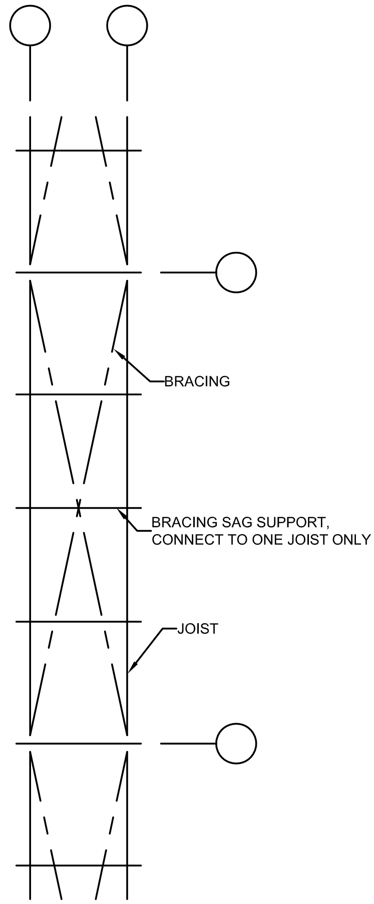



**Fig. 4.2.16 Expansion Joint Shear Transfer**

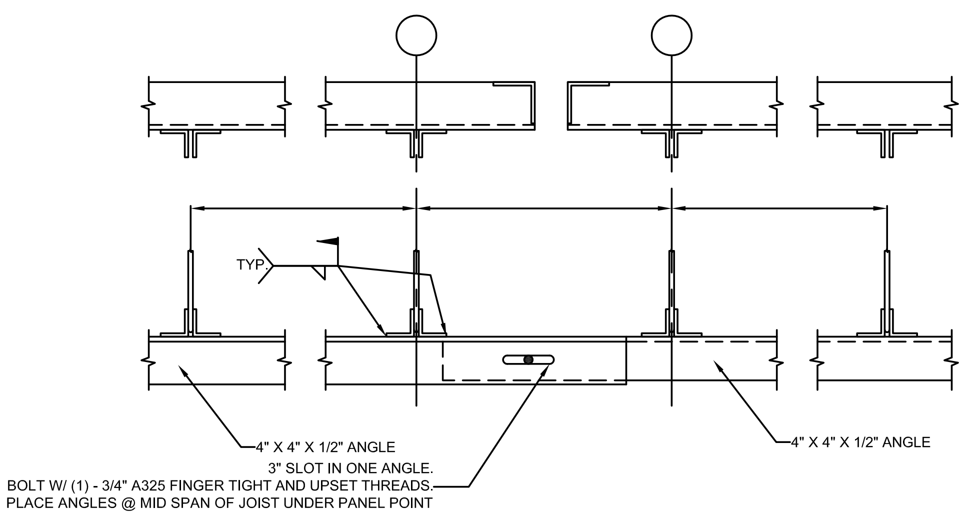



**Fig. 4.2.17 Expansion Joint Shear Transfer**

In Figure 4.2.17 the diaphragm shear is transferred thru the web members of the joists to the bottom chord. The angles perpendicular to the joist are designed to transfer the shear thru bending in the cantilevered portion of the angles. The slotted hole in the angles allows the expansion joint to function. As mentioned earlier, the joists must be properly specified for the web shears and chord forces.

## 4.3 HORIZONTAL BRACING

There are occasions when diaphragm action cannot be used to provide lateral stability for the structure. This most frequently occurs in single story structures where standing seam roofs are used. In these cases, lateral forces can be resisted in the plane of the roof with a horizontal bracing system. The bracing system can be designed to resist forces in one or both framing directions. In Figure 4.3.1 a roof plan is shown in which horizontal bracing is used to resist lateral forces in only one framing direction.

In Figure 4.3.2 a roof plan is shown in which the horizontal bracing is positioned to resist lateral loads in both framing directions. Any arrangement of bracing that forms a stable configuration can be used.

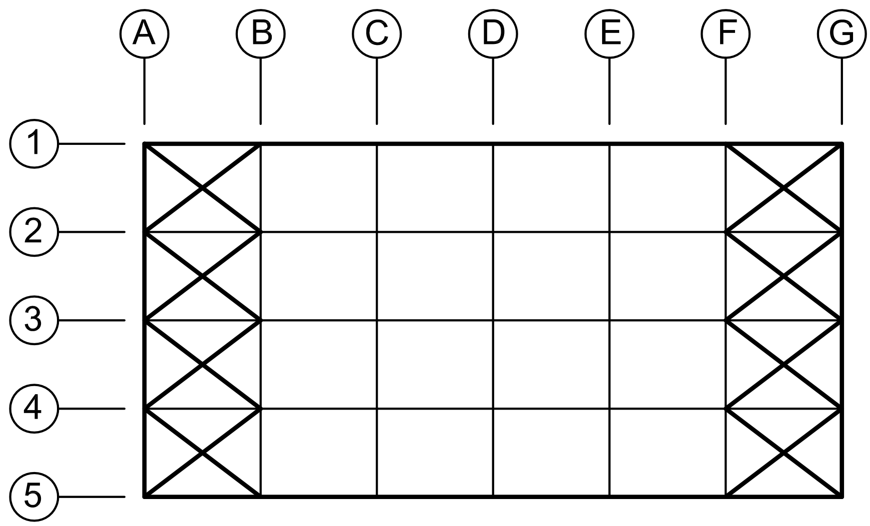



**Fig. 4.3.1 Roof Bracing, One Direction**

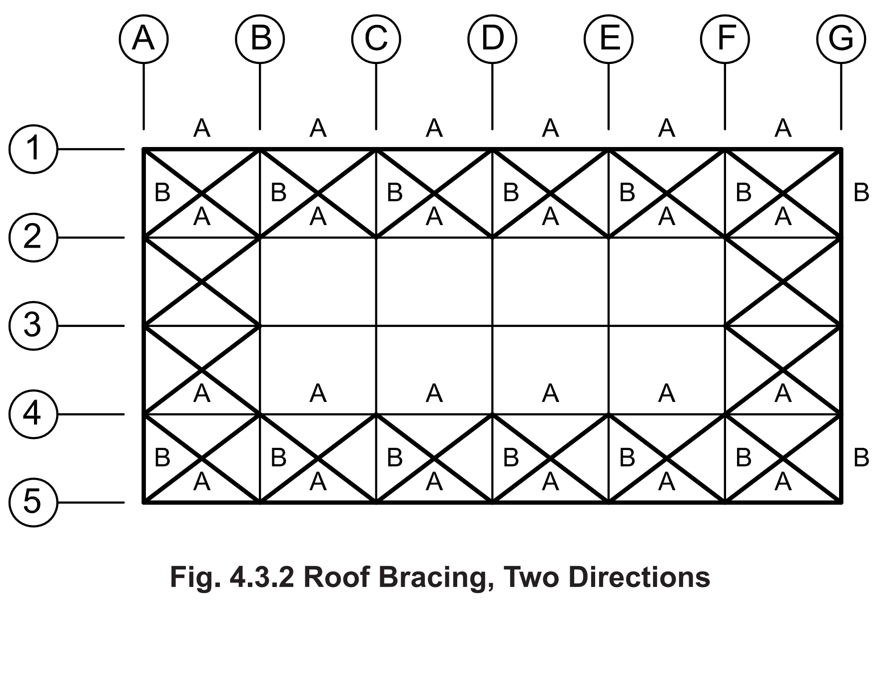



**Fig. 4.3.2 Roof Bracing, Two Directions**

The bracing members illustrated in Figures. 4.3.1 and 4.3.2 can take several forms. The bracing can be fabricated from angles, rods, channels, etc. hung directly under the roof joists, or made from thin strap material placed directly on top of the joists. Some attempts have been made to string rods thru the joist webs and have proven to be expensive, time consuming and often not workable. In most situations the roof strap bracing concept has been the most economical solution. If the strap braces are held to a ¼-inch maximum thickness they generally do not interfere with the standing seam roof application. If the design requires the straps to be much wider than 6-inches an alternate framing system should be considered, e.g. adding additional bracing or choosing an alternate lateral load system. Forces in straps of ¼ in. x 6 in. become so large that connections to structural elements are not practical.

There is concern about strap sag in that a large deflection will be required to remove the sag from the straps before the strap is capable of resisting load. If the steel erector attempts to remove most of the sag from the straps this movement will be minimal, but some sag will always be present. The strap is akin to a tight cable, in that it takes an infinite force to remove the sag from the cable or strap. Attempts have been made to provide "draw" in the strap bracings, but this is generally not worth the expense. It is a good idea to tack weld the straps to the tops of all the joists at each crossing to hold them in place.

## Analysis Procedure

Most horizontal bracing systems are analyzed assuming that the horizontal bracing works in combination with the joists and Joist Girders forming a deep truss. The joints are taken as pinned and only the tension diagonals are considered in the analysis. One question that arises is how to distribute the lateral loads among multiple horizontal trusses. If only one horizontal truss is contained within the roof framing the decision is simple. When two or more trusses exist then lateral loads must be distributed in some manner. A computer analysis could be made of the entire roof framing. However, this is generally not necessary. For the framing systems shown in Figures. 4.3.1 and 4.3.2 three options are viable. These are:

1. Design each horizontal truss for the full lateral load
2. Distribute the lateral loads equally to each of the horizontal trusses (assuming the trusses have equal stiffness)
3. Design each truss for wind pressure or wind suction loads

For seismic loads the specifying professional must provide a mechanism to "drag" the seismic lateral loads to each truss. This is also true for the column stability forces for gravity loads. However, the stability loads are generally quite small and special connections may not be required. For wind loads, forces must be transferred across the structure if option 2 is chosen. When more than two sets of horizontal trusses are provided similar distributions can be assumed. However, it is generally more economical to provide only two trusses in order to limit the number of special connections in the roof system.

The arrangement of the horizontal trusses within the roof has an economic bearing. This can be seen by referring to the truss layout shown in Figure 4.3.2. If the joists run left to right, then the joists marked A in Figure 4.3.2 would act as the chords in the long direction horizontal truss system, and the members marked B act as compression struts. The forces in the joists must be provided to Vulcraft for sizing. By positioning the bracing as shown in Figure 4.3.3, the magnitude of the chord force in the joists is reduced by 100 percent. In addition, eight strap braces are eliminated plus the number of joists with tie plates is reduced. See Figure 4.2.4. The only negative feature of this bracing arrangement is that the wind loads which are applied to the side walls must be transmitted through the Joist Girders before being transferred to the bracing system.

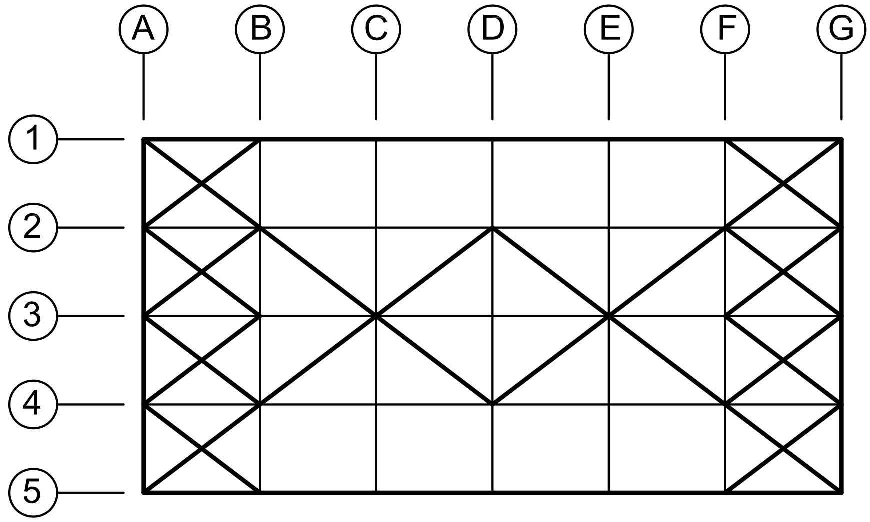



**Fig. 4.3.3 Horizontal Bracing**

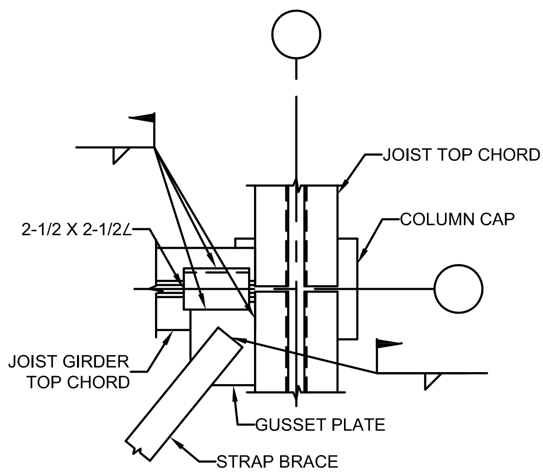



**Fig. 4.3.4 Strap Bracing Detail**

## Connections

Connections for horizontal bracing systems present the same design considerations as for connections with diaphragm systems. The loads must get into and out of the bracing. Bracing details must be provided to transfer the loads into the joists and Joist Girders without overstressing the end seats or chords, and the chord loading information must be provided to Vulcraft. A detail such as the one shown in Figure 4.3.4 can be used to transfer the strap bracing forces into the joists and Joist Girders. For structures with wind columns, details must be furnished to transfer the pressure and suction reactions at the top of the column into the bracing system.

## 4.4 BRACED FRAMES

Braced frames can be used as the only lateral system, or they can be used in conjunction with other systems, such as moment frames or shear walls. The following are items to consider with incorporating braced frames into the building design.

There are multiple options for how the bracing can be configured. These include chevron configuration, inverted chevron, eccentric braces, single diagonal, X-bracing and K configuration. The diagonal members can be structural shapes or Buckling Restrained Braces. The configuration, number of bays and detailing requirements will vary based on the building geometry, height and building code requirements/restrictions. The horizontal members of braced frames are typically wide flange beams. As discussed earlier, joists and Joist Girders can be used as the horizontal beam member, except in eccentric braced frames. This is typically only done on frames up to two or three stories in height. For taller frames or heavily loaded frames, it is typically simpler and more economical to use a wide flange beam. When a joist or Joist Girder is used in a braced frame, it may only qualify as an Ordinary Braced Frame, which may limit the uses as well. The use of joists and Joist Girders for the typical floor framing and wide flange beams in braced frames is common and not a problem.

### Multistory Frames

For multi-story steel buildings, braced frame systems provide an economical option. However, braced frames are not always allowed around the perimeter of the building. This is due to their interference with windows. On occasion, braced frames can be in bays where stair wells and elevator shafts are positioned at the perimeter bays. Depending on the building size and geometry, it may be possible to coordinate with the owner and architect to determine perimeter bays that can be used for the braced frames. It will often be a compromise between the structural needs of the building and the end use of the building.

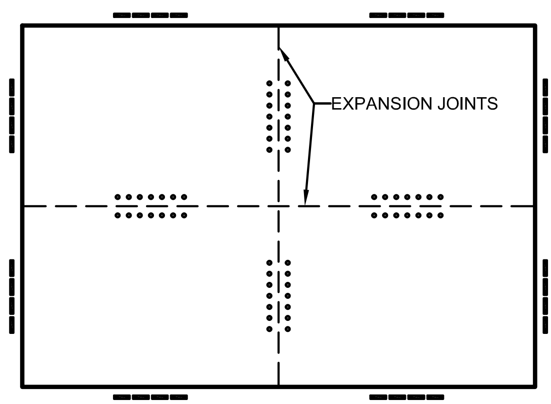
If the design team is considering designing a building with a dual system of braced frames and concrete or masonry shear walls, the construction aspect of the project should be considered in addition to any benefits from the dual system. This would also apply for dual moment frame and shear wall systems. Because these systems are built by different trades, there will need to be extra coordination on the project, especially where the steel framing connects to the shear walls. In addition, there are scheduling considerations with a dual system. The time to construct concrete or masonry shear walls is different than the time necessary to erect the steel portion of the structure. This can affect the overall length of schedule on a project or create challenges for the project schedule.

### Single Story Frames

Unlike multistory buildings, single story braced frames are often located at the perimeter of the structure. Generally, fewer windows exist and their locations can be avoided. Only overhead doors and exits must be avoided. All the braced frame configurations mentioned earlier can be used for single-story braced frames. The X-bracing configuration, either with rods or angles, is very common.

Much like diaphragms, the economical layout and number of braced frame bays will depend a great deal on the building geometry. When the length-to-width ratio between braced frame lines exceeds about 4 to 1, lateral forces become quite large. The drag strut loads that must be transferred to the frames also become large. When the lateral loads become large, it is often necessary to have multiple braced frame bays in a given frame line. This helps reduce the size of the members in the braced frames. It also reduces the column uplift forces due to overturning, reducing foundation costs. To reduce uplift forces, it is recommended that braced frames be placed in adjacent bays rather than separated. This allows the downward forces of one frame to be used to resist the uplift forces of the adjacent frame, hence minimizing uplift and reducing foundation costs. When the length to width ratio of the building exceed 4 to 1 the specifying professional should discuss the cost advantages of interior braced frame bays with the client. The client's first reaction is often negative, so it is often helpful to have sketches of the proposed braced frame configuration for the discussion. Sketches should include the clearances under the diagonals for forklift trucks or pedestrian traffic for the client's consideration. If additional clearances are desired, it may be possible to use moment frames for the interior frame bays. Moment frames are significantly more expensive than braced frames, so the owner will want to consider the cost-benefit ratio when deciding with the design team what is best for the project.

The need for interior braced frame lines often occurs in large structures that are quartered by expansion joints. Figure 4.4.1 shows the typical location of expansion joints in a large warehouse facility.




NOTES: STRUCTURE SHOW WITH ONLY ▬▬▬▬ BRACING IS TORSIONALLY UNSTABLE.
ADDING ••••••• BRACING PROVIDES STABILITY.

**Fig. 4.4.1 Expansion Joints**

The bracing shown around the perimeter does not provide lateral stability for the structure. Each building segment is torsionally unstable. The structure cannot be made stable even using the shear transfer details previously described in the diaphragm section. Using selected interior bays for bracing, usually on each side of the expansion joints is the most economical solution. If these bays cannot be braced, then a rigid frame solution may be in order.

## 4.5 MOMENT FRAMES

Ordinary Moment Frames (OMF) can be designed utilizing a joist or Joist Girder as the beam member of the frame. These frames are also referred to as Ordinary Truss Moment Frames (OTMF). Designing an OMF utilizing a joist or Joist Girder is no more difficult than designing with a wide flange beam. To obtain a cost-effective design the specifying professional must be aware of the interrelationships between the framing elements, i.e. joists, Joist Girders, columns and connections. In general, the most economical design is one which minimizes fabrication and erection costs of the connections, and one which reduces the special requirements (seat stiffeners, chord reinforcement, etc.) for the joists and Joist Girders. In high seismic zones, there are significant restrictions on the use of OMF. In these areas, a different type of moment frame may be required.

### Design Considerations

The first consideration is to determine if moment frames are required in both framing directions. When moment frames are required in only one direction, the framing scheme should be such that the Joist Girders are part of the moment frames. If moment frames are required in both directions, the framing scheme that creates the smallest end moments in the joists should be examined first.

For single story buildings, the Basic Connection (see Chapter 7) is often the most economical if it is adequate to resist the required lateral loads without modification. If the Basic Connection is not adequate or cannot be used, providing moment connections to each column in the line typically provides the most economical system. If the frame column is adjacent to a bearing wall, the Basic Connection may need to be modified or an alternate connection used (see Chapter 7).

For multistory projects, moment frames are best placed around the perimeter of the building as shown in Figure 4.5.1. Deep joists and Joist Girders on the perimeter do not interfere with head room requirements for the building interior. In addition, the exterior joists and Joist Girders can act as the spandrel system for the structure.

In multistory structures, moment connections are not always required in the roof. In some cases, the lateral forces can be transferred from the roof plane to the story below through cantilever columns extending from the floor below.

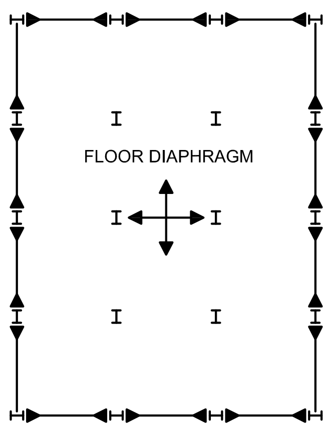



**Fig. 4.5.1 Multistory Framing**

## Rigid Moment Connections

As mentioned earlier, the Basic Connection is the most economical connection for moment frames, provided it has the capacity to resist the imposed lateral loads. This capacity is generally limited by bending stresses which are induced in the joist or Joist Girder top chords by eccentricities in the resisting moment force path. A complete discussion of these eccentricities is contained in Chapter 7, along with other design considerations relating to the Basic Connection and modifications that can be made to strengthen the moment resistance of the connection. Other types of moment connections are also discussed in detail.

As an aid to the specifying professional, typical moment details are summarized and discussed below. After determining the moments that exist at the connections in the frame, the specifying professional can evaluate which of these connections will provide the best solution. The capacities of the connections are discussed in terms of top chord forces in the joists and Joist Girders. This is because the top chord force generally limits the capacity of the connection. To determine the chord forces the specifying professional can divide the required moment by the appropriate force couple lever arm. For calculations involving the Basic Connection and most modifications to the Basic Connection, the lever arm is the distance from the centroid of the bottom chord to the underside of the seat. When this is not appropriate, the appropriate lever arm is indicated in the summary.

## Joist Girder Details

In all the details presented below, the column web must be checked by the specifying professional to determine if web stiffening is required. The design of the welds connecting the Joist Girders to the columns is the responsibility of the specifying professional. See Chapter 7 for further discussion relative to the design of the details presented here.

*Detail A* **-** (*Figure 4.5.2*)

The basic gravity load connection for Joist Girders becomes a moment connection when the bottom chords of the Joist Girder are welded to the stabilizer plates. This connection is the least expensive moment connection. It requires the fabricator to only provide the column cap plate and the stabilizer plates. The erector simply welds the Joist Girder seat to the column cap and to the stabilizer plates. The allowable top chord force for a Joist Girder is 8 to 16 kips, depending upon the size of the Joist Girder top chord. For LRFD the design strength equals 12 to 24 kips. In addition to the standard 3/4-inch A325 erection bolts the seat of the girder must be welded to the column cap using a 1/4-inch fillet weld 5-inches long on each side of the seat to achieve the loads indicated above. To reduce the forces from continuity, the specifying professional may want to specify the welding of the bottom chords after dead loads are applied to the Joist Girders.

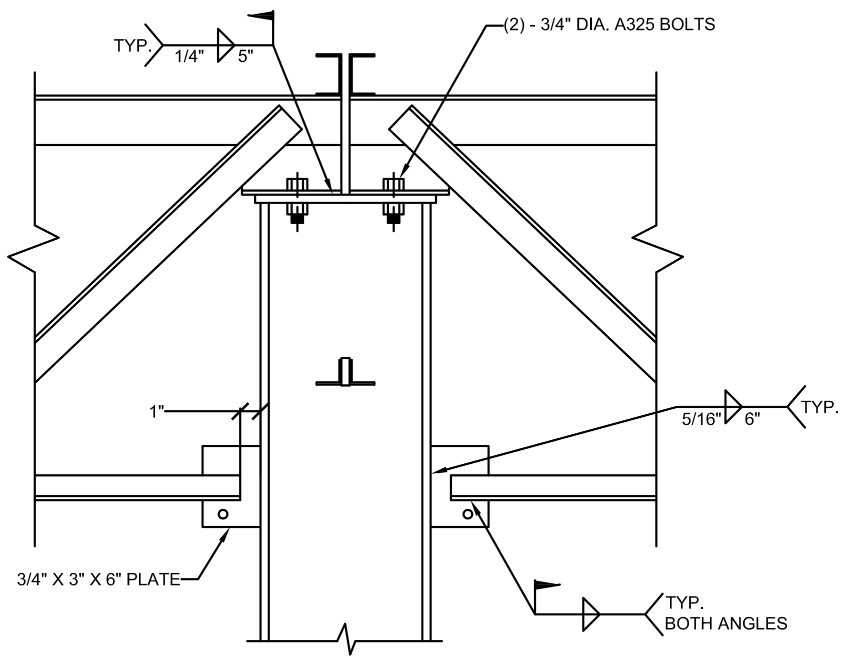



**Fig. 4.5.2 Detail A - The Welded Basic Connection**

*Detail B - (Figure 4.5.3)*

Illustrated in Figure 4.5.3 is a modification which can be made to the Basic Connection. The modification connects the top chords of adjacent Joist Girders using tie angles. The angles provide a path for the continuity moment chord forces to be transferred from one girder to the adjacent girder without requiring the force to be transferred thru the seat of the girder. Since this eliminates the bending stresses in the top chord of the girders due to the gravity load continuity forces, the 8 or 16-kip available force is totally available to transfer the lateral load forces into the column. A top plate can be used in lieu of the tie angles. However, the top tie plate usually interferes with the joist seat. The use of the continuity tie increases the strength of the Basic Connection to resist lateral loads. However, the cost of field welding the tie in place is significant.

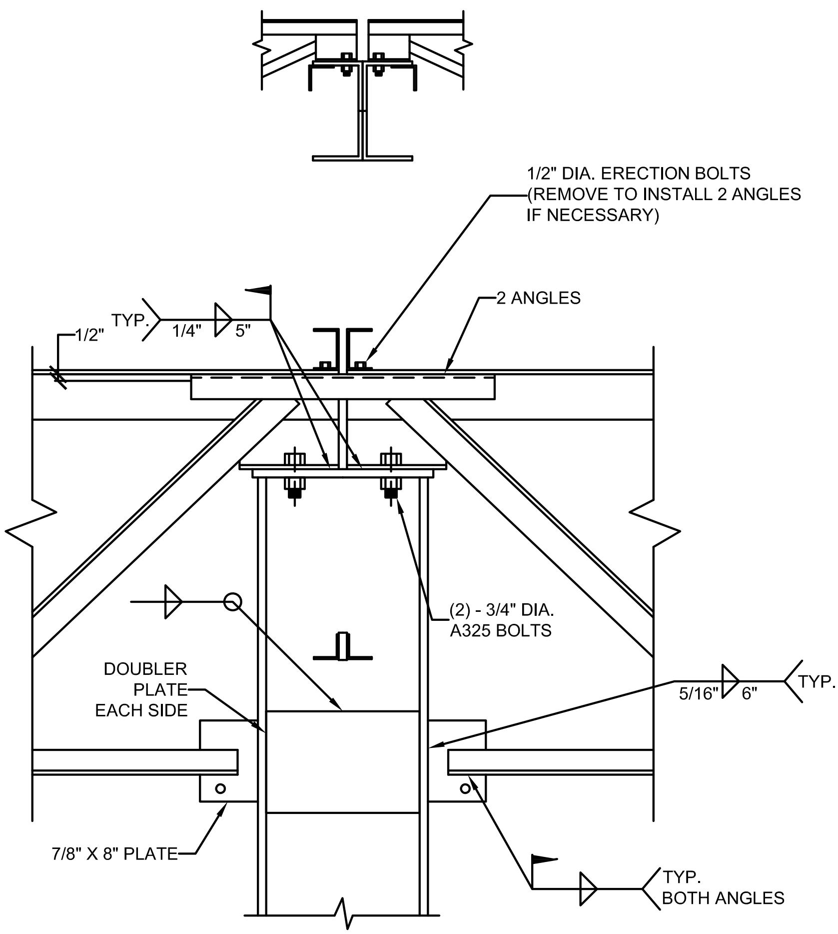



**Fig. 4.5.3 Detail B - The Welded Basic Connection with Tie Angles**

*Detail C - (Figure4.5.4)*

Joist Girder top chords can be reinforced during manufacturing by inserting a one-inch thick bar between the top chord angles or by extending the seat angles along the top chord of the girder. Both types of reinforcement serve to increase the eccentric load bending resistance of the top chord and must extend past the first vertical web member in the girder. This type of detail is expensive and should be specified by the specifying professional sparingly. Shown in Figure 4.5.4 are both types of reinforcement. Based upon practical weld sizes and lengths available to connect the Joist Girders to the columns, the chord force should be limited to approximately 50 kips for ASD and 75 kips for LRFD.

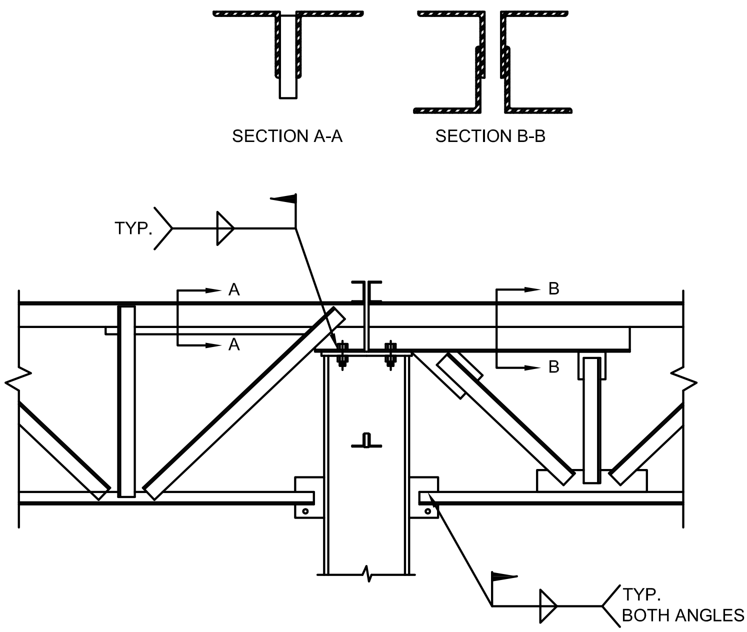



**Fig. 4.5.4 Detail C - Reinforced Chords**

*Detail D - (Figure 4.5.5)*

The special girder seat condition shown in Figure 4.5.5 is best used at sidewall columns. To be effective, the bolts in the seat must be separated by more than 6-inches. A practical chord force limitation for this detail is 40 kips for ASD and 60 kips for LRFD. Because the seat is designed as a rigid extension of the column, the force couple lever arm for this connection is the distance between the centroid of the top chord angles and the centroid of the bottom chord angles.

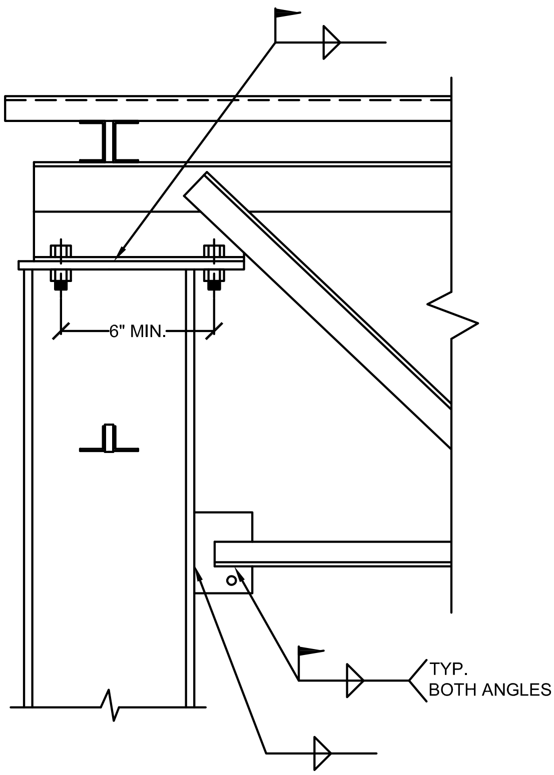



**Fig. 4.5.5 Detail D - Rigid Seat Connection**

*Detail E - (Figures. 4.5.6 and 4.5.7)*

Detail E can develop larger Joist Girder chord forces than Details A thru D. They also have the advantage of giving the specifying professional more control over the design, thus less coordination with Vulcraft is required. A disadvantage is that a support bracket is required on the side of the column for the Joist Girder seat for the moment plate to be welded to the column. The detail shown in Figure 4.5.7 is the most effective connection for multistory frames because the additional support is necessary on continuous columns to support Joist Girders. The force couple lever arm for these details is the distance between the centroid of the top chord angles and the centroid of the bottom chord angles. The strength of Detail E is limited by the top chord axial load capacity. Maximum force limits of 200 kips (ASD) and 300 kips (LRFD) are recommended for this detail.

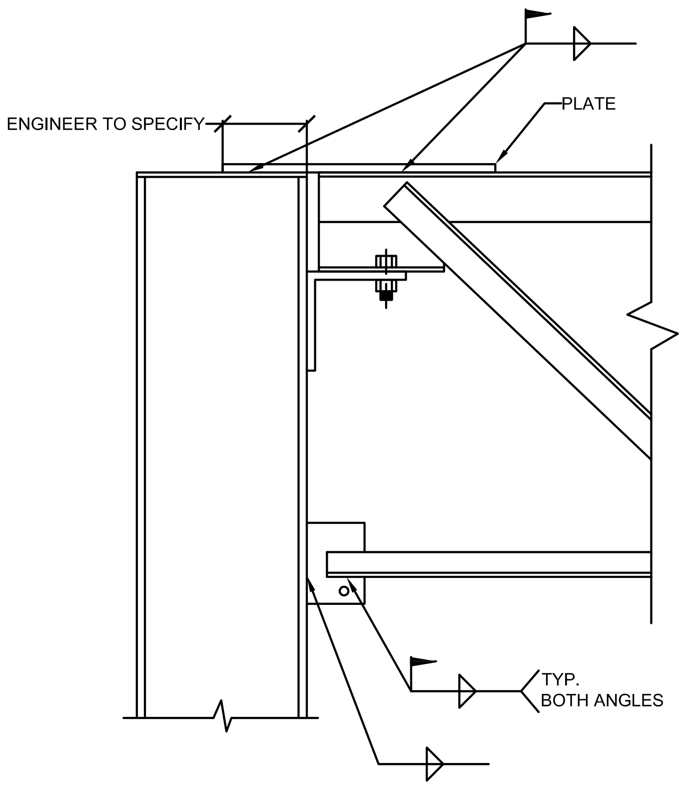



**Fig. 4.5.6 Detail E - Moment Plate Connection (Single Story)**

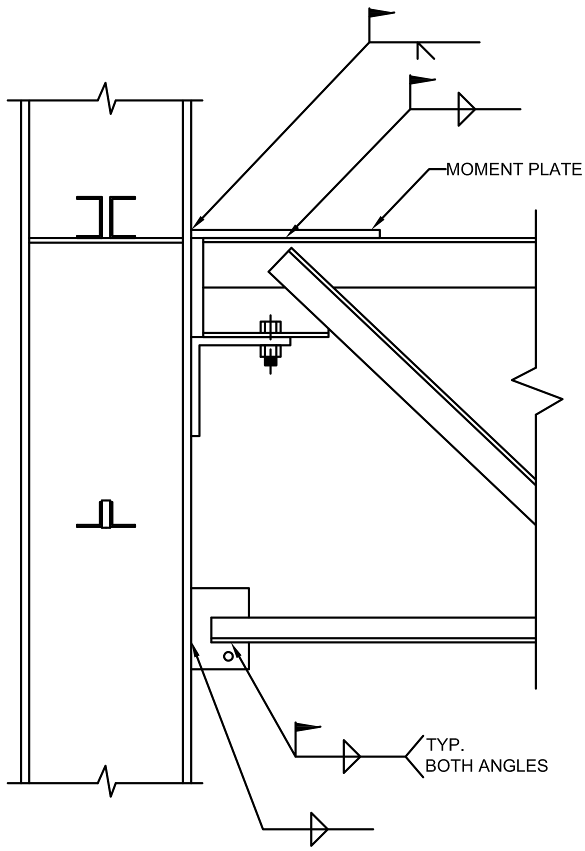



**Fig. 4.5.7 Detail E - Moment Plate Connection (Multi-story)**

*Detail F - (Figures 4.5.8a and 4.5.8b)*

Modifications to Detail E are shown in Figures 4.5.8a and 4.5.8b. Knife plate connections have been used successfully in single story and multistory moment frames. An advantage of the knife plate connection is that it eliminates some of the field welding required with Detail E. A disadvantage is that Vulcraft must give special attention in the Joist Girder seat design to accommodate the knife plate. Welding to the column cap should be partial pen or full pen rather than fillet welded, so that the seat on the Joist Girder can fit tight to the knife plate. For the multi-story connection, welding the knife plate to the column bracket will also interfere with the Joist Girder seat. The knife plate should be shop welded only to the column flange to avoid interference. Since the girder is being fitted over the knife plate in the field, use of a 7/8 in. thick knife plate should be considered to allow for field and fabrications tolerances. The force couple lever arm is the distance between the chord centroids.

A chord load limit of 200 kips for ASD and 300 kips for LRFD is recommended for this detail. A more detailed discussion of the knife plate connection is contained in Chapter 7.

The specifying professional is encouraged to contact the local Vulcraft sales engineer regarding the use of the knife plate connection.

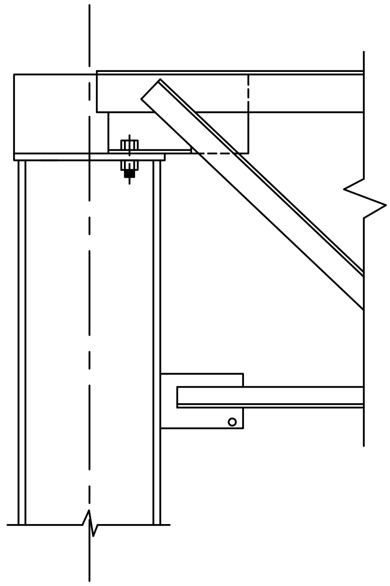


**Fig. 4.5.8a Detail F - Knife Plate Connection (Single Story)**

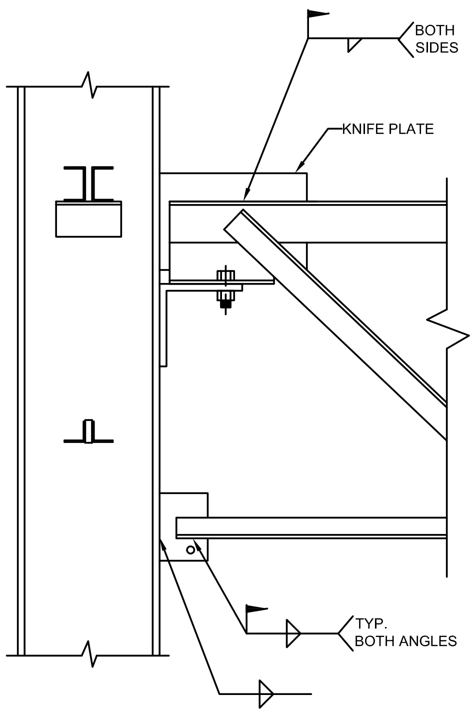



**Fig. 4.5.8b Detail F - Knife Plate Connection (Multi-story)**

## Joist Details

In the details presented below the building specifying professional must check the columns to determine if column web stiffening is required. The design of the welds which connect the joists to the columns is the responsibility of the building specifying professional. See Chapter 7 for further details.

*Detail G -- (Figure 4.5.9)*

The basic gravity load connection for joists becomes a moment connection when the bottom chords of the joists are extended and welded to the column. The capacity of this connection to resist joist moments is minimal. As shown in Chapter 7 the rollover capacity of standard Joist Girder seats is 5.89 kips (ASD), and 8.85 kips (LRFD). Use of the rod bottom chord extensions reduce the continuity chord forces as explained in Chapter 7.

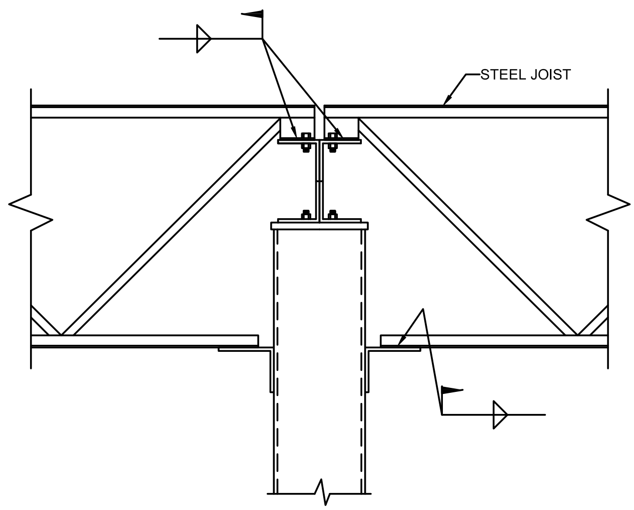



**Fig. 4.5.9 Detail G - The Basic Connection**

*Detail H -- (Figure 4.5.10)*

Use of the continuity tie plate provides a force path for the continuity forces to be transferred from joist to joist without requiring the force to be transferred thru the joist seats, thus eliminating the continuity secondary moments in the joists.

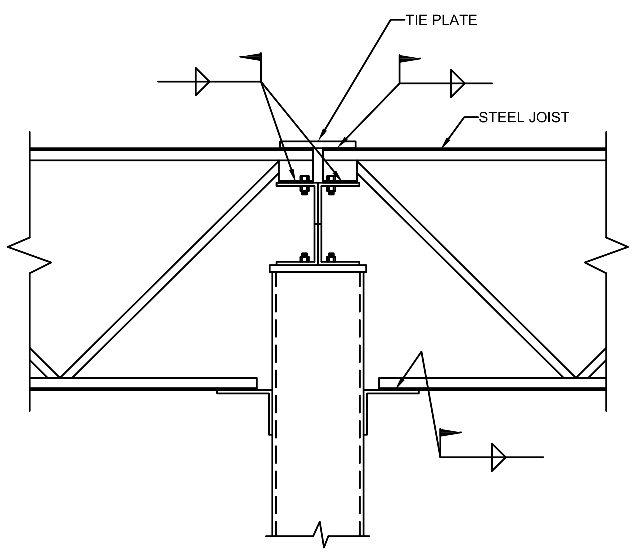



**Fig. 4.5.10 Detail H - The Basic Connection with Tie**

*Detail I - (Figure4.5.11)*

Placing stiffeners in the Joist Girder seat increases the rollover capacity to 8.55 kips (ASD), and 12.85 kips (LRFD), thereby increasing the detail's capability in transferring lateral load moments to the column. Because the cost of providing these stiffeners by Vulcraft is significant, an alternative to Detail I should be used.

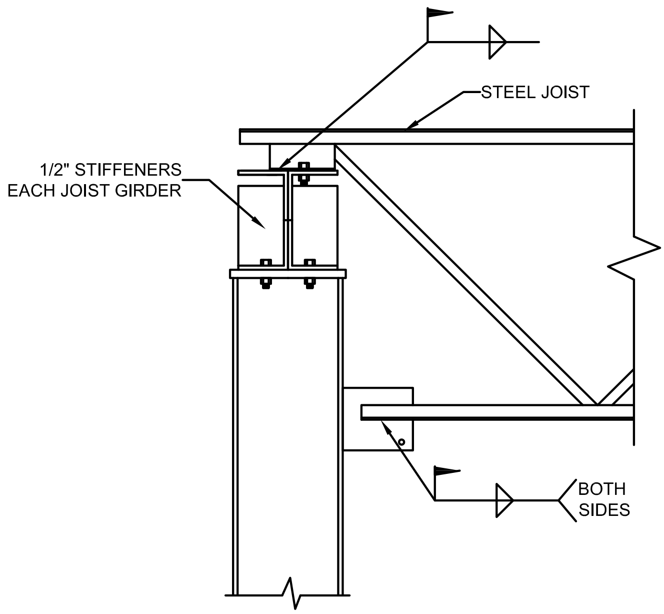



**Fig. 4.5.11 Detail I - Stiffeners in Girder Seat**

*Detail J - (Figure 4.5.12)*

Joist top chords can be reinforced by extending the seat angles along the top chord as shown in Figure 4.5.12. This significantly increases the chord capacity to resist bending forces; however, the detail is expensive to fabricate and the economics of its use by the building specifying professional must be carefully considered.

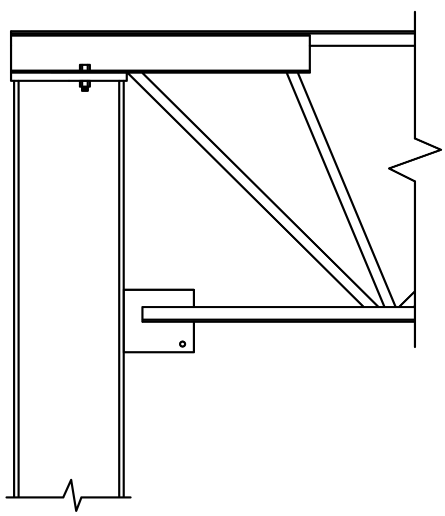


**Fig. 4.5.12 Detail J - Joist E Member Extension**

*Detail K- (Figure 4.5.13)*

The bracketed connection shown previously in Figure 4.5.6 can also be used for joists. It provides the same advantages for joist connections as it does for Joist Girders. The force couple lever arm is from the centroidal distance between the joist chords.

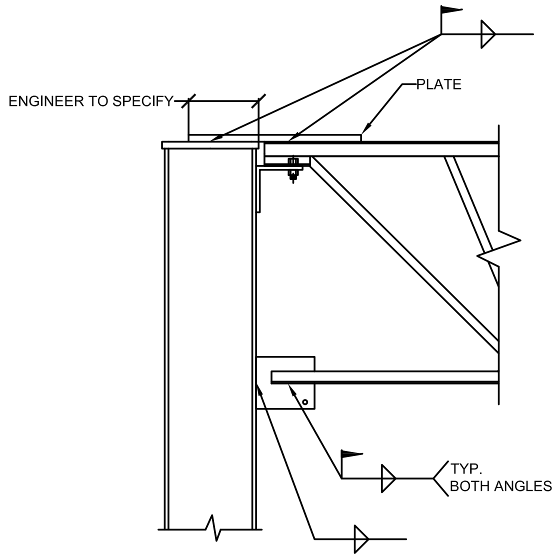



**Fig. 4.5.13 Detail K – Moment Plate Connection (Single Story)**

## Frame Analysis

Chapter C in the AISC Specifications (AISC, 2016d) provides stability and strength analysis provisions for the design of rigid frames. Chapter C is based on the direct analysis method, which can be used for all framing cases. Alternative methods of analysis are also permitted, specifically the effective length method and the first-order method which are presented in Appendix 7 of the Specifications; however, certain restrictions apply to these methods. Most software programs provide analysis solutions which satisfy the AISC analysis and design requirements. It is beyond the scope of this book to discuss all the analysis methods.

If one performs the required second order analysis which accounts for $P\Delta$ and $P\delta$ displacements, then Equations H1-1a and H1-1b in Chapter H of the Specifications can be used directly for determining the column requirements. The advantage of using second order analysis techniques is that effective length values are not required, i.e. $L_c = K$ can be taken as 1.0.

Analysis methods require that, for the lateral force resisting system, the moments of inertia and cross-sectional areas of joists, Joist Girders and columns be input into the analysis.

For joists, the moment of inertia is easily approximated from the joist load tables. The Vulcraft Manual provides the following equation for calculation of the joist moment of inertia:

$I_j = 26.767\,(W_{LL})(L^3)(10^{-6})$

where

$W_{LL}$ (lbs./ft.) equals the live load causing a deflection of span over 360 per the joist load table, in.[4]

$L$ equals (span - 0.33), in feet

The moment of inertia does not include a reduction for web deformations. For a stiffness analysis or for deflection calculations divide Ij by 1.15 to obtain an approximate effective moment of inertia $I_{eff} = I_j/1.15$.

For Joist Girders, an approximate moment of inertia can be obtained from the equation:

$I_{JG} = 0.027\ NPLd$

where

$N$ = number of joist spaces

$P$ = panel point load in kips

$L$ = Joist Girder length in feet

$d$ = effective depth of the Joist Girder in inches

The effective moment of inertia for the Joist Girders should again be divided by the 1.15 factor.

$I_{eff} = I_{JG}/1.15$

There are several ways to model joist or Joist Girder in Ordinary Moment Frames (OMF).

1. Using any stiffness analysis program, the specifying professional can perform the following:

   Forces and moments in moment frames comprised with joists or Joist Girders can be determined in a manner like other ordinary moment frames comprised of steel columns and beams. The first step in the design process is to perform a preliminary analysis. It is suggested that the OMF be considered as a pinned base frame in order to eliminate moment resisting foundations. However, for drift control, partially restrained or fixed bases can be considered. The specifying professional should consider serviceability criteria and drift control at the preliminary design phase of the project. After selecting trial member sizes for the columns and joists (the term joist as used here means joist or Joist Girder), a computer analysis is performed to determine forces, moments and deflections (both first order and second order) for the load combinations prescribed by the applicable building code. Because second order analysis is a non-linear problem, the analysis must be performed for each required load combination.

   It is suggested to use a simplified model for the joist frame by modeling the joist as an equivalent beam section with the effective moment of inertia as mentioned above. The node at the interface of the column and joist should be located at the mid-height of the joist to more closely approximate the relative stiffness of these two elements and to more accurately predict lateral drift in the frame. This model will provide conservative results for the moments in the joists and columns since the joists are attached to the columns at their bottom chord.

   The reader is encouraged to refer to SJI's Technical Digest 11, "Design of Lateral Load Resisting Frames Using Steel Joists and Joist Girders" for additional information for the design of Ordinary Moment Frames.

2. Commercially available software packages that make use of SJI's Virtual joists or Joist Girders can be used. Virtual joists and Joist Girders were developed by the SJI to facilitate use in software that was designed to use tables for wide flange beams. Tables "in like format" were developed for software suppliers. Several suppliers are currently making use of the tables.

The reader can contact any software supplier of their choice to determine how to use the Virtual joists and Joist Girders. One can also visit the SJI website www.steeljoist.org and view Webinars on the use of Virtual joists and Joist Girders.

To insure the joist or Joist Girder supplied for the project will meet the requirements used in the frame analysis, the moment of inertia for the joist ($I_j$) or the Joist Girder ($I_{JG}$) should be specified on structural plans as the minimum moment of inertia (i.e. $1.15I_{eff}$ used in the analysis).

### Drift Considerations

As with any structure, the stiffness of the frame must be considered, and drift must be controlled not only for strength considerations but also for serviceability requirements. The drift for most single-story industrial warehouse buildings with metal wall panels is generally in the range of the height/60 to height/100. More stringent requirements may be necessary depending upon the wall system, or if overhead crane runway systems are used. Typical drift limitations are provided in Table 4.5.1 for single story industrial buildings.

For multistory frames, most specifying professionals limit wind drift to $H/500$. Where $H$ is the story or building height. Wind forces are based on 10-year recurrences which result in pressures of approximately 3/4 of 50-year recurrences.

"Serviceability Design Considerations for Low- Rise Buildings, Steel Design Guide Series 3," is an expanded treatise on serviceability requirements in low rise steel buildings. See also the provisions in ASCE7 for drift considerations with seismic loads.

**DRIFT CRITERIA FOR STRUCTURAL ELEMENTS**

| Structural Element | Drift Criteria | Loading |
|---|---|---|
| Metal Panels | H/60 to H/100 | 10 year. wind |
| Precast Walls | H/100 | 10 year. wind |
| Unreinforced Masonry Walls | * 1/16, crack width at wall base | 10 year. wind |
| Reinforced Masonry Walls | * H/200 | 10 year. wind |
| Cab Operated Crane | H/240 | Crane lateral load or 10 year. wind. |
| Top Running Pendant Cranes | H/100 | Crane lateral load or 10 year. wind |

*Note: These drift indices can be increased with proper detailing. See, AISC's "Serviceability Design Considerations for Low- Rise Buildings, Steel Design Guide Series 3."

**Table 4.5 Drift Criteria**

## 4.6 SELECTION OF THE LATERAL LOAD SYSTEM

The various methods of resisting the lateral loads have been discussed in the previous sections. The three basic options are:

1. Roof and floor diaphragm systems with braced frames
2. X-Braced roof systems with braced frames
3. Diaphragms and moment frames

The systems can be mixed to provide the optimum structure. For example, diaphragms with moment frames in one direction and braced frames in the perpendicular direction

The choice of the most economical lateral load system is dependent on several parameters. These principally include:

1. The building end use
2. The building geometry
3. Expansion joint requirements
4. The type of roofing system
5. Future expansion requirements

As a rule braced frames with horizontal roof or floor diaphragms provide the most economical framing system for joist and Joist Girder buildings. This should be the specifying professional's first choice as a system. The parameters listed above can cause a different framing system to be used.

### Building End Use

The end use of a building may affect the lateral system to be used for the building. As previously mentioned, it might be necessary to have all interior vertical lateral force resisting systems be moment frames so there are no obstructions inside the building. The windows in an office building might dictate the locations a brace frame system can be used or force a moment frame to be used. Truck doors on a warehouse project might require that shear walls be used on that wall line. These are just a few examples of how end use might affect the lateral system.

### Building Geometry

As mentioned in the discussion on diaphragms and horizontal bracing when the length to width ratio of the structure exceeds approximately 4 to 1, the structural requirements placed on the diaphragm or horizontal bracing system become severe. This may require that interior frame lines be used to reduce the length to width ratio and the loading. In addition, uplift forces due to overturning become significant at frame locations. For these structures, depending on the building size, the most economical approach may be to create moment frames with Joist Girders at some or all the frame lines. Likely, the Basic Connection will not be suitable, since relatively few interior columns would be available to participate in the moment frames. Thus, the Joist Girders should be rigidly connected to the exterior columns only, and the Basic Connection used with any interior columns. For lateral loads in the perpendicular direction of the building, the first choice would be to use a diaphragm to transfer the lateral loads to the frame lines at the perimeter of the building.

### Expansion Joints

When the structure is of such a size that expansion joints are required, the expansion joints prevents the entire roof from acting as a single diaphragm. The roof would then act as separate diaphragms each side of the joint. If the diaphragm shears can be transferred across a singular expansion joint, an interior frame line would only be required on one side of the joint. For large diaphragm shears that cannot be transferred across the joint, interior frame lines would be required each side of the joint.

### Roofing System

When a standing seam roof is used, typically a horizontal roof bracing system is used. For the vertical system, braced frames or moment frames are typically be used. Again, if the length-to-width ratio is greater than 4 to 1, the moment frame system will most likely be the least expensive. If the loads and size of the building are such that the Basic Connection can be used without modification, then the moment frame system will probably be less expensive than the braced frame system. The moment frame solution will most likely have heavier columns than an X-braced frame system, but the erection cost of the X-bracing may be more expensive than the extra cost for the columns.

### Future Expansion

Usually, future expansion considerations only influence the vertical lateral force resisting system. The frame line at the expansion may have to be designed for additional load if the expansion is going to connect to the existing structure. The number of bays used for a braced frame may be limited at the expansion location. It may even require a moment frame system to be used at the expansion location.

## 4.7 SEISMIC CONSIDERATIONS

Joists or Joist Girders are commonly used to resist some seismic forces. Joists and Joist Girders can be used as the beam in ordinary moment frame type systems, the beam in ordinary braced frame type systems, as chords/collectors of the diaphragm system and continuous ties in the wall anchorage system. When designing a structure to resist seismic forces, the engineer must first select a Seismic Force Resisting System (SFRS). ASCE 7 Table 12.2-1 “Design Coefficients and Factors for Seismic Force-Resisting Systems” lists distinct types of Seismic Force-Resisting Systems.

### Moment Resisting Frame Systems

For steel moment frame systems, there are four types applicable to steel framed structures:

1. Steel Special Moment Frames (Type C1, SMF)
2. Steel Special Truss Moment Frames (Type C2, STMF)
3. Steel Intermediate Moment Frames (Type C3, IMF)
4. Steel Ordinary Moment Frames (Type C4, OMF)

The Structural System Selection noted in Chapter 12 of ASCE 7-16 provide the limitations for when each of these frame types may be used in a structure. These limitations are based on the applicable Seismic Design Category, height, and mass of the structure. Each of these four systems must be designed to meet the applicable requirements of the AISC 341 “Seismic Provisions for Structural Steel Buildings,” (AISC, 2016c).

Special Moment Frames and Intermediate Moment Frames require the use of a moment connection between the beam and column that has been demonstrated by testing to allow for varying degrees of inelastic rotation without significant degradation in the flexural strength of the two members. Tests have not been conducted on the connections typically used for joist or Joist Girders for compliance with the requirements for Special or Intermediate Moment Frames. Special Truss Moment Frames are required to provide significant inelastic deformation capacity within a special segment of the truss. These provisions require a specially designed and detailed truss to be used. Thus, a joist or Joist Girder moment frame must be categorized as an Ordinary Moment Frame in AISC 341.

Ordinary Moment Frames are expected to withstand minimal inelastic deformations in their members and connections when subjected to a design-level earthquake. Ordinary Moment Frames, that incorporate joists or Joist Girders as the beam, rely upon the column to provide this inelastic rotation. Therefore, the use of this type of system is limited to one-story buildings where the hinging of the column will not immediately create a stability problem. AISC 341 does provide requirements for the strength of the moment connection that need to be included in the design for frames resisting seismic loads. In higher seismic design categories, there are significant restrictions on the height and mass of the building for which OMF can be used. Due to these restrictions and the low Response Modification Coefficient for OMF, they are not used very often to resist seismic loads.

### Braced Frame Systems

For steel braced frame systems, there are four types applicable to steel framed structures:

1. Steel Eccentrically Braced Frames (Type B1, EBF)
2. Steel Special Concentrically Braced Frames (Type B2, SCBF)
3. Steel Ordinary Concentrically Braced Frames (Type B3, OCBF)
4. Steel Buckling-Restrained Braced Frames (Type, BRBF)

Like moment frame systems, the Structural System Selection noted in Chapter 12 of ASCE 7-16 provide the limitations for when each of these frame types may be used in a structure. The limitations again are based on the applicable Seismic Design Category, height, and mass of the structure. Each of these systems must be designed to meet the applicable requirements of the AISC 341.

Joists and Joist Girders cannot be incorporated into Steel Eccentrically Braced Frames. Steel Special Concentrically Braced Frames and Buckling-Restrained Braced Frames require details that are not practical for connecting joists and Joist Girders to Diagonal Braces in these systems. Thus, a joist or Joist Girder braced frame must be categorized as a Steel Ordinary Concentrically Braced Frame in AISC 341. In higher seismic design categories, there are significant restrictions on the height and mass of the building for which OCBF can be used. Due to these restrictions and the low Response Modification Coefficient for OCBF, they are not used very often to resist seismic loads.

The beams (joist or Joist Girder serving as the beam) and connections in the OCBF system must typically be designed for load combinations including overstrength. The provisions include certain exceptions to the overstrength requirement for the design of the connection.

### Steel Systems Not Specifically Detailed per AISC 341

For a structure in Seismic Design Category B or C, the engineer may select the SFRS category of “Steel Systems not Specifically Detailed for Seismic Resistance, Excluding Cantilever Column Systems.” These are commonly called “R=3” systems. Moment frames and braced frames that are part of an “R=3” system do not need to conform to AISC 341.

### Chords and Collectors

Joists and Joist Girders also act as chords and collectors in diaphragms for all types of Seismic Force Resisting Systems. These chords and collectors must be designed for the load combinations in the applicable building code including overstrength factor ($W_o$). The requirement to design for load combinations including overstrength factors applies to structures in Seismic Design Categories C and higher. The overstrength factor on large projects can often cause the drag load on collectors to exceed the capacity of the joist. The overstrength factor can also cause the drag load to be large enough that the connections to transfer the load between joists or Joist Girders become impractical. In these cases, a wide flange beam can be used for the collector members.

### Anchorage of Structural Walls

For projects with concrete or masonry walls, joists and Joists Girders are often used as part of the wall anchorage system. The wall anchorage forces are intended to prevent the concrete or masonry walls from pulling away from the structure. The horizontal seismic wall anchorage forces are axial loads on the joists and Joist Girders. The building code requires continuous ties or struts between diaphragm chords in order to distribute the anchorage forces into the diaphragm in higher seismic areas. By connecting the joists with tie plates or tie angles, and the Joist Girders with knife plates, continuous ties can be created. The seismic axial load on the joists and Joist Girders must be specified on the structural plans. The factor for Steel Elements of the Structural Wall Anchorage System in ASCE7 needs to be included in the axial load specified in higher seismic areas.

### Seismic Provision Requirements

To design and supply joists or Joist Girders as part of a SFRS per AISC 341, Vulcraft must know certain facts about the SFRS as designed by the specifying professional. The specifying professional is required to designate on the Structural Design Drawings, and/or in the Project Specifications, the items listed in Section A4.1 and Section A4.2 (as applicable) of the AISC 341.

Per Section A4.1 General, Structural design drawings and specifications shall "include the following as applicable:

(a) Designation of the SFRS

(b) Identification of the members and connections that are a part of the SFRS

(c) Locations and dimensions of protected zones

(d) Connection details between concrete floor diaphragms and the structural steel elements of the SFRS

(e) Shop drawing and erection drawing requirements not addressed in Section I1"

For systems designed according to the AISC 341, the joists or Joist Girders that are a part of the SLRS (including chords and collectors) must follow the following specific member, welding and bolting requirements.

- Section **A3.4a Consumables for Welding**: "All welds used in members and connections in the SLRS shall be made with filler metals meeting the requirements specified in clauses 6.1, 6.2, and 6.3 of Structural Welding Code-Seismic Supplement (AWS D1.8/D1.8M) …." [AWS, 2016]."
- Section **D1 MEMBER REQUIREMENTS:** "Members of moment frames, braced frames and shear walls in the seismic force-resisting system (SFRS) shall comply with the *Specification* and this section."
- Section **D2. CONNECTIONS Section D2.1 General:** "Connections, joints and fasteners that are a part of the SFRS shall comply with Specification Chapter J, and with the additional requirements of this section."
- Section **D2.2 Bolted Joints**
- Section **D2.3 Welded Joints**: "Welded Joints shall be designed in accordance with the AISC Specifications Chapter J."

The AISC 341 welding requirements are applicable to joist and Joist Girder chord splices, connections to columns in moment frames and braces in braced frames. AISC 341 also applies to the welded connections between joist and Joist Girder chords and web members with the exception that the welding be performed per SJI requirements. Section A3.4a applies and shall be followed in its entirety.

The AISC 341 bolting requirements require conformance with AISC "Specification for the Design, Fabrication and Erection of Structural Steel Buildings," and place additional restrictions on the bolt, hole types and allowable limit states for design. All bolts shall be designed as bearing bolts and must be installed as pretensioned high strength bolts with faying surfaces that satisfy the requirements for slip critical connections.

## 4.8 DESIGN EXAMPLES

### Example 4.8.1 Building Braced at Walls

Using ASD, design a bracing system for the building shown in Figure 4.8.1.

*Design the following:*

1. Metal Deck Diaphragm attachment for MWFRS wind loads
2. Diaphragm Chord/Collector Members
3. Shear Transfer at Lines 1 & 6
4. Specifications and Loading on members at Braced Frames
5. Diagonal Members of Braced Frames
6. Check Diaphragm Deflection
7. Final call outs for structural plans

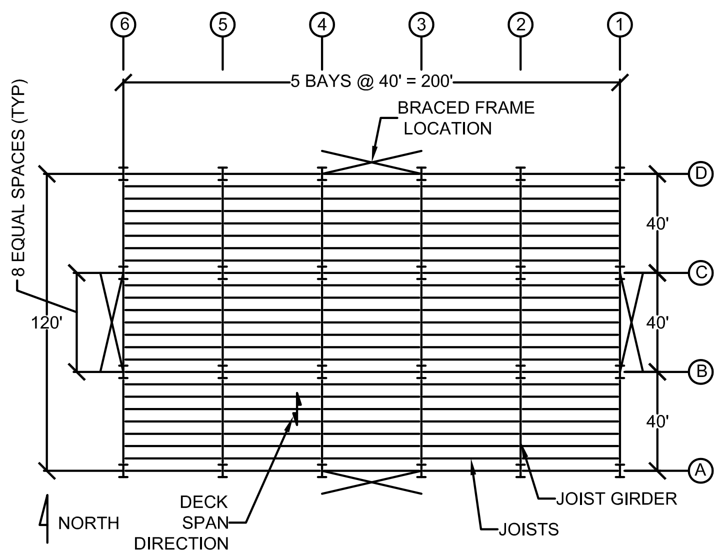



**Fig. 4.8.1 Example 4.8.1 Roof Plan View**

**Given:**

Frames are Ordinary Braced Frames with Tension only bracing.

Typical Joist: 22K7

Interior Joist Girder: 42G 8N 9.2K/6.0K

Perimeter Joist Girder: 42G 8N 4.6K/3. 0K

Deck: Vulcraft 22ga. 1.5B-36 Grade 50 (22 gage, 1.5" tall B deck, 36" wide)

Eave Height = 24 feet (top of metal roof deck)

Design Loads:

Dead Load, Joist, $D$ = 15 psf

Roof Live load, $L_r$ = 30 psf (Not reducible)

MWFRS loads:

Wall wind load, $0.6W$ = 20 psf (Windward + Leeward load)

Roof wind loads, $0.6W$ = -17 psf (negative sign signifies uplift load)

$0.6W$ = 0 psf

The specifying professional needs to check both the case with uplift loading and the case without uplift loading, since there may be instances where including uplift is the worst case, and other instances where the wind load being 0 psf will be the worst case.

Applicable load combinations:

$D + L$

$D + 0.6W$

$D + 0.75(0.6W) + 0.75\ L_r$

**Solution:**

1. Metal Deck Diaphragm attachment for MWFRS wind loads

*Diaphragm Loads: Based on MWFRS pressures*

The uniform load to the roof diaphragm is the reaction of the wind load on the wall. This building does not have a parapet, thus the diaphragm load is half the eave height times the wind pressure on the wall.

$w_w$ = (24 ft./2)(20 psf) = 240 plf

*Determine the wind load at each frame line:*

The roof diaphragm for this building is a flexible diaphragm. As a result, the roof diaphragm will behave as a simply supported beam. The wind load to the frames will be the end reaction of the simply supported beam uniform wind load times the half span of the diaphragm in the direction of the wind.

Lines 1 & 6, North-South direction:

$V_1 = V_6 = (w_w)(L/2)$ = (240 plf)(1 kip/1000 lb)(200 ft./2) = 24.0 kips

Lines A & D, East-West direction:

$V_A = V_D = (w_w)(L/2)$ = (240 plf)(1 kip/1000 lb) (120 ft./2) = 14.4 kips

*Determine the maximum wind shear force per foot in the diaphragm:*

The diaphragm depth for this building does not vary, so the worst-case shear will be at the frame lines. Diaphragm shear equals the wind load to the frame line divided by the diaphragm depth.

$v_1 = v_6 = (V_1)$/(Diaphragm depth N-S) = (24.0 kips)(1000 lbs/kip)/120 ft. = 200 plf

$v_A = v_D = (V_A)$/(Diaphragm depth E-W) = (14.4 kips)(1000 lbs/kip)/200 ft. = 72 plf

Note for buildings where the diaphragm depth varies, the specifying professional will need to check the diaphragm shear for each segment of the diaphragm with a different depth in order to determine the worst-case shear.

*Determine Diaphragm Chord forces:*

The diaphragm chord force is the tension-compression couple force from the bending moment in the diaphragm. The load for each wind direction must be determined.

North-South Direction, chord force at Line A and D

$M_{N\text{-}S} = w_w L^2/8$ = (240 plf)(200 ft.)$^2$/8 = 1200 kip-ft.

$P_{chord\ N\text{-}S} = (M_{N\text{-}S})$/(Diaphragm Depth) = (1200 kip-ft.)/(120 ft.) = 10.0 kips

East-West Direction, chord force at Line 1 and 6

$M_{E\text{-}W} = w_w L^2/8$ = (240 plf)(120 ft.)$^2$/8 = 432 kip-ft.

$P_{chord\ E\text{-}W} = (M_{E\text{-}W})$/(Diaphragm Depth) = (432 kip-ft.)/(200 ft.) = 2.16 kips

The above diaphragm wind loads will be used in sizing the members for the building.

The joist spacing can be determined from the information provided in Figure 4.8.1. The Joist Girder span is 40 feet. There are 8 equal joist spaces, thus the joist spacing = 40 ft./8 = 5.0 feet on center. The deck is Vulcraft 22ga. 1.5B-36. This preliminary size is based on the vertical loads, and the component and cladding wind loads on the deck. For diaphragm shear capacities the deck is to be installed with 2 spans minimum. The deck is being supported by joists, so the deck support members will be at least 1/8" thick (0.125"). The MWFRS uplift load on this roof diaphragm is 17 (0.6*W* psf).

For this building, use Hilti Powder Actuated Fasteners (PAF) for deck attachment to the supports and #10 screws for the side-lap connection. Screws used for side-lap connections need to be SDI recognized screws produced by Buildex, Elco, Hilti or Simpson Strong-Tie. The Vulcraft Roof Deck Diaphragm Design Tool should be used to determine the diaphragm capacity. The deck diaphragm tools can be accessed at www.vulcraft.com/design-tools. Figure 4.8.2 is the output from the Vulcraft 2018 IBC Deck Diaphragm tool based on the information and requirements above. This tool is based on the “North American Standard for the Design of Profiled Steel Diaphragm Panels,” (AISI S310-16, 2016b).

AISI S310-16 for the diaphragm capacity based on combined shear and wind uplift. Based on the tables in Figure 4.8.2, the diaphragm attachment can be chosen.

**22 ga 1.5B-36 Grade 50 Roof Deck**
Diaphragm Shear & Wind Uplift Interaction
with MWFRS Allowable Net Wind Uplift (ASD) of 17 psf

NUCOR VULCRAFT GROUP

Hilti X-HSN24 PAF Connections to Supports
36 / 4 Perpendicular Connection Pattern to Supports
#10 Screw Sidelap Connections

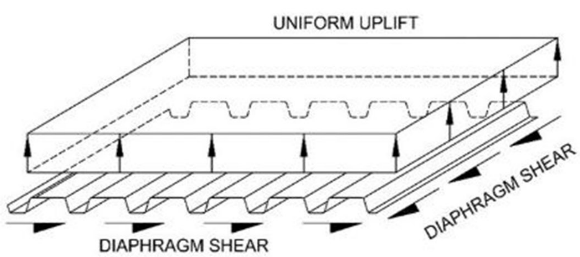
A572 GR50 Support Member or Equivalent
0.25 ≤ Support Thickness (in.) ≤ 0.375
2 in. Minimum Deck End Bearing Length

**ASD Allowable Combined Wind Uplift & Diphragm Shear Strength Sn/Ω (plf)** — 2 Span Condition

| Sidelap Connections per Span | Span 5'-0" | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 229 | 202 | 180 | - | - | - | - | - | - |
| 2 | 273 | 248 | 226 | 207 | 187 | 170 | 155 | 143 | 132 |
| 3 | 311 | 285 | 261 | 241 | 223 | 206 | 192 | 178 | 166 |
| 4 | 344 | 317 | 293 | 271 | 252 | 235 | 219 | 205 | 192 |
| 5 | 372 | 345 | 320 | 298 | 278 | 260 | 243 | 228 | 215 |
| 6 | 397 | 369 | 345 | 322 | 302 | 283 | 266 | 250 | 235 |
| 7 | 417 | 390 | 366 | 343 | 322 | 303 | 285 | 269 | 254 |

**Average Connection Spacing to Supports at Parallel Chords & Collectors (in.)**

| Sidelap Connections per Span | Span 5'-0" | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 20 | 22 | 24 | - | - | - | - | - | - |
| 2 | 20 | 22 | 24 | 20 | 21 | 23 | 24 | 26 | 27 |
| 3 | 20 | 22 | 24 | 20 | 21 | 23 | 24 | 26 | 27 |
| 4 | 20 | 22 | 24 | 20 | 21 | 23 | 24 | 26 | 27 |
| 5 | 20 | 22 | 24 | 20 | 21 | 23 | 24 | 26 | 27 |
| 6 | 20 | 22 | 24 | 20 | 21 | 23 | 24 | 26 | 27 |
| 7 | 18 | 20 | 18 | 20 | 21 | 23 | 24 | 20 | 22 |




**Seismic or Wind Diaphragm Shear Stiffness, G' (kip/in.)** — 2 Span Condition

| Sidelap Connections per Span | Span 5'-0" | 5'-6" | 6'-0" | 6'-6" | 7'-0" | 7'-6" | 8'-0" | 8'-6" | 9'-0" |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 9 | 9 | 10 | - | - | - | - | - | - |
| 2 | 9 | 9 | 10 | 11 | 11 | 12 | 13 | 13 | 14 |
| 3 | 9 | 10 | 10 | 11 | 12 | 12 | 13 | 13 | 14 |
| 4 | 9 | 10 | 10 | 11 | 12 | 12 | 13 | 14 | 14 |
| 5 | 9 | 10 | 10 | 11 | 12 | 13 | 13 | 14 | 14 |
| 6 | 9 | 10 | 10 | 11 | 12 | 13 | 13 | 14 | 15 |
| 7 | 9 | 10 | 11 | 11 | 12 | 13 | 13 | 14 | 15 |

**Fig. 4.8.2 Deck Diaphragm Capacity & Shear Stiffness**

**Diaphragm Attachment:**

Hilti X-HSN24 PAF with a 36/4 Pattern at Perpendicular supports, (1) #10 side-lap screw per span, and Hilti X-HSN24 PAF at 20” on center (1'-8" on center) to parallel supports.

Allowable capacity $v_{allow}$ = 227 plf > 200 plf = $v_1$ - attachment is acceptable.

Due to the size of this building and the low diaphragm shears, only the 1 attachment pattern is required. Note, for large buildings, it is common to use multiple attachment patterns, where the patterns increase as the diaphragm shear increases. This is done to make the deck and its attachment as economical as possible.

2. Diaphragm Chord/Collector Members

*2.1 Check Diaphragm Chord/Collector Members in North-South Direction, Lines A and D:*

The typical joists at Line A and D will be used as the Diaphragm Chord/Collector members for the North-South direction. Determine the joist top chord axial forces in the edge joists based on the required load combinations to determine if the typical 22K7 is acceptable. The edge joist has a tributary load width of 2.5 feet.

Load Combinations:

$D + L_r$
$D + 0.6W$
$D + 0.75(0.6W) + 0.75L_r$

For dead load axial:

The required joist moment = (15 psf)(2.5 ft.)(40 ft.)$^2$/8 = 7,500 ft.-lbs = 7.5 kip-ft.
The required axial chord force $D$ = (7.5 kip-ft.)(12 in./21.0 in.) = 4.3 kips

where

The effective depth of the joist is taken as 21.0 in.

For live load axial:

The required joist moment = (30 psf)(2.5 ft.)(40 ft.)$^2$/8 = 15,000 ft.-lbs = 15.0 kip-ft.
The required axial chord force $L_r$ = (15 kip-ft.)(12 in./21.0 in.) = 8.6 kips

For wind load axial:

Axial load from wind loads will be the worst-case of the collector load and the diaphragm chord force. The worst-case collector load will be at Line 3 and Line 4, load will be the same (the joist that is part of the frame is a separate case and is addressed later in the example).

Collector Load = $0.6W$ = (80 ft.)(75 plf) = 6.0 kips

$P_{chord\ N\text{-}S} = 0.6W$ = 10.0 kips (controls)

Combined Axial Loads:

$D + L_r$ = 4.3 kips + 8.6 kips = 12.9 kips

$D + 0.6W$ = 4.3 kips + 10 kips = 14.3 kips

$D + 0.75(0.6W) + 0.75L_r$ = 4.3 kips + 0.75(10 kips) + 0.75(8.6 kips) = 18.3 kips (controls)

*Determine the allowable joist top chord axial force for the typical edge joist:*

For the 22K7 with a 40 ft. span the available gravity load = 231 plf

The available moment = (231 plf)(40 ft.)$^2$/8 = 46.2 ft.- kips.

The available chord force = (46.2 kip-ft.)(12 in./ft.)/(21.0 in.) = 26.4 kips

26.4 kips > 18.3 kips **o.k**.

This capacity is based on the wind axial loads being transferred from joist to joist with a tie plate or similar. If the load is transferred thru the seat, it will cause additional bending in the joist top chord and reduces the capacity.

**The 22K7 is adequate as the diaphragm chord/collector member at Lines A and D with tie plate to transfer wind axial load.**

*Size the tie plate and weld:*

The axial load may act in tension or compression, therefore design for the compression case.

Use a ¼-in. thick x 4-in. wide x 8 in. long tie plate with 1/8 in. fillet weld 3 in. long each side of the plate to each joist.

$R_n/\Omega = 11.1$ kips > 10.0 kips = $P_{chord\ N\text{-}S}$ - Tie plate and weld **o.k.**

See example 4.2.2 for how to calculate capacity of plate and weld.

*2.2 Check Diaphragm Chord/Collector Members in East-West Direction, Lines 1 and 6:*

Line 1 and Line 6 are girder lines. Due to the joist seat the roof deck is not attached directly to the girder. The following are two common ways to create a chord/collector member at girder lines. The first is to use a continuous shear transfer member (in some cases referred to as blocking), between the joists and welded to the girder, allowing the girder to be used as the chord/collector member (best for high diaphragm shears and seismic loads). The second is to use a continuous edge angle as the chord/collector member (best for small diaphragm shears).

With the small diaphragm shears on this building use a continuous edge angle as the chord/collector member. The diaphragm shear load path is from the deck to the continuous angle, then from the angle to the joist seat. The joist seat transfers the force to the girder based on the joist seat rollover strength. The axial load on the angle will be the worst-case of the diaphragm chord force and the collector load to transfer the shear to the joist seat.

Axial Load:

$P_{collector} = 0.6W$ = (Joist Spacing)(Diaphragm Shear) = (5 ft.)(200 plf) = 1.0 kips

$P_{chord\ E\text{-}W} = 0.6W = 2.16$ kips (controls)

Try an edge angle L2 ½ x 2 ½ x 3/16 with a full penetration butt weld at the splice.

$A = 0.818$ in.$^2$

$r_y = 0.771$ in.

*Determine the allowable capacity for the edge angle per AISC Specification Section E:*

To determine the slenderness ratio for the edge angle, use $r_y$. The welding of the angle to the joists and the attachment of the deck to the angle is assumed to prevent buckling about $r_z$. The joists are spaced 5 ft. on center, 60 in apart.

$L/r_y = 60$ in./0.771 in. = 78

$F_{cr}/\Omega = 15.6$ ksi

The allowable axial force = (15.6 ksi)(0.818 in.$^2$) = 12.8 kips > 2.16 kips – **o.k.**

**Use L 2 ½ x 2 ½ x 3⁄16 continuous angle at Lines 1 and 6 with a Full Penetration Butt weld.**

<u>3. Shear Transfer at Lines 1 and 6</u>

As noted above, the diaphragm shear is transferred from the edge angle to the joist seats. The joist seats then transfer the load using rollover down to the girder. As a result, the joist seat must be designed for the rollover force.

From the above section the edge angle collector load = 1.0 kips. From Section 7.6.1, the allowable rollover strength = 1.82 kips. (Based on 2x2x1/8 angle joist seats). As a result, it is possible to use the joist seat for rollover.

**Specify in the structural details that joist seats at Line 1 and 6 to be designed for the rollover force 0.6*W* = 1.0 kips.**

Note: For large diaphragm shears, the rollover force will easily exceed the seat capacity, so a shear transfer member between the joists should be used in those cases.

Collector Loads in Joist Girders at Lines 1 and 6:

The Joist Girders spanning from Line A to B and from Line C to D will act as collectors to bring the diaphragm shear to the Joist Girder that is part of the braced frame from Line B to C. These Joist Girders must be designed for the collector wind axial loads. Due to the wind axial load the Joist Girder designation must include the live load portion of the panel point load, as shown above in order to properly design the Joist Girder for the required load combinations. The collector length of all 4 of these girders is the same, 40 feet.

Joist Girder Collector Axial Load:

$P_{collector} = 0.6W$ = (Collector Length)(Diaphragm Shear) = (40 ft.)(200 plf) = 8.0 kips

**Design Joist Girders at Line 1 and 6, spanning from A to B and from C to D for axial load 0.6*W* = 8.0 kips.**

The collector axial load must be transferred from these Joist Girders to the Joist Girder in the Brace Frames. Shown in Table 7.1.1 is the maximum allowable axial load thru the Joist Girder seat of 10.0 kips, which is greater than the 8.0 kips axial load.

**Design the Joist Girder seat to transfer axial load.**

Note it may be possible to transfer a higher axial load through the Joist Girder seat. If the specifying professional desires to transfer axial loads, through the Joist Girder seat, that are greater than those given in Table 7.1.1, they should contact Vulcraft to evaluate the specific Joist Girder design and the loading.

4. <u>Specifications and Loading on members at Braced Frames</u>

*4.1 Specification and Loading for Joist at Line A and D in Braced Frame:*

The braced frame bay is in the middle bay for 2 reasons. First having the frame in the middle of the bay allows for thermal movement in both directions from the frame. The second reason is having the frame in the middle bay keeps the collector loads as small as possible.

For constructability concerns, one must avoid conflicts between the braced frame diagonal members and the members of the joist. This can be accomplished by connecting the bottom chord of the joist to a gusset plate on the column and connecting the diagonal braces to the gusset plate.

From the diaphragm chord analysis earlier in the example, the joists at Lines A and D were required to be a 22K7. For the joist at the frame, the joist must be called out to at least meet the requirements of a 22K7. Due to the connections of the joist at the frame, there are no other loading requirements for Vulcraft to include in the design.

*Determine the axial load for which the joist needs to be designed:*

The axial load will be the worst-case of the diaphragm chord load in the North-South direction and the collector load. The braced frames for this building are tension only bracing, as a result, the entire wind load for the frame must be collected to the top chord of the joist. Thus, the collector axial load to the joist is the frame load. The load at Line A and Line D are the same, so the same joist can be used.

$V_A = V_D$ = 14.4 kips (controls)

$P_{chord\ N\text{-}S}$ = 10.0 kips

Since the bottom chord of the joist is connected to gusset plates at the column, the wind load for the frame will need to be transferred from the top chord of the joist through the joist webs to the bottom chord of the joist. This needs to be noted on the structural plans so the webs are properly designed.

**Joist Wind Axial Load 0.6$W$ = $V_A$ = 14.4 kips, transfer axial load through webs to bottom chord.**

The detailing of the joist connections will also impact the design. The bottom chord of the joist is connected to a gusset plate at each end of the joist. In addition, the top chords of the joist are connected to the adjacent joist at each end. This will cause some fixity at the ends of the joist. If the bottom chords of the joist are not welded until after the dead load is installed, the worst-case end moments would only be due to the live load.

$w_{Lr}$ = (2.5 ft.)(30 psf) = 75 plf

$M_{Lr} = w_{LL}L^2/12$ = (75 plf)(40 ft.)$^2$/12 = 10.0 kip-ft. (specify this end moment on the plans)

Because the joist must be designed for axial loads, Vulcraft will also need to know the uniform loading that is to be included in the load combinations. This includes the uniform dead load and the MWFRS uplift loads. The following should be specified on the plans.

$w_D$ = (2.5 ft.)(15 psf) = 38 plf

MWFRS Uplift:Max $w_W$ = (2.5 ft.)(17 psf) = 43 plf

Min $w_W$ = (2.5 ft.)(0 psf) = 0 plf

*4.2 Specification and Loading for Joist Girder at Line 1 and 6 in Braced Frame:*

The braced frame bay at Lines 1 and 6 was in the middle bay for the same reasons the bracing at Lines A and D was located in the middle of the frame line. Just like with the joist at the braced frame, a gusset plate will be used at the bottom chord of the Joist Girder to transfer the axial load to the braced frame diagonal.

The detailing of the Joist Girder connections will also impact the design. The bottom chord of the Joist Girder is connected to a gusset plate at each end of the Joist Girder. The Joist Girder seat is welded to the column cap at each end of the Joist Girder. As a result of these connections, the Joist Girder will have fixity at the ends of the Joist Girder. If the bottom chords of the Joist Girder are not welded until after the dead load is installed, the worst-case end moments would only be due to the live load. The live load moment can be taken from the rigid frame analysis, since there is fixity at the ends of the joists.

From a rigid frame analysis:

$M_{Lr}$ = 209 in.- kips = 17.4 kip-ft. (this moment will need to be specified on the plans)

*Check Joist Girder seat for collector load and continuity load from live load moment:*

Collector load at seat = 0.6$W$ = 8.0 kips

Continuity load = $L_r = M_{Lr}$/(42 in – 0.5 in – 0.5 in) = (209 in-kips)/(41 in) = 5.1 kips

0.75$L_r$ + 0.75(0.6$W$) = 0.75(5.1 kips) + 0.75(8.0 kips) = 9.83 kips < 10 kips from Table 7.1.1, so it is possible to design the Joist Girder seat to create fixity at the column. Axial loads thru the Joist Girder seat will need to be specified on the plans, so the seat is properly designed.

*Determine the axial load for which the Joist Girder needs to be designed:*

The axial load will be the worst-case of the diaphragm chord load in the East-West direction and the collector load. The braced frames on this building use tension only bracing, as a result, the entire wind load for the frame must be collected to the top chord of the Joist Girder. Thus, the collector axial load to the Joist Girder will be the frame load. The load at Line 1 and Line 6 are the same, so the same Joist Girder can be used.

$V_1 = V_6 = 24.0$ kips (controls)

$P_{chord\ E\text{-}W} = 2.16$ kips

Since the bottom chord of the Joist Girder is connected to gusset plates at the column, the wind load for the frame must be transferred from the top chord of the joist girder through the webs to the bottom chord of the Joist Girder. This needs to be noted on the structural plans so the webs are properly designed.

**Joist Girder Wind Axial Load 0.6$W$ = $V_1$ = 24.0 kips, transfer axial load through webs to Bottom Chord.**

Because the Joist Girder must be designed for axial loads, Vulcraft will also need all of the loads that are to be included in the load combinations. This includes the live load at each Joist Girder panel point (which was shown in the original designation in the **Given** information) and the MWFRS uplift loads. The following should be specified on the plans.

MWFRS Uplift:Max $w_W$ = (40 ft./2)(17 psf) = 240 plf

Min $w_W$ = (40 ft./2)(0 psf) = 0 plf

5. Diagonal Members of Braced Frames

The braced frame widths are the same on all 4 sides of the building and the heights of the frames are similar. As a result, use the same steel section for all the braced frame diagonal members on this building. Since the braced frames are being designed as tension only frames, use angles for the diagonals in the frames. Bolt the diagonal angles together where they intersect. The centerline of the connection of the diagonals at the base of the column is 6 inches below the slab.

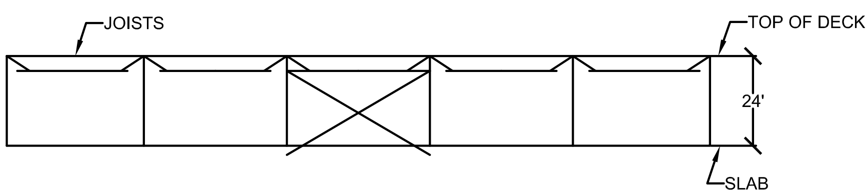



**Fig. 4.8.3 Schematic Elevation at Line A and D**

Requirements at Lines A and D: (loads to frames are the same)

Horizontal length of brace = bay width = 40 feet

Brace Height = 24 ft. + 0.5 ft. (connection below slab) - (1.5 in for deck) - (22 in - 0.5 in for centroid of joist bottom chord) = 22.58 ft.

Brace Length = [ (40 ft.)$^2$ + (22.58 ft.)$^2$ ]$^{1/2}$ = 45.93 feet

Horizontal component of brace load = wind load to frame = $V_A$ = 14.4 kips

Brace Load $T$ = ($V_A$)(Brace Length / Horizontal Length) = (14.4 kips)(45.93 ft. / 40 ft.) = 16.5 kips

*Determine the brace area required (A36 angle):*

Allowable tensile stress = $F_y/\Omega$ = 36 ksi/1.67 = 21.6 ksi

$A_{req'd}$ = 16.5 kips/21.6 ksi = 0.76 in.$^2$

*Determine the radius of gyration required:*

Due to tension only braces, use a maximum slenderness ratio of 300. For the in-plane slenderness ratio of single angles with bolt at intersection, use one half of the brace length divided by $r_z$. For the out-of-plane slenderness ratio use 0.75 times the brace length divided by $r_y$ (ASCE, 1987).

$r_{z\ (req'd)}$ = (45.93 ft./2) (12 in./ft.)/300 = 0.919 in. (controls)

$r_{y\ (req'd)}$ = (0.75)(45.93 ft.)(12 in./ft.)/300 ft.= 1.38 in. (controls)

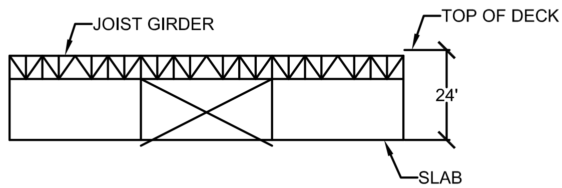



**Fig. 4.8.4 Schematic Elevation at Line 1 and 6**

*Requirements at Lines 1 and 6: (loads to frames are the same)*

Horizontal length of brace = bay width = 40 ft.

Brace Height = 24 ft. + 0.5 ft. (connection below slab) - (1.5 in for deck) - (2.5 in joist seat) - (42 in -1.5 in for centroid of Joist Girder bottom chord) = 20.79 ft.

Brace Length = [ (40 ft.)$^2$ + (20.79 ft.)$^2$ ]$^{1/2}$ = 45.08 ft.

Horizontal component of brace load = wind load to frame = $V_1$ = 24.0 kips

Brace Load T = ($V_1$)(Brace Length / Horizontal Length) = (24.0 kips)(45.08 ft. / 40 ft.) = 27.0 kips

*Determine the brace area required (A36 angle):*

$A_{req'd}$ = 27.0 kips/21.6 ksi = 1.25 in.$^2$ (controls)

*Determine the radius of gyration required:*

$r_{z\ (req'd)}$ = (45.08 ft./2) (12 in./ft.)/300 = 0.902 in.

$r_{y\ (req'd)}$ = (0.75)(45.08 ft.)(12 in./ft.)/300 ft.= 1.35 in.

**Use L5x5x5⁄16 Diagonals with bolt at intersection**

$A$ = 3.07 in.$^2$ > 1.25 in.$^2$ **o.k.**

$r_z$ = 0.990 in. > 0.919 in. **o.k.**

$r_x = r_y$ = 1.56 in. > 1.38 in. **o.k.**

Note, the connection of diagonal members to gusset plates, connection of joist and Joist Girder bottom chords and design of gusset plates are not covered in this example. For the gusset plates, since they are going in between the bottom chords of the joist and Joist Girder, the design professional will want to make sure the plate is wide enough. For Joist Girders, the gap is typically 1 in between the angles, so a ⅞ in plate should be used. For joists, the gap is often 1 in for larger joists and longer spans, so a ⅞ in plate should be used. For smaller K-Series joists with rod webs the gap may be less than 1 in, so the specifying professional will want to coordinate with Vulcraft to make sure the gusset plate is of the proper thickness.

6. Check Diaphragm Deflection:

The diaphragm deflection in the North-South direction will be the worst-case. It is the widest side of the building and the narrowest diaphragm. If the North-South direction is acceptable, the East-West direction will be acceptable by inspection. Diaphragm deflection has 2 parts, the shear deflection and the bending deflection.

The effective stiffness of the deck G' is given in the output tables of the 2018 IBC Deck Diaphragm tool, see Section 1 of this example. G' = 9 kips/in

The equation for the shear deflection is:

$$\Delta_s = \frac{wL^2}{8DG'} = \frac{(0.24\text{ kips/ft})(200\text{ ft})^2}{8(120\text{ ft})(9\text{ kips/in.})} = 1.11\text{ in.}$$

where

$w$ equals the diaphragm load = 0.24 kips/ft.

$L$ equals the diaphragm length = 200 feet

$D$ equals the diaphragm depth = 120 feet

The equation for bending deflection is:

$$\Delta_b = \frac{5wL^4}{384EI} = \frac{5(0.24\text{ kips/ft})(200\text{ ft})^4\,1728}{384(29{,}000\text{ ksi})(1{,}037{,}000\text{ in.}^4)} = 0.29\text{ in.}$$

where

$I = 2A(d/2)^2$ and is normally calculated using $A$ equal to the perimeter member area (diaphragm chord member). The area of the tie plate between chord member is 1.0 in.$^2$, the top chord area of a 22K7 joist will be greater. Use A = 1.0 in.$^2$

$$I = 2(1.0\text{ in.}^2)(120\text{ ft/2})^2\left(\frac{144\text{ in.}^2}{\text{ft}^2}\right) = 1{,}037{,}000\text{ in.}^4$$

Total Deflection:

$D_{max} = D_s + D_b$

$D_{\text{max}}$ = 1.11 in.+0.29 in. = 1.40 in.

The maximum deflection for a ten-year wind is 75% of the deflection for a 50-year wind.

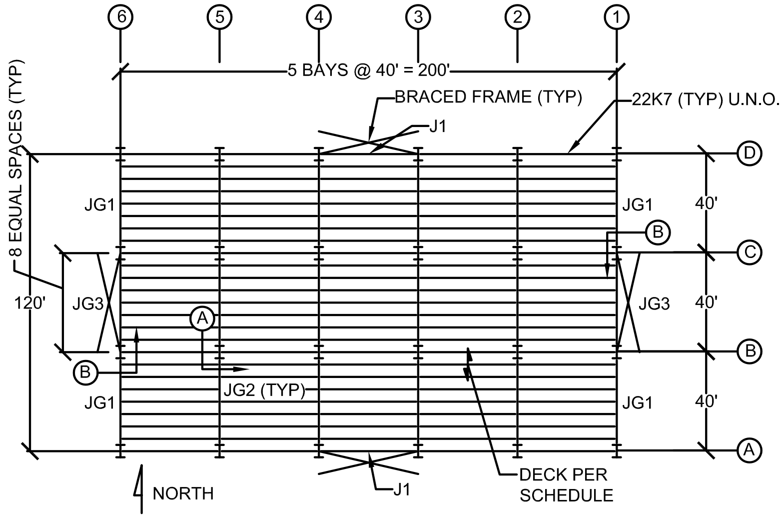
Therefore:

$D_{\text{max}}$ for a 10-year wind = 0.75(1.40 in.) = 1.05 in.

Allowable deflection = $H$/100 = (24 ft.)(12 in/ft.)/100 = 2.88 in > 1.05 in **o.k.**

7. Final call outs for structural plans:

The following figures and tables illustrate the specification of the Joists and Joist Girders for this example problem that should be included on the structural plans.




| METAL DECK SCHEDULE | | | | | |
|---|---|---|---|---|---|
| | | | | Hilti X-HSN 24 Fasteners | |
| Mfr | Deck Type (1) | Deck Gage | Min Side-lap connection (2) | Perpendicular Supports | Parallel Supports |
| Vulcraft | 1.5B-36 | 22 | (1) #10 TEK screw per span | 36/4 pattern | 1'-8" o.c. |

(1) Deck to be Grade 50
(2) Side-lap screws to be SDI recognized screws by Buildex, Elco, Hilti, or Simpson Strong-tie

**Fig. 4.8.5 Plan View and Deck Schedule for Final Design**

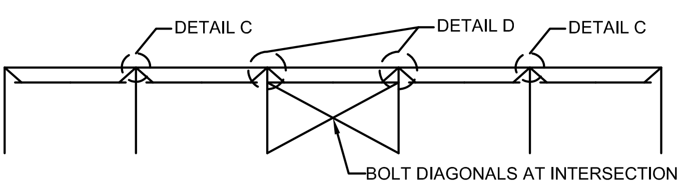



**Fig. 4.8.6 North Elevation and South Elevation**

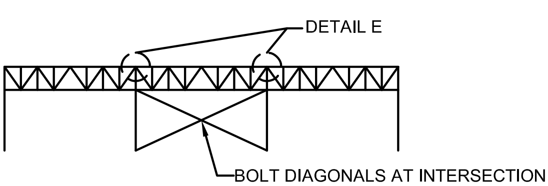
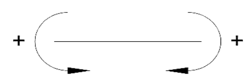



**Fig. 4.8.7 East Elevation and West Elevation**

| JOIST SCHEDULE | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Loads for Combined Bending and Axial Check (2) | | | | | | | | |
| | | Uniform Loads | | | | End Moments | | Axial Load (4) | | |
| | | | | MWFRS Load, 0.6W (3) | | Live Load Continuity Moment $L_r$ (5) | | | | |
| Joist Mark Number | Designation (1) | Dead Load | Roof Live Load $L_r$ | Max Uplift | Min Uplift | Left | Right | Wind Axial Load 0.6W | Add'l Requirement | Comments |
| J1 | 22K7 | 38 plf | 75 plf | 43 plf | 0 plf | 10.0 kip-ft | 10.0 kip-ft | 14.4 kips | Design Joist Webs to Transfer Axial load from Top Chord to Bottom Chord | Braced Frame Joist |

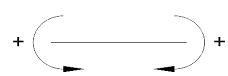
(1) Standard designation is minimum requirement. Joist Manufacturer to modify joist design as required for combined loading requirements.
(2) Joist manufacturer to use these load in the applicable code load combinations to design the joist for combined bending and axial.
(3) Joist Manufacturer to check joist design with both Maximum and Minimum MWFRS loads in load combinations for worst case loading.
(4) Tension or Compression Load. Axial load is transfered from Top Chord to Bottom Chord and into gusset plates for tension only Braced Frame.
(5) End Moment Sign Convention, Positive moments:




## Table 4.8.1 Joist Schedule at Braced Frame

| JOIST GIRDER SCHEDULE (1)(2) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | End Moments | | Axial Loads | | | | |
| | | MWFRS Load, 0.6W (3) | | Live Load Continuity Moment $L_r$ (8) | | | Seat Axial Load (5) | | | |
| Girder Mark Number | Designation (Total Load/Live Load) | Max Uplift | Min Uplift | Left | Right | Top Chord Wind Load 0.6W (4) | Wind Load 0.6W | Live Load $L_r$ (6) | Add'l Requirement | Comments |
| JG1 | 42G 8N 4.6K/3.0K | 240 plf | 0 plf | - | - | 8.0 kips | 8.0 kips | - | Seat Axial = 0.0 kips at Line A & D | |
| JG2 | 42G 8N 9.2K/6.0K | - | - | - | - | - | - | - | - | |
| JG3 | 42G 8N 4.6K/3.0K | 240 plf | 0 plf | 17.4 kip-ft | 17.4 kip-ft | 24.0 kips | 8.0 kips | 5.1 kips | Design Joist Girder Webs to Transfer Axial load from Top Chord to Bottom Chord | Braced Frame Joist Girder (7) |

(1) Deflection Criteria: Live Load Deflection ≤ L/240 .
(2) See framing plan for joist spacing along Joist Girder.
(3) Joist Girder Manufacturer to check Joist Girder design with both Maximum and Minimum MWFRS loads in load combinations for worst case loading.
(4) Axial load is Tension or Compression Load.
(5) Axial load to Joist Girder seat is already included in the Top Chord Axial Loads shown and does not need to be added.
(6) Seat Axial Live Load is due to Continuity End Moment.
(7) Axial load is transfered from Top Chord to Bottom Chord and into gusset plates for tension only Braced Frame.
(8) End Moment Sign Convention, Positive moments:




## Table 4.8.2 Joist Girder Schedule at Braced Frame

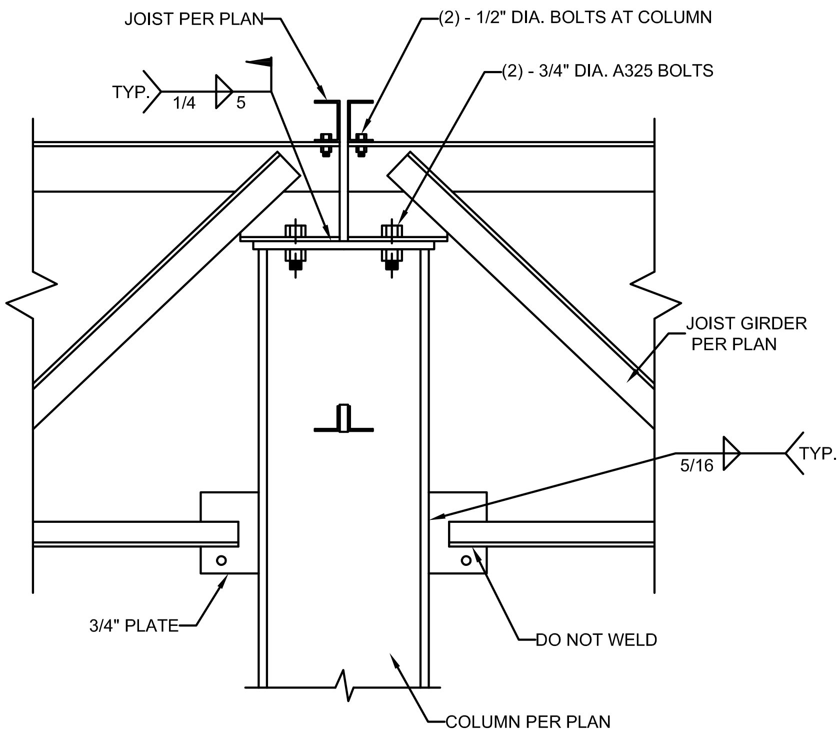



**Fig. 4.8.8 Detail A**

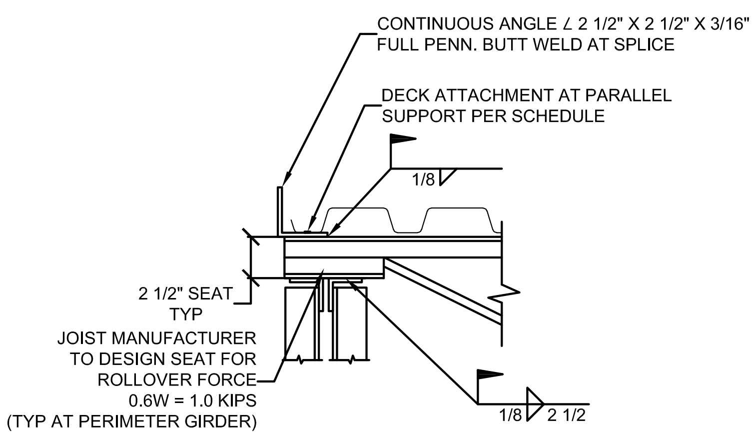



**Fig. 4.8.9 Detail B**

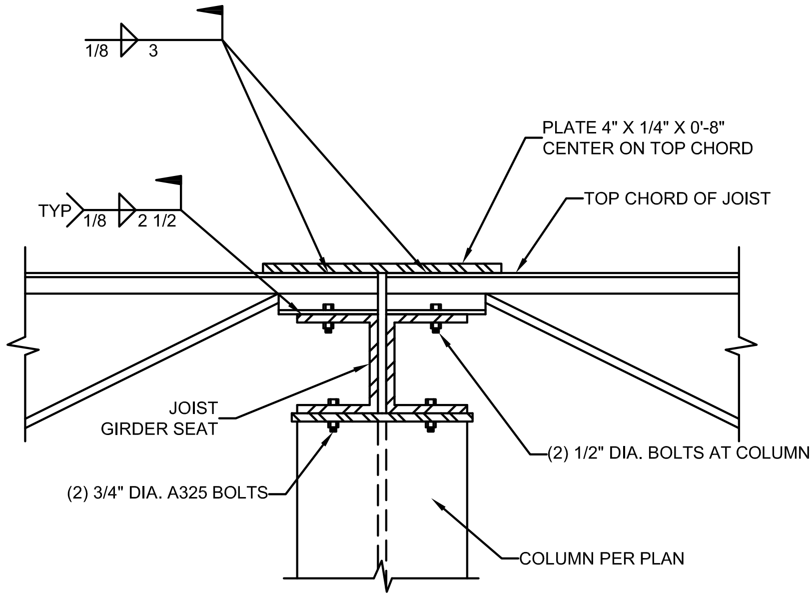



**Fig. 4.8.10 Detail C**

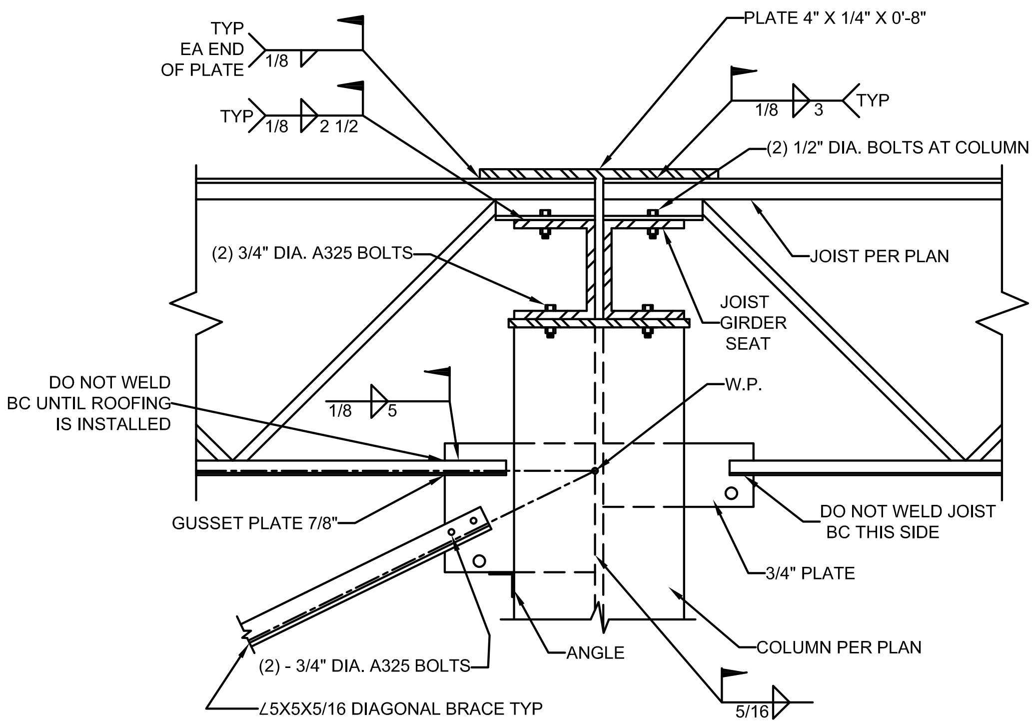



**Fig. 4.8.11 Detail D**

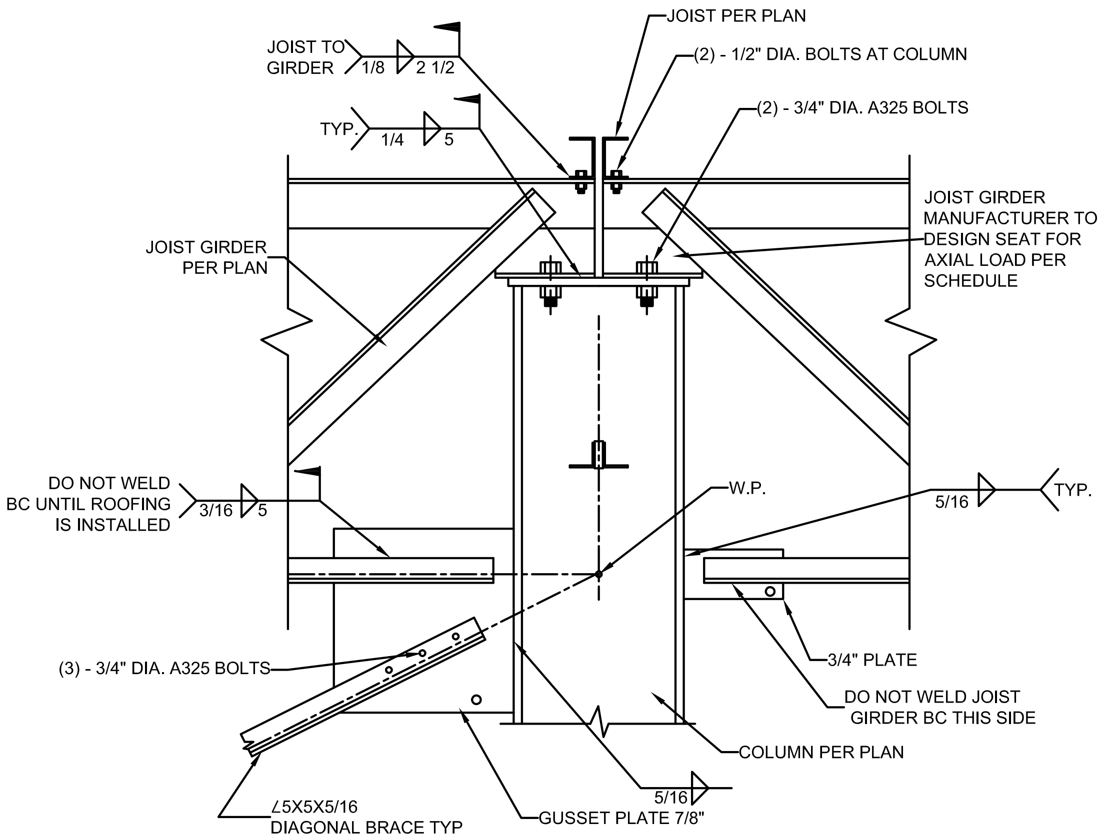



**Fig. 4.8.12 Detail E**

## Example 4.8.2 Rigid Frame Building

Using LRFD and Building shown in Figure 4.8.13.

*Design the following:*

1. Rigid Frame System with Joist Girders at Lines 2 through 7 (frames are identical)
2. Rigid Frame System with Joists at Lines B and C (interior frames, frames are identical)

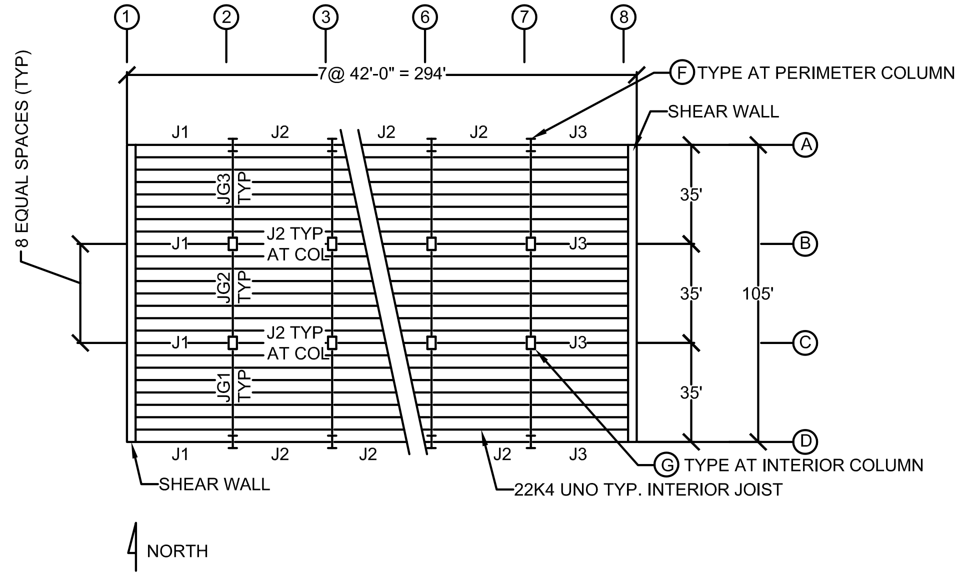
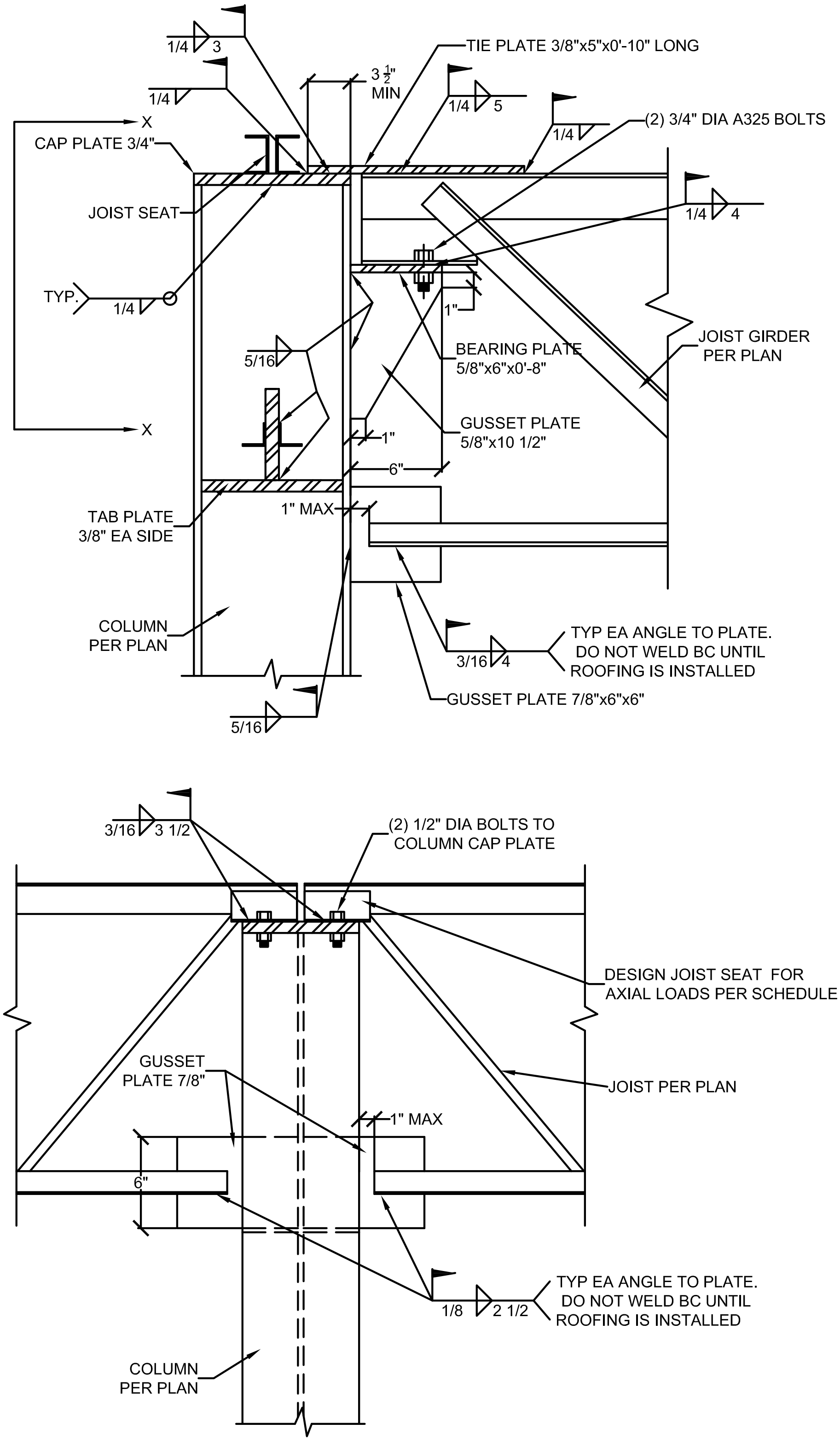


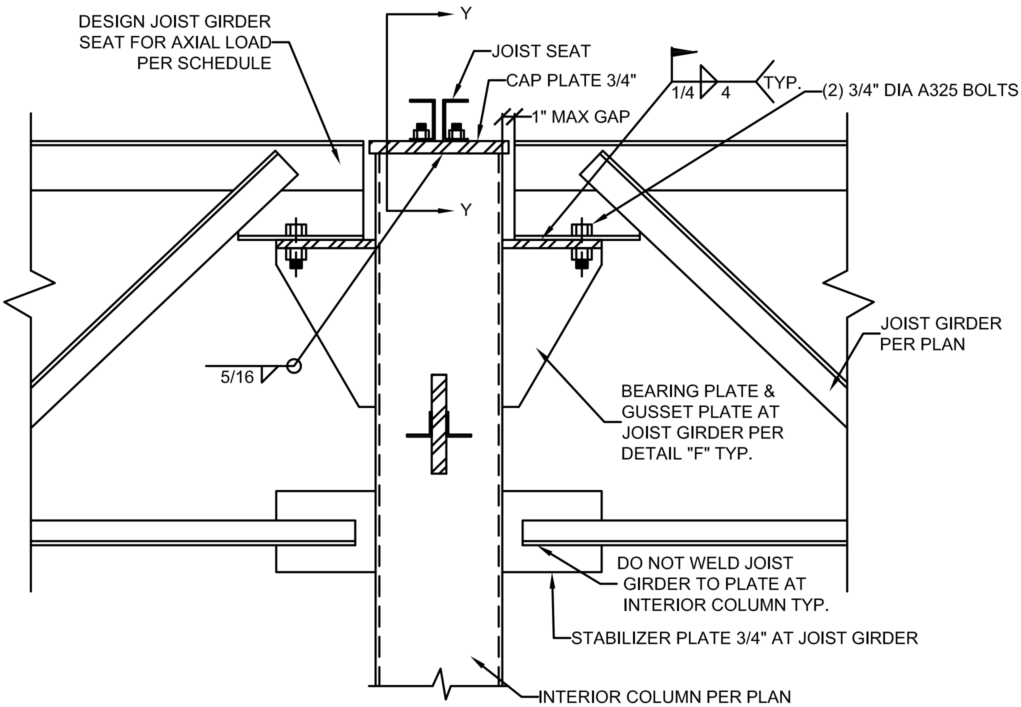

**Fig. 4.8.13 Example 4.8.2 Roof Plan View**



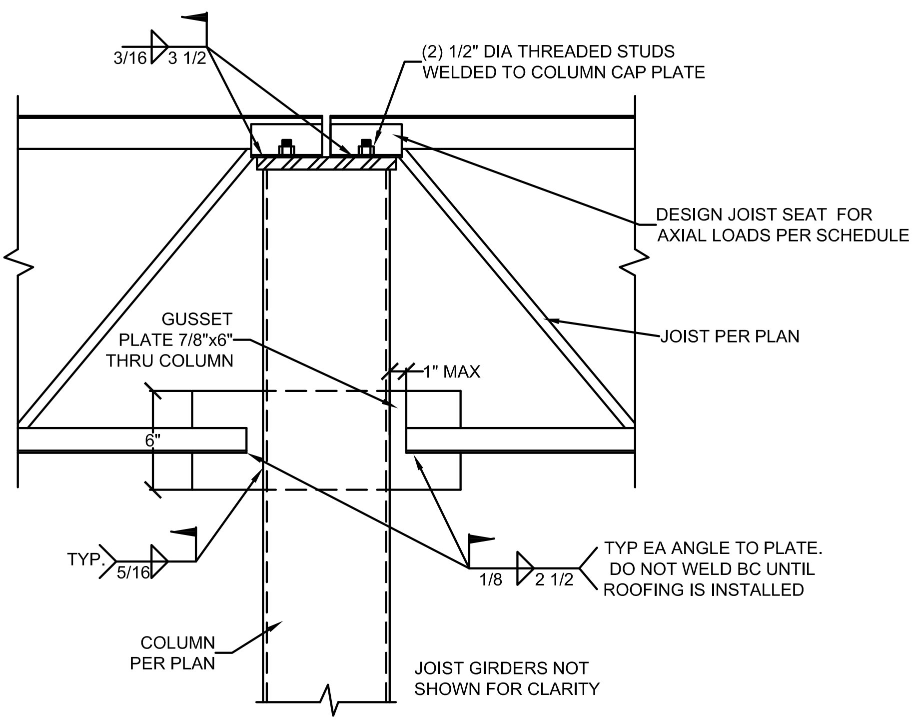

SECTION X-X

**(Perimeter Column Detail)**

**Fig. 4.8.14 Detail F**




**(Interior Column Detail)**




SECTION Y-Y

**Fig. 4.8.15 Detail G**

**Given:**

Frames are Ordinary Truss Moment Frames

Dead load:

Joist: $D$ = 7 psf + 3 psf (Collateral load) = 10 psf

Joist Girder self-weight = 1 psf

Joist Girder $D$ = 10 psf + 1 psf = 11 psf

Live load = 20 psf Reducible

Joist: $L_r$ = 20 psf (Tributary Area < 200 sq.ft.)

Joist Girder: $L_r$ = 12 psf (Tributary Area > 600 sq.ft.)

Typical Joist: 22K4

Joist Girders: 36G8N (loading to be determined)

Eave Height = 18feet (top of joist)

MWFRS loads – LRFD (Simplified Method ASCE 7-16 section 27.4):

110 mph 3 second gust, Exposure C, Risk Category II

Wall wind load, $W$ = 26.2 psf (Windward + Leeward load)

Roof wind load, $W$ = 0 psf (Neglect the uplift load for this example)

Strength Load Combinations:

$1.2D + 1.6L_r + 0.5W$

$1.2D + 0.5L_r + 1.0W$

Wind Drift:

Serviceability Level Loads: $D + 0.6W$, $D + 0.75L_r + 0.75(0.6W)$

Maximum Drift = H/100 for a 10-year wind

10-year wind is 75% of a 50-year wind

Girts at North & South walls brace Perimeter Columns at 7.5 ft on center maximum in weak axis

Perimeter Columns: W14x38 (ASTM A992, $F_y$ = 50 ksi) initial size

Interior Columns: HSS 8x8x1/4 (ASTM A500 Gr. B, $F_y$ = 46 ksi) initial size

**Solution:**

1. Rigid Frame System with Joist Girders at Line 2 through 7

Joist Spacing = 35 ft. / 8 Spaces = 4.375 feet on center

Girder is 36G8N.

*Determine panel point loads:*

Because the Joist Girders will have wind loads and axial loads, provide the panel point dead load and panel point live load separately in a schedule so Vulcraft can design the Joist Girders for the LRFD load combinations.

Panel Point Loads:

Dead Load $D$ = (11 psf)(4.375 ft.)(42 ft.) = 2.02 kips, Use $D$ = 2.1 kips

Live Load $L_r$ = (12 psf)(4.375 ft.)(42 ft.) = 2.21 kips, Use $L_r$ = 2.3 kips

*Determine Joist Girder Approximate Moment of Inertia:*

Effective depth $d$ = 36 in. - ¾ in. - ¾ in. = 34.5 in.

(¾ in. is estimated distance to centroid of top and bottom chord angles)

$I_{girder} = 0.027NPLd$

$= (0.027)(8)(2.1 \text{ kips} + 2.3 \text{ kips})(35 \text{ ft.})(34.5 \text{ in.}) = 1{,}148 \text{ in.}^4$

(Since Joist Girder Moment of Inertia will be used in the frame analysis, the approximate moment of inertia should be specified in the Joist Girder schedule as a minimum required $I$.)

Initial Column Sizes:

W14x38 Perimeter Columns and HSS 8x8x1/4 Interior Columns

| W14x38: | HSS 8x8x1/4: |
|---|---|
| $A$ = 11.2 in.$^2$ | $A$ = 7.10 in$^2$ |
| $I_x$ = 385 in.$^4$ | $I_x$ = 70.7 in.$^4$ |

MWFRS loads:

Wind load will push on wall (either North side or South side). Load the perimeter column with a uniform horizontal wind load.

$w_w$ = (42 ft.)(26.2 psf) = 1,100 plf (LRFD)

*Frame Analysis:*

Provide a rigid connection between the perimeter column and the Joist Girder as shown in the Figure 4.8.16. The connection is to be made after all dead loads are applied. The Joist Girder connection at the interior column will be a pinned connection (Joist Girder bears on column bracket on side of column). Use pinned connections at base of columns.

Use the Direct Analysis Method and Second Order Effects, but not including Pδ.

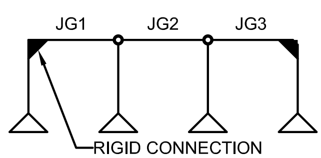



**Fig. 4.8.16 Schematic of Rigid Frame at Line 2 through 7**

*Frame Analysis Results:*

Joist Girder JG1 & JG3:

| End Moments: | Axial Load: |
|---|---|
| $M_{Lr}$ = 30.6 kip-ft. | $W$ = 5.7 kips |
| $M_W = \pm 99.5$ kip-ft. | |

Joist Girder JG2:

Axial Load:

$W = 5.7$ kips

The above moments and axial loads need to be provided to Vulcraft so the Joist Girders can be designed. The axial load will be transferred through Joist Girder seats at interior columns.

Perimeter Column:

$1.2D + 0.5L_r + 1.0W$

$M_{ux1} = 148.5$ kip-ft.

$P_u \quad = 14.1$ kips

$1.2D + 1.6L_r + 0.5W$

$M_{ux1} = 132.4$ kip-ft.

$P_u \quad = 23.8$ kips

Drift (due to 0.6W loading)

$\Delta = 2.09$ in.

10-year wind: $\Delta_{10year} = 0.75\Delta = (0.75)(2.09 \text{ in.}) = 1.57$ in.

Allowable Drift = H/100 = (18 ft.)(12 in./ft.)/100 = 2.16 in. > 1.57 in. **o.k.**

*Perimeter Column Check*: W14x38

Joist Girder bears on T plate on the side of the column (column bracket), so there is an additional moment due to vertical load not at centerline of column (tie force from end moment will be larger than Joist Girder seat can handle).

Column Effective Lengths:

Since the direct analysis with second order effects was used for the analysis, $K_x = 1.0$.

Connection of Joist Girder bottom chord to column will laterally brace the column.

$L_x = 18.0$ ft. - (2.5 in. joist seat depth) - (36 in. Joist Girder Depth) = 14.8 feet

$L_{cx} \quad = K_x L_x = 14.8$ feet.

Column is laterally braced in weak axis by girts at 7.5 feet on center maximum.

Girts are also detailed to brace the column for bending.

From the AISC Manual Table 6-2, use worst case of $L_{cy}$ & $L_{cy\,eq}$:

$$L_{cy\ eq} = \frac{\frac{L_{cx}}{r_x}}{r_y} = \frac{\frac{14.8 \text{ ft}}{5.87 \text{ in.}}}{1.55 \text{ in.}} = 3.91 \text{ ft.}$$

$K_y = 1.0$

$L_y = 7.5$ feet

$L_{cy} = K_y L_y = 7.5$ feet (controls)

$L_b = L_y = 7.5$ ft.

From AISC Manual Table 6-2, $L_c = 8$ feet.

$P_c = \phi_c P_n = 381$ kips

From AISC Specification Section F7

$M_{cx} = \phi_b M_{nx} = 210$ kip-ft.

Use AISC Appendix 8 method to approximate second order effects, $B_1$.

$1.2D + 0.5L_r + 1.0W$

$P_r = P_u = 14.1$ kips

$e$ = (depth/2) + 1 in. gap + 2 in. to centerline of bearing = (14.1 in./2) + 1 in. + 2 in. = 10.1 in.

$M_{ux2} = (P_u)(e)$ = (14.1 kips)(10.1 in.)(1 ft./12 in.) = 11.8 kip-ft.

$B_1 = 1.0$

Conservatively:

$M_{rx} = B_1(M_{ux1} + M_{ux2})$ = 1.0(148.5 kip-ft. + 11.8 kip-ft.) = 160 kip-ft.

$P_r / P_c$ = 14.1 kips / 381 kips = 0.037

Because $P_r / P_c < 0.20$ AISC Eq. H1-1b applies

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0 \qquad \text{AISC Eq. (H1-1b)}$$

$$\frac{14.1\text{ kips}}{2(386\text{ kips})} + \left(\frac{160\text{ kip-ft}}{210\text{ kip-ft}}\right) = 0.78$$

$1.2D + 1.6L_r + 0.5W$

$P_r = P_u = 23.8$ kips

e = 10.1 in.

$M_{ux2} = (P_u)$(e) = (23.8 kips)(10.1 in)(1 ft./12 in.) = 20.0 kip-ft.

$B_1 = 1.01$

$M_{rx} = B_1(M_{ux1} + M_{ux2})$ = 1.01(132.4 kip-ft. + 20.0 kip-ft.) = 154 kip-ft.

$P_r / P_c$ = 23.8 kips / 386 kips = 0.062

Because $P_r / P_c < 0.20$ AISC Eq. H1-1b applies

$$\frac{P_r}{2P_c}+\left(\frac{M_{rx}}{M_{cx}}\right)\leq 1.0$$ AISC Eq. (H1-1b)

$$\frac{23.8 \text{ kips}}{2(386 \text{ kips})}+\left(\frac{154 \text{ kip-ft}}{210 \text{ kip-ft}}\right)=0.76\leq 1.0 \text{ o.k.}$$

**Use W14x38 Perimeter columns**

*Joist Girder to Column Connection Design:*

*Connection at top of Joist Girder:*

The Joist Girder bears on a column bracket, therefore, use a top plate welded to the Joist Girder and to the column cap plate to transfer the force.

$M_u$ = 148.5 kip-ft. (worst case moment for column)

$P_u = M_u/d$

= (148.5 kip-ft.)(12 in./ft.)/(36 in.- 0.75 in.) = 50.6 kips

The distance to the top chord centroid is not deducted because the plate rests on the top chord.

$P_u \leq \phi\, F_y A_g$, $\phi = 0.90$

$A_{req'd}$ = (50.6 kips)/[(0.90)(36 ksi)]

$A_{req'd}$ = 1.56 in.$^2$

**Use a ⅜-in.-thick x 5-in. wide plate.**

$A$ = 1.875 in.$^2$ > $A_{req'd}$ **o.k.**

*Check the plate for compression:*

When $L_c/r \leq 25$ (AISC Specification Section J4.4) the compressive strength of the plate equals the tensile strength. If the fillet welds connecting the plate to the top chord and the column cap plate extend to the edges of the top chord and the cap plate only a small unbraced length of the plate exists. Consider this is the case for the connection. Therefore, use the 3/8-in.-thick x 5-in. wide plate.

*Determine the top plate weld required:*

**Use a ¼ in. fillet weld.**

$\phi R_n$ = (1.392 kip/in.)D AISC Manual Eq.(8-2a)

= (1.392)(4) = 5.57 kips/in.

$L_{req'd}$ = 50.6 kips/5.57 kips/in. = 9.1 in. < 2(5 in.) =10 in. **o.k.**

¼ in. min Joist Girder top chord angles, Block Shear:

$\phi R_n$ = 67.5 kips

Shear Lag does not control for this connection.

**Use Tie Plate ⅜ in. x 5 in. x 0'-10" long with ¼ in. fillet weld 5 in. on both sides of tie plate to Joist Girder, use 10 in. total weld from tie plate to column cap plate. Joist Girder ¼ in. min top chord angles.**

Connection at base of column and overturning check are not part of this example.

*Determine the bottom chord connection to stabilizer plate:*

There is a 1 in. gap between the bottom chord angles, so use ⅞" thick plate to allow for field tolerances (and so weld does not have to be built up).

Try ⅞ in. x 6 in. wide (minimum size for detailing)

$\phi T_n = \phi\, F_y A_g$ = (0.9)(36 ksi)(0.875 in)(6 in) = 170.1 kips > 50.6 kips **o.k.**

*Weld from Joist Girder bottom chord to stabilizer plate:*

Since the Joist Girders are not heavily loaded and the span is not long, the bottom chord angles may not be that large, perhaps ¼ in. or less thick. Since part of the welding will be to the toe of the angle, size weld based on 3/16 in fillet weld.

$L_{req'd}$ = 50.6 kips/[(1.392 kips/in)(3)] = 12.1 in. < 4(4 in.) = 16 in. **o.k.**

*The perimeter column web should also be checked for local web yielding and web crippling per AISC Sect. J10, due to load from Joist Girder Bottom chord* (not shown here).

**Use Stabilizer Plate ⅞ in. x 6 in. wide with 3⁄16 in fillet weld 4 in. long each side of each Joist Girder bottom chord angle to stabilizer plate.**

2. Rigid Frame System with Joists at Line B and C

Typical joist is 22K4, use this as minimum requirement for joists at rigid frame. Because the joists have axial loads and end moments, the uniform loads on the joists will also need to be provided so that Vulcraft can check all the load combinations.

Dead Load: $w_D$ = (4.375 ft.)(10 psf) = 44 plf

Roof Live Load: $w_{Lr}$ = (4.375 ft.)(20 psf) = 88 plf

*Determine approximate moment of inertia for a* 22K4:

From the SJI joist load tables:

$w_{LL}$ = 79 plf for L/360

$I_j = 27.767(w_{LL})(L^3)(10^{-6})$

$L$ = 42 ft. - 0.33 ft. = 41.67 ft.

$I_j = (26.767)(79)(41.67^3)(10^{-6}) = 153$ in.$^4$

Since the Joist Moment of Inertia will be used in the frame analysis, the approximate moment of inertia should be specified in the Joist schedule as minimum required *I*.

Initial Column Sizes: HSS 8x8x1/4 Interior Columns

HSS 8x8x1/4:

$A$ = 7.10 in$^2$

$I_x$ = 70.7 in.$^4$

Columns also support the girders.

MWFRS loads:

Column Lines A, B, C, and D are all frame lines. (Design of frames at Line A and D are not included in this example). Wind load will push on the wall, and the roof diaphragm will distribute the load to the frame lines.

Line B & C Wind Load to frame:

$W$ = (35 ft./2 + 35 ft./2)(18 ft./2)($\pm$26.2 psf) = $\pm$ 8.3 kips (LRFD)

**Frame Analysis:**

Provide a rigid connection between the interior columns and the joists as shown in the Figure 4.8.17. The connection is to be made after all dead loads are applied. The connection of the joist to the shear walls at Line 1 and 8 will be treated as a roller connection. Use pinned connections at base of columns.

Use the Direct Analysis Method and Second Order Effects.

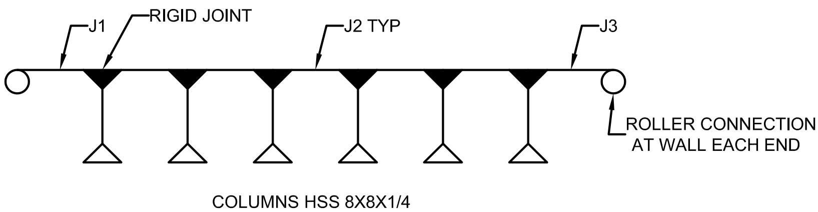



**Fig. 4.8.17 Schematic of Rigid Frame in Joist Direction**

*Frame Analysis Results:*

Joists J1 & J3:

| End Moments: | Axial Load: |
|---|---|
| $M_{Lr}$ = 17.38 kip-ft. | $W$ = 8.3 kips |
| $M_W = \pm 8.65$ kip-ft. | |

Joist J2:

| End Moments: | Axial Load: |
|---|---|
| $M_{Lr}$ = 15.30 kip-ft. | $W$ = 7.0 kips |
| $M_W = \pm 13.84$ kip-ft. | |

The above moments and axial loads must be provided to Vulcraft.

Axial load will be transferred through Joist seats at interior columns.

Interior Column:

$1.2D + 0.5L_r + 1.0W$

$M_{ux}$ = 26.5 kip-ft.

$P_u$ = 28.2 kips

$1.2D + 1.6L_r + 0.5W$

$M_{ux}$ = 16.7 kip-ft.

$P_u$ = 47.6 kips

Drift (due to $0.6W$ loading)

$\Delta$ = 2.29 in.

10-year wind: $\Delta_{10year} = 0.75\Delta$ = (0.75)(2.29 in.) = 1.72 in.

Allowable Drift = H/100 = (18 ft.)(12 in./ft.)/100 = 2.16 in. > 1.72 in. **o.k.**

Interior Column Check: HSS 8x8x1/4

Joist Girders bear on column bracket each side of the column. The unbalanced roof live load case was checked in the initial selection of the column size. Unbalanced roof live load on Joist Girders does not need to be checked with the MWFRS frame loads.

Column Effective Lengths:

Since the direct analysis with second order effects was used for the analysis, $K_x = 1.0$.

Connection of Joist bottom chord to column will brace the column.

$L_x = 18.0$ ft. – (22 in. joist depth) = 16.2 feet

$L_{cx} = K_x L_x = 16.2$ feet.

$L_b = L_x = 16.2$ feet

Column is braced in weak axis at the Joist Girder seat.

$K_y = 1.0$

$L_y = 18.0$ ft. – (2.5 in. joist seat depth) – (7.5 in. Joist Girder seat depth) = 17.2 ft.

$L_{cy} = K_y L_y = 17.2$ ft.

From AISC Manual Table 4-4 $L_c = 18.0$ feet

$P_c = \phi_c P_n = 227$ kips

From AISC Manual Table 3-13:

$M_{cx} = \phi M_p = 70.2$ kip-ft. (AISC Manual Table 3-13)

Use AISC Appendix 8 method to approximate second order effects, $B_1$

$1.2D + 0.5L_r + 1.0W$

$P_r = P_u = 28.2$ kips

$B_1 = 1.06$

$M_{rx} = B_1(M_{ux}) = 1.06(26.5 \text{ kip-ft.}) = 28.1$ kip-ft.

$P_r / P_c = 28.2 \text{ kips} / 227 \text{ kips} = 0.12$

Because $P_r / P_c \leq 0.20$ AISC Eq. H1-1b applies

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0 \qquad \text{AISC Eq. (H1-1b)}$$

$$\frac{28.2 \text{ kips}}{2(227) \text{ kips}} + \left(\frac{28.1 \text{ ft-kips}}{70.2 \text{ ft-kips}}\right) = 0.46 \leq 1.0 \text{ o.k.}$$

$1.2D + 1.6L_r + 0.5W$

$P_r = P_u = 47.6$ kips

$B_1 = 1.10$

$M_{rx} = B_1(M_{ux}) = 1.10(16.7 \text{ kip-ft.}) = 18.4$ kip-ft.

$P_r / P_c = 47.6 \text{ kips} / 227 \text{ kips} = 0.21$

Because $P_r / P_c \geq 0.20$ AISC Eq. H1-1a applies

$$\frac{P_r}{P_c}+\frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right)\leq 1.0 \qquad \text{AISC Eq. (H1-1a)}$$

$$\frac{47.6\text{ kips}}{227\text{ kips}}+\frac{8}{9}\left(\frac{18.4\text{ ft-kips}}{70.2\text{ ft-kips}}\right)=0.44\leq 1.0\text{ o.k.}$$

**Use HSS 8x8x1/4 Interior Columns**

*Joist to Column Connection Design:*

*Connection at top of Joist:*

The connection of the joist is at the seat to column interface. As a result, the effective depth of the joist for determining the tension-compression load for the connections will be based on the distance from the bottom of the joist seat to the centroid of the bottom chords of the joist. For the wind load, the joist seat must transfer the load due to the wind end moment and the wind axial load. Loads to the joist seats need to be specified in the joist schedule so Vulcraft can properly design the joist seats.

Effective depth for connection $d$ = 22 in. – 2.5 in. – 0.5 in = 19 in.

Tension-Compression load from end moment $T = C = M/d$

*Connection Forces:*

Joists J1 & J3:

$T_{Lr}$ = (17.38 kip-ft.)(12 in./ft.)/(19 in.) = 11.0 kips

$T_W$ = (8.65 kip-ft.)(12 in./ft.)/(19 in.) = 5.5 kips

Joist Axial $W$ = 8.3 kips

Wind load to seat, $W$ = 5.5 kips + 8.3 kips = 13.8 kips

$T_u$ = 0.5(11.0 kips) + 1.0(13.8 kips) = 19.3 kips

$T_u$ = 1.6(11.0 kips) + 0.5(13.8 kips) = 24.5 kips (controls)

Joist J2:

$T_{Lr}$ = (15.30 kip-ft.)(12 in./ft.)/(19 in.) = 9.7 kips

$T_W$ = (13.84 kip-ft.)(12 in./ft.)/(19 in.) = 8.8 kips

Joist Axial, $W$ = 7.0 kips

Wind load to seat, $W$ = 8.8 kips + 7.0 kips = 15.8 kips

$T_u$ = 0.5(9.7 kips) + 1.0(15.8 kips) = 20.7 kips

$T_u$ = 1.6(9.7 kips) + 0.5(15.8 kips) = 23.4 kips

Use a 3⁄16 in. fillet weld, 3.5 in. each side of seat.

$\phi R_n$ = (1.392 kip/in.)(3)(3.5 in. + 3.5 in.) = 29.2 kips > 24.5 kips **o.k.**

The joist seat loads noted above significantly exceed the capacity of a Joist Girder seat to resist rollover. This is the reason the joists are bearing on top of the column and the Joist Girders are being supported by a column bracket on the side of the column.

*Connection at Joist Bottom Chord:*

The connection of the joist bottom chord to the column will only need to resist the tension-compression load from the end moments. Since this is a smaller joist and is lightly loaded, assume a minimum bottom chord thickness of 1/8 in., and specify in joist schedule.

*Connection Forces:*

Joists J1 & J3:

$T_u$ = 0.5(11.0 kips) + 1.0(5.5 kips) = 11.0 kips

$T_u$ = 1.6(11.0 kips) + 0.5(5.5 kips) = 20.4 kips (controls)

Joist J2:

$T_u$ = 0.5(9.7 kips) + 1.0(8.8 kips) = 13.7 kips

$T_u$ = 1.6(9.7 kips) + 0.5(8.8 kips) = 19.9 kips

Use a ⅛ in. fillet weld, 2.5 in. each side of each bottom chord angle.

$\phi R_n$ = (1.392 kip/in.)(2)(2.5 in. + 2.5 in.)(2 angles) = 27.8 kips > 20.4 kips **o.k.**

The gap between the bottom chords is going to be 1 in., so use 7/8 in. thick plate to column (o.k. by inspection).

*Connection to HSS column:*

Check the column to make sure the wall of the HSS member will not be overstressed from the load from the joist bottom chord. The load from the joist bottom chord will be considered an "out-of-plane transverse load" to the wall of the HSS column. AISC Equation 9-30 gives the capacity of the side wall for this type of load. AISC Figure 9-5 can be used to determine T, L, a, and b. The ⅞ in. plate will be centered on the column, so a and b will be the same.

$T$ = column width = 8 in.

$a = b$ = ½(column width - 7/8 in. plate) = ½(8 in. - 7/8 in.) = 3.56 in.

$L$ = plate width = 6 in.

$Q_f = 1.0$

$t$ = 0.233 in. (column design thickness)

$$R_n = \frac{t^2 F_y}{2}\left[\frac{(a+b)\left(4\sqrt{\frac{Tab}{a+b}}+L\right)}{ab}\right]Q_f$$

AISC Manual Eq. 9-30

$$R_n = \frac{(0.233\text{ in.})^2(46\text{ ksi})}{2}\left[\frac{(3.56\text{ in.}+3.56\text{ in.})\left(4\sqrt{\frac{8(3.56\text{ in.})(3.56\text{ in.})}{3.56\text{ in.}+3.56\text{ in.}}}+6\right)}{(3.56\text{ in.})(3.56\text{ in.})}\right]1.0 = 14.8\text{ kips}$$

$\phi R_n$ = 1.0(14.8 kips)

This capacity is less than the bottom chord load. Two possible options exist. One is to increase the thickness of the column to see if a thicker wall column would have enough strength. A second option would be to slot the plate thru the column and weld to both walls of the column. This will have a strength of twice that of a connection to a single wall. It also has the benefit that both joists will connect to the same plate and provide better continuity of the loading.

Capacity = 2(f$R_n$) = 2(14.8 kips) = 29.6 kips > 20.4 kips **o.k.**

**Use ⅞ in. x 6 in. plate slotted through column for joist bottom chord connection.**

Connection at base of column and overturning check are not part of this example.

The following are the Joist Girder schedule and schedule for the Joists at the Rigid frames should be provided on the structural plans.

| JOIST GIRDER SCHEDULE (1)(2) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Panel Point Loading | | End Moments (6) | | | | | | |
| | | | | Live Load Continuity Moment $L_r$ (kip-ft) | | Wind Moment W (kip-ft) | | | | |
| Girder Mark Number | Girder Depth & Number Spaces (3) | Roof Dead Load (kips) | Roof Live Load $L_r$ (kips) | Left | Right | Left | Right | Wind Axial Load W (kips) (4)(5) | Min. Moment of Inertia $I_G$ ($in^4$) | Add'l Requirement |
| JG1 | 36G 8N | 2.1 | 2.3 | 30.7 | - | ± 99.5 | - | 5.7 | 1,148 | 1/4" min Top Chord |
| JG2 | 36G 8N | 2.1 | 2.3 | - | - | - | - | 5.7 | 1,148 | |
| JG3 | 36G 8N | 2.1 | 2.3 | - | 30.7 | - | ± 99.5 | 5.7 | 1,148 | 1/4" min Top Chord |

(1) Manufacturer to design Joist Girders using LRFD. Nominal design loads shown are to be used in the applicable LRFD code load combinations.
(2) Deflection Criteria: Live Load Deflection ≤ L/240 .
(3) See framing plan for joist spacing along Joist Girder.
(4) Top chord axial load, Tension or Compression Load.
(5) Manufacturer to design Joist Girder seat to transfer axial loads shown.
(6) End Moment Sign Convention, Positive moments:

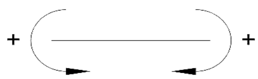



**Table 4.8.3 Joist Girder Schedule**

| JOIST SCHEDULE (1) | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Loads for Combined Bending and Axial Check | | | | | | | | | | |
| | | Uniform Loads | | End Moments (7) | | | | Axial Loads | | | | |
| | | | | Live Load Continuity Moment $L_r$ (kip-ft) | | Wind Moment 1.0W (kip-ft) | | | Seat Axial Load (4) | | | |
| Joist Mark Number | Designation (2) | Dead Load (plf) | Roof Live Load $L_r$ (plf) | Left | Right | Left | Right | Top Chord Wind Load 1.0W (kips) (3) | Live Load $L_r$ (kips) (5) | Wind Load 1.0W (kips) (6) | Min. Moment of Inertia $I_J$ ($in^4$) | Add'l Requirement |
| J1 | 22K4 | 44 | 88 | - | 17.4 | - | ± 8.7 | 8.3 | 11.0 | 13.8 | 153 | 1/8" min Bottom Chord |
| J2 | 22K4 | 44 | 88 | 15.3 | 15.3 | ± 13.9 | ± 13.9 | 7.0 | 9.7 | 15.8 | 153 | 1/8" min Bottom Chord |
| J3 | 22K4 | 44 | 88 | 17.4 | - | ± 8.7 | - | 8.3 | 11.0 | 13.8 | 153 | 1/8" min Bottom Chord |

(1) Manufacturer to design Joists using LRFD. Nominal design loads shown are to be used in the applicable LRFD code load combinations.
(2) Standard designation is minimum requirement. Joist Manufacturer to modify joist design as required for end moments and combined loading requirements.
(3) Axial load is Tension or Compression Load.
(4) Axial load to Joist seat is already included in the Top Chord Axial Loads shown and does not need to be added.
(5) Seat Axial Live Load is due to Continuity End Moment. Live Load Axial = 0 kips at connection to wall.
(6) Seat Axial Wind Load is due to Wind End Moment and Wind Axial load transfer. Wind Axial load 1.0W = 8.3 kips at connection to wall.
(7) End Moment Sign Convention, Positive moments:

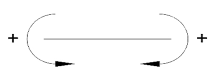
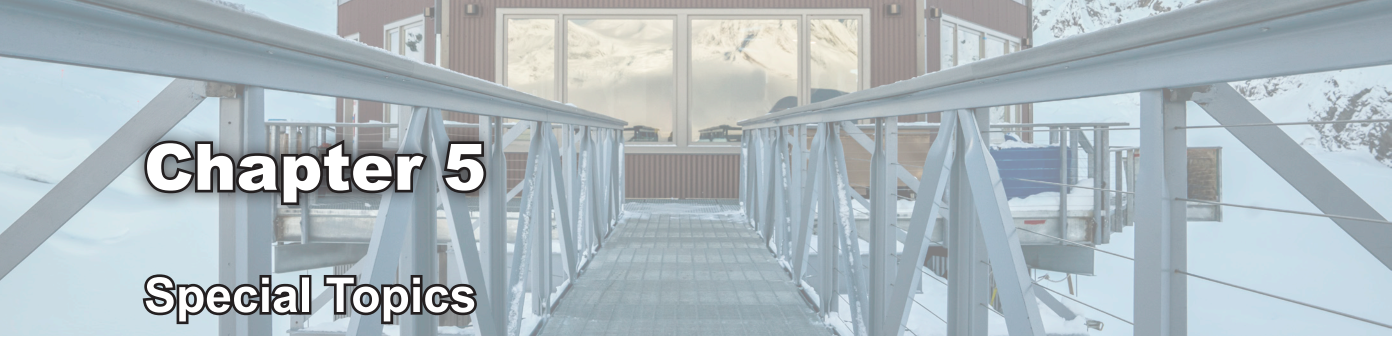



**Table 4.8.4 Frame Joist Schedule**

## SJI Connection Design Tools

Connection design tools can be downloaded from the SJI Website www.steeljoist.org. The tool "Joist Girder Moment Connections to the Strong Axis of Wide Flange Columns" can be used to perform the above connection requirements plus several other checks not shown here.



# Chapter 5

## Special Topics

### 5.1 INTRODUCTION

The information contained in this chapter is presented to make the designer aware of the many considerations that affect the design of joist and Joist Girder systems. These include hanging loads, headers and openings, roof top units, joist reinforcement, spandrel systems, ponding, vibration and fire resistance. In addition, special situations relative to the design and use of joists and Joist Girders are discussed.

### 5.2 HANGING LOADS

#### Cranes and Monorails

Joist systems are often used to support either underhung bridge cranes or monorails. Crane systems are suspended from the joists and impart vertical, lateral and longitudinal forces onto the joist system. The vertical force is equal to the crane beam reaction for the worst-case location of the crane wheels. The lateral forces are due to a combination of many factors such as runway misalignment, trolley movement and/or skewing of the crane bridge. The longitudinal forces are due to the tractive force of the crane accelerating/decelerating or to the crane bumping against the runway stops. Due to the dynamic nature of the forces, the design of crane support systems requires that consideration be given to fatigue and impact.

Since crane loads are not considered in SJI standard joist designations, additional loading and serviceability information must be clearly noted on the contract documents for proper estimating and design. The joist or Joist Girder must be specified as special, "SP," with dead, live, wind or seismic loads and the crane loads clearly noted. Additional design criteria must include the allowable stress range for the joist design, and any special welding requirements. Items that must be noted are as follows:

- Crane classification: see limitations below
- Loading: vertical, horizontal and longitudinal
- Deflection: for the required load cases
- Design Method: follow AISC specifications for fatigue loading
- Inspection: a defined and regular inspection schedule for the joist or Joist Girder should also be specified

These special requirements must be specified by the design professional and coordinated with Vulcraft. It is always best to contact Vulcraft during the specifying process to verify that all requirements can be met.

In underhung crane and monorail support situations, the joists often serve a dual function. They are supporting both a roof (or floor) and the crane system. The specifying professional must take proper precautions to ensure that these functions are compatible. For example, deflections caused by longitudinal and lateral crane thrusts may affect the work of office personnel located adjacent to the crane, or repeated movement from both vertical and lateral loading may have a detrimental effect on the deck to joist attachments.

Crane systems suspended from joists should be limited to the following:

- Cranes that conform to the Crane Manufacturers Association of America, Inc. (CMAA, 2015b) classifications A, B and C

- Crane or monorail capacity of not more than five tons
- Pendant operated cranes only

CMAA crane classifications D, E and F should not be suspended from joists because of their high duty cycles.

It should be noted that the pendant operated limitation excludes radio operated cranes, as these are considered to have the same impact factors as cab operated cranes. It is recommended that crane systems not conforming to the above characteristics have an independent support system. Also, a separate system of structural bracing should be provided in lieu of roof or floor diaphragm bracing to resist the lateral and longitudinal crane thrusts in the plane of the roof for such crane systems.

The crane thrusts provide another load condition to be considered in the design of the frames, but no other special considerations need be addressed.

The crane beam reaction should be increased by the appropriate impact factor for the design of the beam hangers. The impact factor for pendant operated cranes is set by the governing code and is usually 10 percent. The joist supporting the hanger load should also be proportioned to resist impact loading.

The strength of the crane support system is affected by fatigue considerations. The CMAA service classifications (CMAA, 2015a and 2015b) have been established to describe the conditions of use for a crane in a particular situation.

The CMAA Specifications state, "All classes of cranes are affected by the operating conditions, therefore for the purpose of the classifications, it is assumed that the crane will be operating in normal ambient temperature of 0° to 104° F (-17.7° to 40° C) and normal atmospheric conditions (free from excessive dust, moisture and corrosive fumes).

The cranes can be classified into loading groups according to the service conditions of the most severely loaded part of the crane. The individual parts which are clearly separate from the rest, or forming a self-contained structural unit, can be classified into different loading groups if the service conditions are fully known."

The CMAA classifications are as follows:

2.2 CLASS A (STANDBY OR INFREQUENT SERVICE)

This service class covers cranes which may be used in installations such as powerhouses, public utilities, turbine rooms, motor rooms and transformer stations where precise handling of equipment at slow speeds with long, idle period between lifts are required. Capacity loads may be handled for initial installation of equipment and for infrequent maintenance.

2.3 CLASS B (LIGHT SERVICE)

This service covers cranes which may be used in repair shops, light assembly operations, service buildings, light warehousing, etc., where service requirements are light and the speed is slow. Loads may vary from no load to occasional full rated loads with two to five lifts per hour, averaging ten feet per lift.

2.4 CLASS C (MODERATE SERVICE)

This service covers cranes which may be used in machine shops or paper mill machine rooms, etc., where service requirements are moderate. In this type of service, the crane will handle loads which average 50 percent of the rated capacity with 5 to 10 lifts per hour, averaging 15 feet, not over 50 percent of the lift at rated capacity.

2.5 CLASS D (HEAVY SERVICE)

This service covers cranes which may be used in heavy machine shops, foundries, fabricating plants, steel warehouses, container yards, lumber mills, etc. and standard duty bucket and magnet operations where heavy duty production is required. In this type of service, loads approaching 50 percent of the rated capacity will be handled constantly during the working period. High speeds are desirable for this type of service with 10 to 20 lifts per hour averaging 15 feet, not over 65 percent of the lifts at rated capacity.

2.6 CLASS E (SEVERE SERVICE)

This type of service requires a crane capable of handling loads approaching a rated capacity throughout its life. Applications may include magnet/bucket combination cranes for scrap yards, cement mills, lumber mills, fertilizer plants, container handling, etc., with twenty or more lifts per hour at or near the rated capacity.

2.7 CLASS F (CONTINUOUS SEVERE SERVICE)

This type of service requires a crane capable of handling loads approaching rated capacity continuously under severe service conditions throughout its life. Applications may include custom designed specialty cranes essential to performing the critical work tasks affecting the total production facility. These cranes must provide the highest reliability with special attention to ease of maintenance features."

The class of crane, type of crane and loadings all affect the design. The fatigue associated with crane class is especially critical for the design of crane runways and connections of crane runway beams to joist systems.

The CMAA crane classifications do not relate directly to the AISC Specifications for fatigue.

The AISC Specification provides equations to determine an allowable stress range for a given "Stress Category." To use the equations, the designer must enter the value of $n_{SR}$, which is the stress range fluctuations in design life, into the appropriate design equations provided in Appendix 3 of the AISC Specification.

If the specifying professional determines that fatigue considerations must be incorporated into the design of the joists, the specifying professional must select a joist with adequate strength to meet the AISC fatigue provisions. The specifying professional must also inform the joist supplier that the joists are subject to fatigue loading and provide the appropriate allowable stress

Fatigue provisions in the 1989 AISC Specification for Structural Steel Buildings (AISC, 1989), included "Loading Conditions" based on the number of loading cycles. Shown here in Table 5.2.1.

**CRANE LOADING CONDITIONS**

| CMAA Crane Classification | 1989 AISC Loading Condition |
|---|---|
| A, B | 1 |
| C, D | 2 |
| E | 3 |
| F | 4 |

**Table 5.2.1 Crane Loading Conditions**

The approximate number of loading cycles for each loading condition is given in the 1989 AISC Specification Table A-K4.1. The Table is repeated as Table 5.2.2

**AISC LOADING CYCLES**

| Loading Condition | From | To |
|---|---|---|
| 1 | 20,000[a] | 100,000[b] |
| 2 | 100,000 | 500,000[c] |
| 3 | 500,000 | 2,000,000[d] |
| 4 | Over 2,000,000 | |

[a]Approximately equivalent to two applications every day for 25 years.

[b]Approximately equivalent to 10 applications every day for 25 years.

[c]Approximately equivalent to 50 applications every day for 25 years.

[d]Approximately equivalent to 200 applications every day for 25 years.

**Table 5.2.2 AISC Loading Cycles**

The current AISC Specification no longer classifies loading conditions based on the number of cycles as indicated in Table 5.2-2. Regardless, this table provides useful information regarding the number of cycles that can be expected based on different work requirements during a structures service life. The current AISC Specification includes equations that allow the specifying professional to directly calculate the allowable stress range for a given condition based on the number of cycles and the fatigue category of the detail being designed.

For the situation shown in the 2016 AISC Specification, Table A-3.1 Description 8.2, "shear on throat of any fillet weld, continuous or intermittent, longitudinal or transverse" is applicable to fillet welds attaching web members to chords. Calculations for a Class C crane are as follows:

For Description 8.2, the Stress Category is F. Therefore, Equation A-3-2 applies.

$$F_{SR} = 100\left(\frac{1.5}{n_{SR}}\right)^{0.167} \geq 8 \text{ ksi} \qquad \text{AISC (A-3-2)}$$

where

$F_{SR}$ = allowable stress range, ksi

Taking $n_{SR}$ = 500,000 cycles for a Class C

$F_{SR}$ = 12 ksi

Thus, for a Class C crane the fillet welds for web members should be designed for an allowable stress range equal to 12 ksi.

Similar calculations are required to determine the allowable stress range in the base metal adjacent to the welds. The fluctuating stress range in the base metal of the joist chords needs to be evaluated based on the stresses in the chords as well as the forces at the connection.

The AISC Specification also requires, "For axially loaded angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range."

Only the stress range due to the fluctuating load is limited by the AISC fatigue provisions.

The specifying professional should specify a joist depth. Vulcraft will design the joist or Joist Girder for the specified loading and all other specified design criteria.

### Hangers and Bracing

Economical underhung crane runway beams are usually designed to span 15 to 20 feet. Runway beams or monorails may be constructed from standard W or S shapes, or special patented shapes. As discussed above special joists should be designated as special, "SP," joists. The special design criteria as noted above, must be listed. Vulcraft will mark these joists to distinguish them from other joists. This avoids confusion in the field between the special crane support joists and the typical joists.

The crane beam and monorail support hangers must load the joist at a panel point, or concentrated load reinforcement must be provided, or Vulcraft must design the joist chord for the induced bending. The hangers should allow for vertical adjustment. This will allow the crane beams to be leveled after the roofing has been applied and the dead load deflection of the roof system has occurred. The vertical adjustability of the hangers will also accommodate the differences in elevation caused by fabrication and erection tolerances.

The lateral load induced by cranes, varies with the size and type of crane. For the classes of cranes considered, the governing codes usually specify the lateral load to be 20% of the lifted load and the trolley weight. The lateral load is distributed to each runway with due regard to the lateral stiffness of the runway beams and the supporting structure.

Each hanger (only on one side of the runway) should have a lateral brace to prevent the crane beam from swaying at the hanger location. If opposite side hangers are also laterally braced unaccounted for forces will be induced in the system. A typical hanger and brace for this situation is illustrated in Figure 5.2.1.

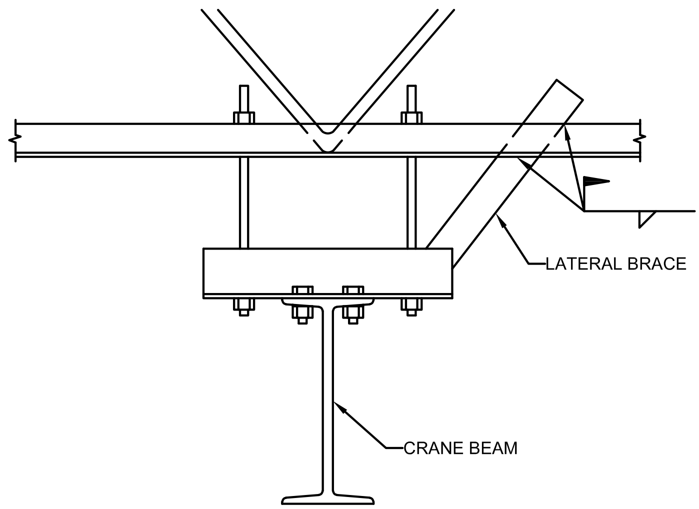



**Fig. 5.2.1 Crane Runway Hanger Preventing Sway**

Care should be taken in the design and detailing of the lateral brace. The brace is intended to resist lateral load. However, the brace may inadvertently pick up some of the vertical load depending on its stiffness relative to the vertical hangers. Since the hangers and the lateral brace are not located precisely at a panel point (as shown in Figure 5.2.1), their loads and locations must be supplied to Vulcraft.

If the crane runway is parallel to the joists, the lateral brace will have to extend to the top chord of an adjacent joist and horizontal members will have to be added directly under the deck to transfer the thrust load into the roof deck. A typical hanger and brace for this situation is illustrated in Figure 5.2.2.

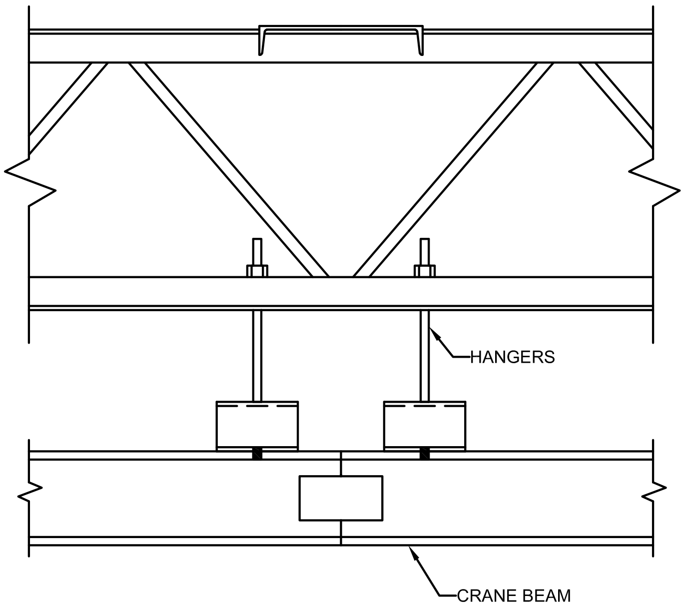



**Fig. 5.2.2 Crane Runway Hanger Parallel to Joist**

The tractive longitudinal force at each runway is typically specified as 10% of the total maximum wheel loads supported by that side of the runway. The longitudinal force created by the crane hitting the crane stops may exceed the tractive longitudinal force. The stopping force is a function of the crane travel speed and the length of stroke of the crane bumper. This bumper force can be controlled by the selection of the bumper. The resulting load to the support system should be coordinated between the specifying professional and the crane supplier.

A bracing system is required to resist the longitudinal crane thrusts. If the crane runway runs parallel to the joists, the longitudinal thrusts are transferred through the joist diagonals to the top chord and into the roof deck or into a bracing system designed by the specifying professional. All joist forces must be specified. Additional deck connections may be required and must be noted. The typical hanger detail will require modification to also transfer the longitudinal load.

Clamp type hangers may be used to attach hangers to the bottom chord of joists. However, the specifying professional must design or specify clamps to avoid bending the outstanding legs of the joist chord. Clamps and hangers are not part of the components designed and supplied by Vulcraft.

If the crane runway is perpendicular to the joists, longitudinal braces must be provided. The braces should be provided at intervals of about every fourth of fifth crane beam (about one hundred feet apart). Monorails also require longitudinal bracing. If the monorail turns a corner, bracing should be provided in both rail directions at each corner. Figure 5.2.3a illustrates the configuration of a lateral brace for a crane system running parallel to the joists and Figure 5.2.3b illustrates the configuration of a longitudinal brace for a crane system running perpendicular to the joists. The number of transfer channels and puddle welds required is a function of the brace force and the strength of the deck. Criteria for the design of welds to light gage steel deck is contained in the AISI Specification for the North American Specification for the Design of Cold-Formed Steel Structural Members.

For the best load path lateral braces should be attached to the transfer channel rather than the joist top or bottom chords.

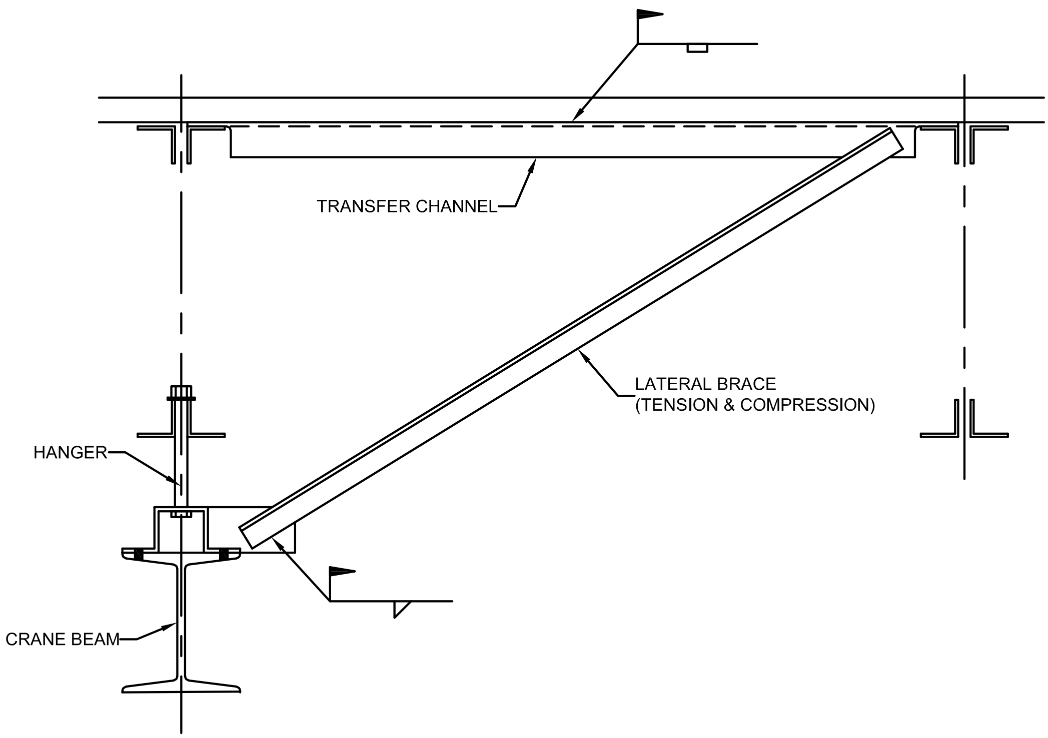



**Fig. 5.2.3a Lateral Crane Brace**

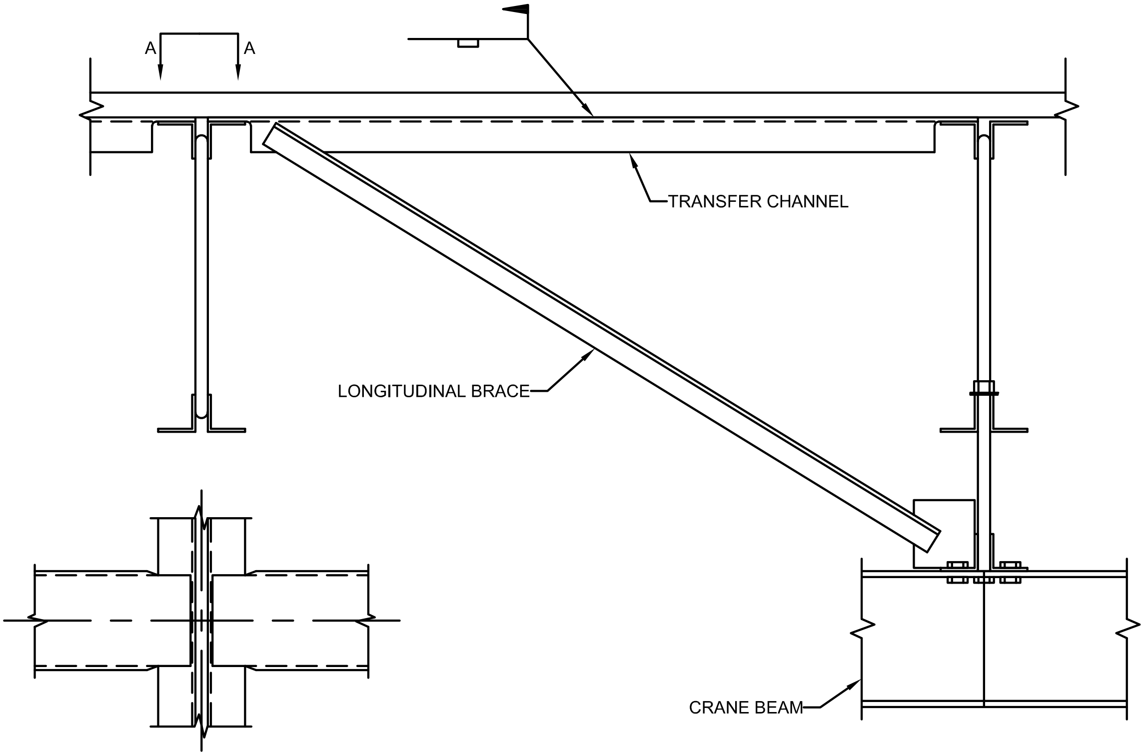



**Fig. 5.2.3b Longitudinal Crane Brace**

The proper specification of joists for monorail loading (including impact) is similar to the specification of joists for any other concentrated loading. This is discussed in detail in Chapter 6. Specification of a joist to support an underhung crane is more complex than supporting a single load. Depending on the trolley location, either the left or right hanger load may be larger. Given the shifting shear and moment diaphragms created by the possible crane loading conditions, the use of KCS joists should be considered for this situation. In Example 5.2.1, a KCS joist is selected to support a one-ton underhung crane.

### Example 5.2.1 KCS Joist/Crane Support

Choose a KCS joist to support a one-ton pendant operated underhung crane in addition to the tributary roof load. The crane runs perpendicular to the joist span.

**Given:**

Joist span = 40 feet

Joist spacing = 5 feet

Dead load = 20 psf

Live load = 30 psf

Crane bridge length = 20 feet

Wheel load = 2.5 kips/wheel

Wheel spacing = 6'-0" (2 wheel/end truck)

Crane bridge weight = 2.8 kips

Trolley weight = 1.0 kips

Crane standby use only

CMAA Class A (Less than 20,000 cycles)

Crane beam weight = 30 plf

Runway hangers at 10 feet. from each joist end

**Solution: (ASD)**

Roof load on the joist = (20 psf+30 psf)(5 ft) = 250 plf = 0.25 k/ft.

Determine the maximum hanger reaction: $R_{max}$ = (2.5 kips)[1.0+(14.0 ft)/(20.0 ft)] = 4.25 kips.

Increase for impact and beam load:

Impact Factor = 10 percent of wheel load

$R_{max}$ = (4.25 kips)(1.1)+(0.03 kips/ft)(20 ft) = 5.3 kips

Determine the wheel loads at the far side of the crane aisle from the maximum wheel loads. The minimum wheel load equals the total weight of the lifted load, bridge and trolley, minus the maximum wheel loads divided by 2.

$R_{min}$ = (2.0 kips+2.8 kips+1.0 kips-2.5 kips-2.5 kips)/2

= 0.40 kips.

Determine the minimum hanger reaction:

$R_{min}$ = (0.40 kips)[1+(14 ft)/(20 ft)] = 0.68 kips

Increase for impact plus beam load.

$R_{min}$ = (0.68 kips)(1.1)+(0.03 kips/ft)(20 ft) = 1.3 kips

The average reaction:

$R_{ave}$ = (5.3 kips+1.3 kips)/2 = 3.3 kips.

Construct the load diagrams (Figures. 5.2.4 and 5.2.5) and solve for maximum shear and moment.

Based on the maximum hanger reaction:

$R_L$ = (0.25 kips/ft)(40 ft/2)+(1.3 kips)(30 ft/40 ft)+(5.3 kips)(10 ft/40 ft)

= 7.3 kips

$R_R$ = (0.25 kips/ft)(40/2)+(5.3 kips)(30 ft/40 ft)+(1.3 kips)(10 ft/40 ft)

= 9.3 kip

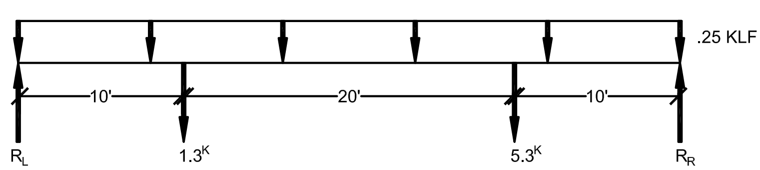



**Fig. 5.2.4 Load Diagram**

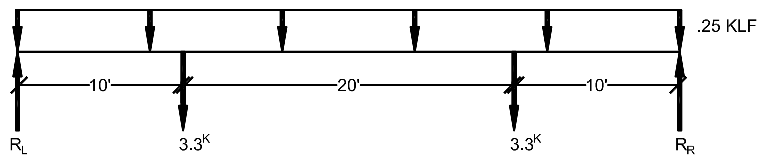



**Fig. 5.2.5 Load Diagram**

From statics, the point of zero shear is located 24 feet from the left support.

$M_{max}$= (7.3 kips)(24 ft)-(1.3 kips)(14 ft) (0.25 kips/ft)(24 ft)$^2$/2

$M_{max}$= 85 kip-ft

= 1020 kip-in.

Based on the average hanger reaction:

$R_L = R_R$ = (0.25 kips/ft )(40 ft/2) + 3.3 kips = 8.3 kips

$M_{max}$ = (8.3)(20 ft)-(3.3 kips)(10 ft)-(0.25 kips/ft)(20 ft)$^2$/2

$M_{max}$ = 83 kip-ft

= 996 kip-in.

The condition with the maximum wheel load on the left side is identical but opposite hand to the case with the maximum wheel load on the right side. The same maximum moments and shears result. The load cases excluding roof live load would have greater shifts in the shear diagram. However, the KCS joist specifications require that the joist diagonals be designed for 100% shear reversal (except for the end diagonal), and a constant moment diagram. Therefore, the location of maximum moment is not a concern.

Select a 26KCS5

Shear strength = 9,200 lbs.

Moment strength = 1,576 kip-in.

The KCS joist load tables can be used to select a joist with a enough moment of inertia to limit the joist deflection to an acceptable level. The deflection should be based on the live load and crane load resisted by the joist as the crane support will deflect this total amount. The crane's ability to travel should not be impaired. The acceptable amount of vertical deflection at the runway support should be determined after consultation with the crane supplier about the crane support requirements. Generally, a one percent grade is acceptable.

In lieu of specifying a KCS joist, the specifying professional can specify the required loads and joist depth for Vulcraft to design the joist.

### Beam Supports

In some cases, it may be undesirable or impractical to support the underhung crane or monorail from joists. In this case, beams could be provided to support the crane loading and to span between the Joist Girders. Depending upon the camber in the joist the designer may decide to specify a similar amount of beam camber. The beams could be designed with an end seat to match the joists in order to bear on the Joist Girder. If the loads are such that a seated connection is not possible then the Joist Girder could be specified to have a vertical web member at the beam support location in order to attach the beam to the Joist Girder. Figure 5.2.6 illustrates a beam to girder web connection. The angles are fabricated with standard holes. The beam should have slotted holes and snug tight bolts. The bolt gage and pitch must be coordinated between the joist supplier and the steel fabricator.

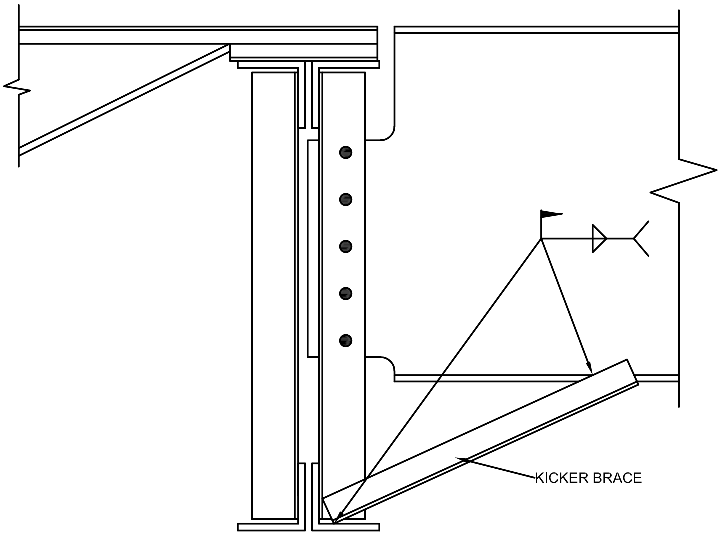



**Fig. 5.2.6 Beam to Girder Connection**

The beam to girder web connection should be designed to deliver the beam reaction to the center of the Joist Girder in order to minimize the amount of beam end rotation induced into the girder. If practical, the end seat connection should be used in place of framing the beam into the web of the girder. The end seat connection requires less material and no special erection considerations. The beam end can be reinforced to act as a shallow seat if the unreinforced web section of the member can transfer the shear load. If the strength is exceeded, a heavier beam should be used, or the beam will have to be connected to the web of the Joist Girder. End seat reinforcement is usually required to resist the bending in the coped section of the beam. The design of a 2.5-inch end seat is illustrated in the following example.

### Example 5.2.2 Beam Seat Design

*Design a 2.5 in. deep seat for the given beam and loading.*

**Given:**

W16x31

Reaction (ASD), $R$ = 9.0 kips

The bearing plate = ½ in.

¼ in. fillet welds are used to attach the bearing plate to the beam web.

Use $F_y$ = 50 ksi for the bearing plate

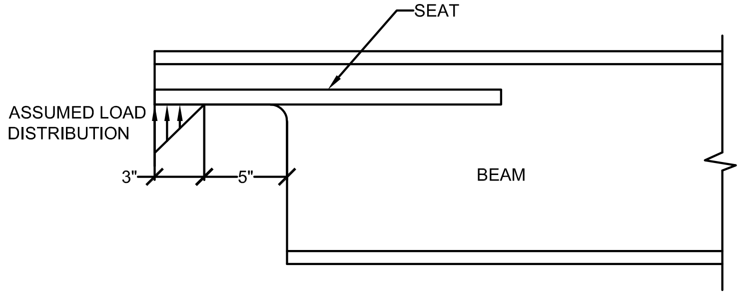



**Fig. 5.2.7 Example 5.2.2**

**Solution:**

Check the bending strength of the cantilever:

*Check the shear strength of 2-½" deep section:*

The nominal shear strength, $V_n = 0.6F_yA_wC_{v1}$
AISC Eq. (G2-1)

$A_w$ = area of web, the overall depth times the web thickness, $dt_w$, in.$^2$

= (2.5 in.)(0.275 in.) = 0.68 in.$^2$

$h$ = clear distance between flanges less the fillet at each flange, in.

where

$k$ = 0.832 in.

= 2.5 in.-(0.832 in.-0.5 in.- 0.25 in.) = 0.92 in.

$t_w$ = thickness of web, in.

$C_{v1}$ = 1.0

$V_n$ = 0.6(50 ksi)(0.68 in.$^2$)(1.0) = 20.4 kips AISC Eq. (G2-1)

$\phi_v$ = 1.0, $\Omega_v$ = 1.5

$\phi_vV_n$ = 1.0(20.4 kips) = 20.4 kips

$V_n/\Omega_v$ =20.4 kips/1.5 = 13.6 kips

For ASD:

9.0 kips < 13.6 kips **o.k.**

*Reinforce the section:*

Try adding a 4 in. wide plate to the T section. (See Figure 5.2.8.)

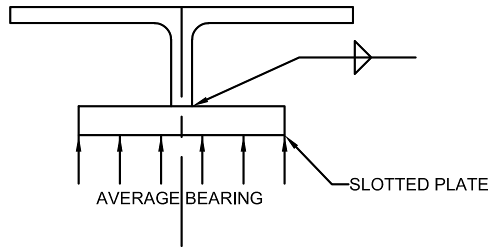



**Fig. 5.2.8 Added Plate**

*Check the plate thickness:*

The average bearing stress = $R$/[(Plate width)(Bearing length)]

= (9.0 kips)/[(4 in.)(3.0 in)]= 0.75 ksi

The length of the cantilever equals the clear distance from the edge of the beam fillet weld to the edge of the plate. Estimate this dimension as 1.70 inches.

The required moment, $M_r = wL^2/2$
= (0.75 ksi.)(1.70 in.)$^2$/2
= 1.08 kip-in.

The nominal moment $M_n = M_p = Z_xF_y$
$Z_x = bt^2/4$ = (1.0 in.)(0.5 in.)$^2$/4 = 0.0625 in.$^3$
$M_n$ = (0.0625 in.$^3$)(36 ksi) = 2.25 kip-in.
$M_n/\Omega$ = (2.25 kip-in.)/1.67 = 1.35 kip-in.
1.35 kip-in. > 1.08 kip-in. **o.k.**

The ½ in. plate is adequate.

*Check the cantilever bending strength of the composite section:*

Based on the triangular stress distribution shown in Figure 5.2.7 the reaction is located one inch from the end of the seat, L = 7 inches.

$M_r$= (9.0 kips)(7 in.) = 63 kip-in.

The section properties for the composite cantilever section are:

$A$ = 4.93 in.$^2$

$Z_x \approx$ 4.67 in.$^3$

$M_n = F_yZ_x$

=(50 ksi)(4.67 in.$^3$)

=234 kip-in.

$M_n/\Omega$ = 140 kip-in.

140 kip-in. > 63 kip-in. **o.k.**

*Determine the weld required to connect the plate to the beam web.*

The weld must resist the shear flow ($v$), where; $v = VQ/I$

$V$ = Shear at the critical section

$I_x$ = Moment of inertia of the composite section

$Q$ = The first moment of area of the added material

$I_x$ = 4.68 in.$^4$

$y_{bar}$ = 1.36 in.

$Q$ = (1.36 in.-0.25 in.)(0.5 in.)(4.0 in.)

= 2.22 in.$^3$

$v$ = (9.0 kips)(2.22 kips)/4.68 in.$^4$

= 4.3 kips/in.

Try a 3⁄16 fillet weld near and far side.

Weld nominal strength:

$R_n/\Omega$ = (2)(0.928)(3) = 5.57 kips/in.
AISC Manual Eq. (8-2b)

5.57 kips/in. > 4.3 kips/in. **o.k.**

*Evaluate the weld required to anchor the plate:*

$P$ = $MQ/I$

= (63 kip-in.)(2.22 in.$^3$)/4.68 in.$^4$

=29.9 kips

Length of 3/16 fillet weld required:

Allowable weld force per in. = 5.57 kips/in.

Length required = 29.9 kips/5.57 kips/in. = 5.4 in.

Extend the plate 6" beyond the cope and weld with 3⁄16 fillet weld ns/fs. See Figure 5.2.9 for final configuration.

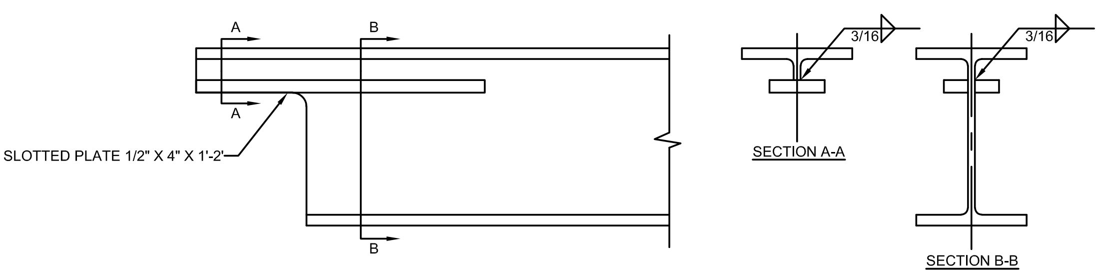



**Fig. 5.2.9 Beam Seat**

## Conveyors

The proper design of joist systems for the suspension of conveyor equipment is analogous to the design of joist systems supporting cranes and many of the same basic considerations apply. The joists must provide support that is sufficiently rigid so that the function of the conveyor is not impeded. Also, the performance of the roof or floor that is also supported by these joists should not be compromised. The key to successfully supporting a conveyor from a joist system is careful coordination between the design professional, the conveyor supplier and Vulcraft. The systems provided by the various conveyor manufacturers are often proprietary and the support requirements vary from project to project.

There are many different types of conveyors. Most conveyors may be grouped under three major headings:

1. Continuous Belt Conveyors
2. Trolley Conveyors
3. Vibratory Conveyors

A continuous belt conveyor consists of a flexible belt that is supported at regular intervals by idlers. The belt returns below, supported by return idlers, forming a continuous loop. The idlers are supported on a frame that requires support from the joists at regular intervals, usually twenty to thirty feet. The specifying professional should obtain the support reactions from the conveyor supplier. The support reactions should include the loaded weight of the conveyor and any service walkways that may be present. A belt conveyor is usually driven by a motor drive at the end of the conveyor. Tensioning of the belt may be provided by a gravity take up device. This is a weight that is suspended from the returning belt to provide the appropriate tension to the belt. The belt drive and take up loads should be located and accounted for in the design. This type of conveyor is usually a self-contained stable unit (with the belt tension resisted by compression in the conveyor frame) requiring only vertical support and nominally sized lateral and longitudinal braces.

A trolley conveyor is a chain driven conveyor in which the chain is supported from a monorail at regular intervals. The chain usually forms a continuous horizontal loop. This type of conveyor is most commonly used for assembly line work. The product is suspended from the chain at regular intervals and is moved along the track from station to station. This type of conveyor may be self-contained regarding thrusts; that is, equal and opposite thrusts may be resolved through the conveyor framing. If the conveyor is not self-contained, the conveyor will impart horizontal loads into the system. The magnitude of these loads is dependent upon the drive system used, the type of support system used at the conveyor level, and the configuration of the overall conveyor system. The longitudinal force in trolley conveyors is commonly referred to as the chain pull force. The vertical loads and thrusts must be determined from the conveyor supplier for each situation. Even if thrust loads are not specified, it is recommended that at least a nominally sized lateral and longitudinal bracing system be provided. If the operation of the conveyor does impart thrusts onto the joist system, the specifying professional should consider the use of a structural bracing system independent of the roof diaphragm.

A vibratory conveyor or shaker table moves the product by shaking the material on to a conveyor. This type of conveyor creates considerable dynamic energy and is subject to high cycle fatigue loadings. It is not recommended that this type of conveyor be suspended from joists or Joist Girders.

Details for the suspension of conveyor systems are like the details required for underhung cranes. The details presented in the preceding section on crane support could be modified to suspend a conveyor system.

Conveyors may be floor mounted as well as suspended from the structure above. If the floor is framed with joists, the special loads induced by the conveyors should be considered. Conveyors or roller tables running perpendicular to the joists may impart significant concentrated loads into the joists. If a shaker table is mounted on an elevated floor system, isolators should be provided, and the natural frequency of the floor joists should be at least fifty percent higher than the frequency of the shaker table. Information regarding the calculation of the natural frequency of floor joist systems can be found in Section 5.6.

## Sprinkler and Other Hanging Piping

The support of sprinkler systems requires that the specifying professional accommodate the load from the sprinkler systems and provide for the hanger attachment for the sprinkler systems. The support of process piping, small ducts and cable trays requires similar considerations. The suppliers for these products should be consulted about support requirements and loads. The support of sprinkler systems is rather a generic problem and standards are available to aid the specifying professional in designing support for these systems.

It must be noted if the sprinkler pipes pass through the webs of the joists. Vulcraft will align the web panels in a bay to accommodate the specified spacing. The pipe spacing must be uniform since the joist web configuration is typically uniform. The typical maximum branch line spacing is 10 feet, thus when possible a 10-foot spacing should be used.

The sprinkler mains are larger pipes and the loads must be noted on the contract documents as they are considerably heavier that the branch lines.

Load capacity for sprinkler systems is usually provided by the specifying professional using a uniform collateral load of enough magnitude to account for the loads induced by the piping system. This collateral load is added to the other loadings, and a joist of enough strength is specified to resist the uniform loads. Although the loads are delivered to the joists at discrete locations, this method is used almost universally and has proven to be reasonable and economical. Cable trays and duct systems are treated in the same fashion. Large ducts for heavy industrial duct collection systems and unusually large sprinkler pipes should be considered a special case, and the section discussing the consideration of concentrated loads should be consulted.

The National Fire Protection Association provides guidelines for the support of sprinkler systems in their publication NFPA 13 Standard for the Installation of Sprinkler Systems (NFPA, 2019). For steel pipe (except threaded light wall pipe), the standard requires hangers at a maximum of 12 feet on center for pipes 1.25 inches in diameter and smaller, and hangers at a maximum of 15 feet on center for pipes 1.5 inches in diameter and larger. The maximum spacing between hangers for other types of pipe can be found in the NFPA document. Additional hangers may be required at branch and bend locations or between branch locations. These criteria apply to piping conforming to NFPA requirements. Table 5.2.3 lists typical weights and hanger reactions for sprinkler pipes.

### SPRINKLER WEIGHTS

| | Pipe & Water (pounds/ft.) | | Hanger Load 5 ft spacing (pounds) | | Hanger Load 12 ft. spacing (pounds) | |
|---|---|---|---|---|---|---|
| Pipe Dia. (inches) | Schedule 10 | Schedule 40 | Schedule 10 | Schedule 40 | Schedule 10 | Schedule 40 |
| 2 | 4.3 | 5 | 22 | 26 | 52 | 61 |
| 3 | 8 | 11 | 40 | 54 | 96 | 130 |
| 4 | 12 | 16 | 60 | 82 | 144 | 196 |
| 5 | 18 | 23 | 90 | 117 | 216 | 280 |
| 6 | 24 | 32 | 120 | 158 | 288 | 378 |
| 8 | 41 | 50 | 205 | 251 | 492 | 603 |
| 10 | 58 | 75 | 290 | 373 | 696 | 895 |
| 12 | - | 99 | - | 493 | - | 1184 |

**Table 5.2.3 Typical Sprinkler System Weights**

Hanger locations are not typically specified on the structural drawings. However, the specifying professional could require hanger spacings at shorter intervals than required by NFPA requirements to reduce individual hanger reactions to the roof system. The specifying professional should provide details to ensure that the pipe hanger loads are located at joist panel points and do not induce bending in the joist chords. Alternately, a concentrated load can be specified to occur at any location on the joist. The contractor installing the piping should be familiar with the NFPA requirements regarding hanger locations and the permissible types of hangers.

The specifying professional and Vulcraft should also be aware of the requirements for "Early Suppression Fast Response Fire Sprinkler Systems" (ESFR). These systems are primarily used in warehouses where high piled storage is used. ESFR systems are designed to 'suppress a fire' meaning they are designed to "knock" the fire down to its point of origin. NFPA 13 does not allow obstructions of any size in the area 1 foot to each side and 2 feet below a pendent ESFR sprinkler. Contact with small obstructions can cause significant disruption to the discharge pattern and will cause failure of the system to function properly.

As indicated earlier, the branch lines pass through the webs of the joists. The spacing of the lines must be coordinated with Vulcraft. Typically, the interior webbing panel of joists are the same length and 10 foot equally spaced branch lines can be easily accommodated. Unequally spaced branch lines can be accommodated but will significantly increase complexity and cost of the joists.

Sprinkler systems in areas subject to earthquakes require sway bracing to resist lateral, longitudinal and vertical movement resulting from seismic loading. Sway bracing must be designed and specified by the design professional and must be anchored at locations that provide a reasonable load path to the lateral load system of the structure.

It should be noted that building codes may have criteria more stringent than the NFPA criteria. Also, FM Global (FM Global, 2018) or other insurance criteria should be consulted, if appropriate.

### Mechanically Field Installed Struts at Concentrated Loads

Bolted or screwed connections can be used to attach reinforcement members to joists or Joist Girders. The use of bolts or screws are best suited for conditions when loads and/or reinforcement members are relatively small and when welding is prohibited. Caution must be used when drilling holes into members since holes can significantly reduce the tensile strength of the member. Also, care must be taken not to drill holes into welds, thus reducing existing weld strength.

*Installation:*

Bolt installation and hole sizes used should follow the provisions of the AISC Specifications. 3/8 in. ASTM A354 Grade BC bolts may be used. These bolts are included as Group A high strength bolts in Section J3 of the AISC Specification.

For screwed connections the AISI Specifications should be followed. Strengths and types of screws can be found in the manufacturer's literature.

Based on Chapter J of the AISC Specification the allowable shear strength of a ⅜" diameter Group A fastener is 3.0 kips.

The reinforcing member should be predrilled using a 7⁄16 in. diameter bit.

### Example 5.2.3 Mechanically Field Installed Struts at Concentrated Loads

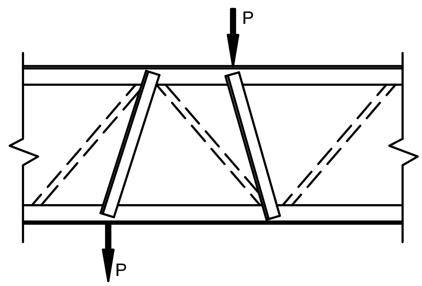
Connect angles to the top and bottom chords of a joist to support two 500-pound point loads as shown in Figure 5.2.10. Use a ⅜ in. diameter A354 Grade BC bolts in lieu of welding.

As a reminder the SJI Specification does not require struts between panel points provided the sum of the concentrated loads within a chord panel does not exceed 100 pounds and the attachments are concentric to the chord. The 100-pound loads must also be accounted for in the specified uniform design loads.

**Given:**

The bending and shear strength of the joist has been determined to be sufficient to support the added 500-pound loads; however, the combined bending and axial forces between the panel points overstress the joist.

Field measurements revealed that the bottom chord angles are 1.5x1.5x0.155 in. and the top chord angles are 2x2x0.155 in.




Use A36 steel for struts.

**Fig. 5.2.10 Concentrated Loads Between Panel Points**

Try 2x2x1/8 A36 angle struts.

The struts are at an angle of 75 degrees from the horizontal.

The strut length = 33 in.

Strut Properties:

A36 steel

$F_u$ = 58 ksi

$A$ = 0.491 in.$^2$

$r_x$ = 0.620 in.

**Solution:**

*Compression Strut:*

Required axial compression per strut:

$P_r = P_{strut}$ / (cos15°) = 250 lbs / 0.966

$P_r$ = 259 lbs.

The struts meet the requirements per AISC Section E5(a). Therefore, the available axial compressive strength may be determined per either equation E5-1 or E5-2 as applicable:

From AISC Section E5, if $b/t \le 0.71\sqrt{E/F_y}$ flexural-torsional buckling need not be considered.

$b/t \le 0.71\sqrt{29{,}000 \text{ ksi}/36 \text{ ksi}} = 20$

$b/t$ = 2 in. / 0.125 in. = 16

16 < 20 therefore flexural torsional buckling need not be considered.

$L/r_x$ = 33 in./0.620 in. = 53

When $\frac{L}{r_a} \leq 80$

$$\frac{L_c}{r} = 72 + 0.75\frac{L}{r_a} \qquad \text{AISC Eq. (E5-1)}$$

$$= 72 + 0.75\left(\frac{33 \text{ in.}}{0.620 \text{ in.}}\right) = 112$$

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} = \frac{\pi^2 (29{,}000 \text{ ksi})}{(112)^2} = 22.8 \text{ ksi} \qquad \text{AISC Eq. (E3-4)}$$

$$\frac{F_y}{F_e} = \frac{36 \text{ ksi}}{22.8 \text{ ksi}} = 1.58 \leq 2.25$$

$$\text{Therefore, } F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y = \left(0.658^{\frac{36 \text{ ksi}}{22.8 \text{ ksi}}}\right)36 = 18.6 \text{ ksi}$$

The available stress = $F_{cr}/\Omega$ = 18.6 ksi/1.67 = 11.1 ksi

The available strength = ($F_{cr}/\Omega)A_g$ = 11.1 ksi)(0.491 in.2) = 5.45 kips

5.45 kips > 0.259 kips Angles are **o.k.**

*Tension Strut:*

The available axial tensile strength is the lesser value of the limit states of yielding and rupture.

For tensile yielding in the gross section:

$P_n = F_y A_g$ AISC Eq. (D2-1)

$P_n$ = (36 ksi)(0.491 in.$^2$) = 17.7 ksi

$P_n / \Omega$ =17.7 /1.67 = 10.6 kips

For tensile rupture:

The AISC Commentary (AISC, 2016d) indicates, "There is insufficient data for establishing a value of U if all lines have only one bolt, but it is probably conservative to use $A_e$ equal to the net area of the connected element. The limit states of block shear (Section J4.3) and bearing and tear out (Section J3.10, which must be checked, will probably control the design.

$A_e$ = (2.0 in.)(0.125 in.) - (0.5 in.)(0.125 in.) = 0.1875 in.$^2$

$P_n = F_u A_e$ = (58 ksi)(0.1875) = 10.875 kips

$P_n/\Omega$ = 10.875/2.0 = 5.44 kips

Bearing check:

$R_n = 3.0dtF_u$ AISC Eq. J3-6b

where

$d$ = nominal fastener diameter, in. = 0.375 in.

$t$ = thickness of connecting material, in.

$R_n$ = 3.0(0.375 in.)(0.125 in.)(58 ksi) =8.16 kips

$R_n/\Omega$ = 4.10 kips

Tearout check:

$R_n = 1.5 l_c t F_u$ AISC Eq. J3-6d

where

$l_c$ = clear distance, in direction of the force, between the edge of the hole and the edge of the material, in.

Assume the distance from the center of the hole to the angle edge = 0.75 in.

$l_c$ = 0.75 in.- 0.4375 in./2 = 0.5313 in.

$R_n$ = 1.5(0.5313 in.)(0.125 in.)(58 ksi) = 5.78 kips

$R_n/\Omega$ = 6.12 kips/2.0 = 2.89 kips

Block shear check:

$R_n = 0.60 F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.60 F_y A_{gv} + U_{bs} F_u A_{nt}$ AISC Eq.(J4-5)

By observation will not control.

**Summary:**

Limit state of 3/8 in. diameter bolt shear = 3.0 kips

Compression strut buckling = 5.45 kips

Limit state of angle rupture per $R_n/\Omega = F_u A_e$ = 5.44 kips

Limit state of bolt bearing = 4.10 kips

Limit state of bolt tear out = 2.89 kips (0.5313 in. clear distance bolt hole to angle end

*Check the strength reduction in the tension chord due to the bolt hole:*

Hole size = 7/16 in. = 0.4375 in.

Chord angle area = 0.441 in.$^2$

$A_e$ = 0.441 in.$^2$ - (0.4375 in.+.0625 in.)(0.155 in.) = 0.364 in.$^2$

Percent reduction in chord strength = (0.441 in.$^2$ - 0.364 in.$^2$)/0.441 in.$^2$ = 17.5%

The designer must check bottom chord for adequacy due to the reduction.

**Self-drilling screw solution:**

In lieu of using a bolted connections screws can be used. Section J4 of the AISI Specifications can be used for screwed connections:

- J4.2 Minimum Edge Distance

- J4.3.1 Shear Strength [Resistance] Limited by Tilting and Bearing
- J4.3.2 Shear in Screws
- J4.4.2 Pull-Over Strength [Resistance]

**Given:**

Try a ¼ in. self-drilling fastener.

Screw head diameter = 0.50 in.

**Solution:**

*Minimum Edge Distance:* AISI Eq. (J4.2)

The distance from the center of a fastener to the edge or end of any part shall not be less than 1.5*d*. For a 0.25 in. diameter screw, the minimum edge distance = 1.5(0.25 in.) = 0.375 in.

*Shear Strength [Resistance] Limited by Tilting and Bearing:* AISI Eq. (J4.3.1)

For $t_2/t_1 \leq 1.0$, $P_{nv}$ shall be taken as the smallest of

$$P_{nv} = 4.2\left(t_2^3 d\right)^{1/2} F_{u2} \quad \text{AISI Eq. (J4.3.1-1)}$$

$$P_{nv} = t_1 d F_{u1} \quad \text{AISI Eq. (J4.3.1-2)}$$

$$P_{nv} = t_2 d F_{u2} \quad \text{AISI Eq. (J4.3.1-3)}$$

where

$P_{nv}$ = Nominal shear strength of sheet per screw

$t_1$ = Thickness of the member in contact with screw head or washer (0.125 in.)

$t_2$ = Thickness of the member not in contact with screw head or washer (0.155 in.)

$d$ = Nominal screw diameter (0.25 in.)

$F_{u1}$ = 58 ksi (Reinforcing angles)

$F_{u2}$ = 65 ksi (Joist chords, $F_y$ = 50 ksi)

For $t_2/t_1 \geq 2.5$, $P_{nv}$ shall be taken as the smaller of

$$P_{nv} = 2.7 t_1 d F_{u1} \quad \text{AISI Eq. (J4.3.1-4)}$$

$$P_{nv} = t_2 d F_{u2} \quad \text{AISI Eq. (J4.3.1-5)}$$

For $1.0 < t_2/t_1 < 2.5$, $P_{nv}$ shall be calculated by linear interpolation between the above two cases.

$t_2/t_1$ = 0.155 in./0.125 in. = 1.24

Interpolation is required.

$t_2 / t_1 \leq 1.0$

$$P_{nv} = 4.2\left[(0.155 \text{ in.})^3 (0.25 \text{ in.})\right]^{1/2} (65 \text{ ksi}) = 8.33 \text{ kips} \qquad \text{AISI Eq. (J4.3.1-1)}$$

$$P_{nv} = (0.125 \text{ in.})(0.25 \text{ in.})(58 \text{ ksi}) = 1.81 \text{ kips} \qquad \text{AISI Eq. (J4.3.1-2)}$$

$$P_{nv} = (0.155 \text{ in.})(0.25 \text{ in.})(65 \text{ ksi}) = 2.52 \text{ kips} \qquad \text{AISI Eq. (J4.3.1-3)}$$

*For* $t_2 / t_1 \geq 2.5$

$$P_{nv} = 2.7(0.125 \text{ in.})(0.25 \text{ in.})(58 \text{ ksi}) = 4.89 \text{ kips} \qquad \text{AISI Eq. (J4.3.1-4)}$$

$$P_{nv} = (0.155 \text{ in.})(0.25 \text{ in.})(50 \text{ ksi}) = 1.94 \text{ kips} \qquad \text{AISI Eq. (J4.3.1-5)}$$

Interpolating for the 0.125 in. material:

$$P_{nv} = 1.81 \text{ kips} + \frac{0.24}{1.5(4.89 \text{ kips-}1.81 \text{ kips})} = 1.86 \text{ kips}$$

Interpolating for the 0.155 in. material:

$$P_{nv} = 2.52 \text{ kips - } \frac{0.24}{1.5(2.52 \text{ kips-}1.94 \text{ kips})} = 2.25 \text{ kips}$$

$$\frac{P_{nv}}{\Omega} = \frac{1.86 \text{ kips}}{3.0} = 0.62 \text{ kips}$$

*Screw shear strength:* AISI Eq. (J4.3.2)

$P_{available}$ = 0.81 kips (manufactures literature for a ¼ in. self-drilling fastener)

*Pull-Over Strength [Resistance]:* AISI Eq. (J4.4.2)

$$P_{nov} = 1.5 t_1 d'_w F_{u1} = 1.5(0.125 \text{ in.})(0.50 \text{ in.})(58 \text{ ksi}) \;=\; 5.44 \text{ kips}$$

$$d'_w = d_h = 0.50 \text{ in.}$$

$$P_{available} = P_{nv} / \Omega = 5.44 \text{ kips} / 3.0 = 1.81 \text{ kips}$$

**The limit state of tilting and bearing controls.**

$P_{available}$ = (0.62 kips)(1000 lbs/kip) = 600 lbs

620 lbs > 259 lbs **o.k.**

Check the strength reduction in the tension chord due to the screw hole:

Hole size = 0.25 in.

Chord angle area = 0.441 in.$^2$

$A_e$ = 0.441 in.$^2$ - (0.25)(0.155 in.) = 0.40 in.$^2$

Percent reduction in chord strength = (0.441 in.$^2$ - 0.400 in.$^2$)/0.441 in.$^2$ = 2.5%

The designer must check bottom chord for adequacy due to the reduction.

## 5.3 HEADERS AND OPENINGS

In this section, the effects of small openings in the roof or floor deck are considered. This discussion treats openings and headers for deck support. In the case of very small openings (6" to 12") deck reinforcement can be used in lieu of headers. The maximum size of opening considered, is an opening that can fit between two joists without disrupting the specified joist spacing for a given framing situation. Openings often coincide with additional concentrated loads, such as at roof top units. This situation will be treated in Section 5.4. This discussion is limited to the framing around openings and the effect on joists when the overall load to the joist is essentially unchanged from the typical situation for surrounding joists.

Small openings are often required in roofs for items such as access hatches, vents or small domed skylights. The manufacturers of these items typically provide the products mounted on their own light gage metal curbs. Usually, these curbs can be set atop the steel roof deck and may be screwed directly to the deck. The deck opening is cut to match the inside dimensions of the curb. Headers or a small frame should be provided to carry the curb loading to the joists. Wood or steel blocking is often placed between the deck flutes to prevent the deck from crushing between the curb and the headers. The typical configuration can be seen in Figure 5.3.1.

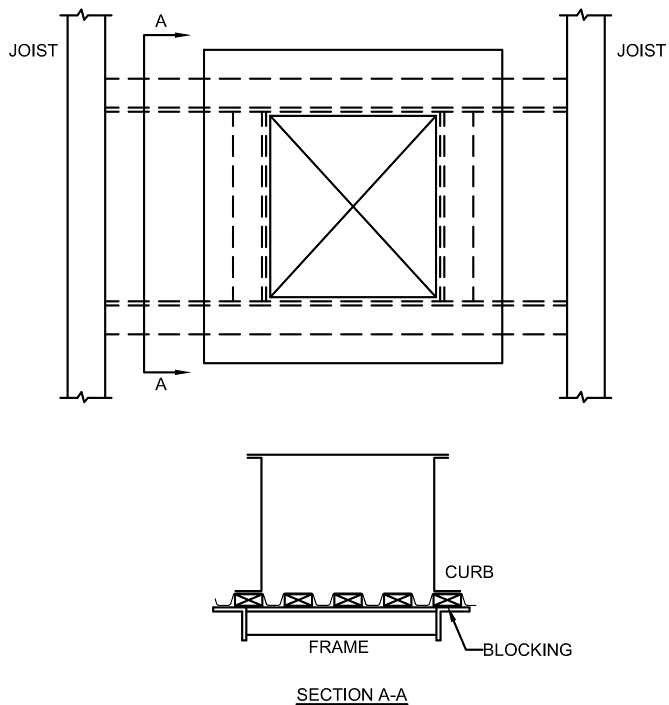



**Fig. 5.3.1 Typical Roof Opening**

The frames are usually constructed from hot rolled angles with welded connections. The vertical leg of the header angle is coped, or a short piece of angle is welded to the end of the header to create a seat.

This latter method is more economical than coping the angle and is also a better detail.

The headers should be installed prior to placing the roof deck, so the header seat can bear on top of the joist chord. If the frame is not installed prior to placing the roof deck, then the frame must be welded to the bottom of the outstanding horizontal leg of the joist chord. This is not as desirable as setting the frame on top of the joist, because it requires an overhead fillet weld for installation. This attachment may also cause twisting of the joist chord.

Headers at openings impose concentrated loads on joists. These concentrated loads can occur either on or off panel points. If located at a panel point the effect thereof is limited to the design for shear and moment on the joist. If the load is located between panel points, then top chord bending is introduced. The discussion of the provision for concentrated loads on joists is presented in Section 6.3.

It is always desirable to locate concentrated loads at panel points and thus eliminate top chord bending. Small isolated openings for vents can usually be shifted to align with panel points. This, however, requires that the opening frame is made to conform with the panel point spacing. For repetitive openings with a consistent pattern, special joists designed for the uniform and concentrated loads can be used.

In these special joists, if the loads cannot be placed on the panel points, Vulcraft must increase the chord size or add web members as illustrated in Figure 6.3.4. In general, it has been found that additional web members are less costly than increasing the chord size. Lastly, in order to ensure that loads are located at the panel points, a special double frame such as that shown in Figure 5.3.2 can be used. This frame requires that prior to its fabrication, the dimensional relation between panel point location and the opening dimensions be known. Alternately, a double frame such as the one shown in Figure 5.3.3 can be used. If the angles which rest on the joist chords are designed to span between joist panel points, then the double frame can be positioned at any location on the joists without overloading the joist chord.

It may be required to interrupt a line of joist bridging at the opening location. This is acceptable if the bridging is properly anchored. Additional comments regarding bridging are included in the Section 5.10.

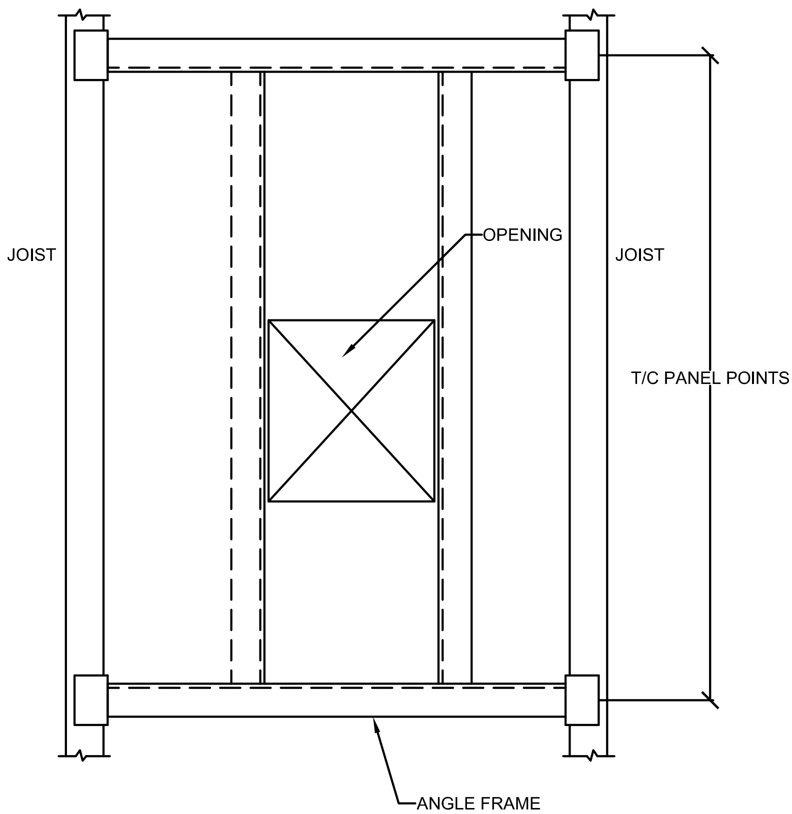



**Fig. 5.3.2 Double Frame**

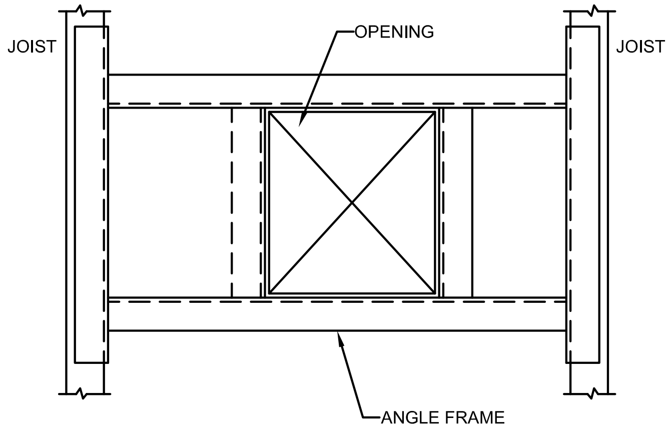



**Fig. 5.3.3 Double Frame**

## 5.4 ROOF TOP UNITS

It is common practice for mechanical units to be placed on the roofs of buildings. These units may be part of the buildings heating and ventilating system, or the units may be a portion of the building's process equipment. Most are supported on a cold formed metal curb. The curb bears on the deck or on structural members that fit between the deck flutes and span between the joists. Inside the curb, there are openings for ductwork and piping. As an alternative to the curb support, units may be supported on a steel framework above the level of the roof deck. The elevated frame may support several units and a walkway. This frame is typically supported on small pipe or tube columns attached to the joists. This system is most commonly used for equipment that does not require large penetrations through the roof. The elevated walkway provides the advantage of eliminating wear and tear on the roof during servicing of the equipment.

Roof top units may vary in weight from a hundred pounds to in excess of twenty thousand pounds. Sizes vary from two or three-square feet to hundreds of square feet in area. Given the large variation in unit size and weight, the specifying professional must be particularly concerned with the load imposed by the unit on specific joists. Roof top units seldom have a uniform density with the center of gravity at the center of the unit. Frequently the units have a large plenum at one end that weighs very little and most of the weight is concentrated over a small area. The unit supplier can provide the specifying professional with reactions at the corners of the unit. Alternatively, the supplier may provide the location of the center of gravity and the weight of the unit. The load and unit configuration vary considerably from one supplier to another, and the specifying professional should take care to obtain information specific to the project at hand and not extrapolate from previous projects. The weight of the curb or support frame is often not included in the weight information provided by the unit supplier. The curb weight should be added to the unit weight. Also, it should be confirmed that the unit provided for is the unit which is ordered, shipped and installed.

The specifying professional should be aware that it is common for there to be substitutions in the final selections of roof top units during the bidding and construction phases of projects. The structural design provisions for roof top units must be based on the unit weights, sizes and layout provided to the specifying professional during design. The structural drawings should show this information, as it is the basis of the design. This will facilitate confirmation of the appropriateness of any proposed substitutions that differ from the information provided at the time of design.

The specifying professional's decision about how to best provide strength for the roof top units will depend on the size, number and similarity of the units. The specifying professional may provide the required strength for large roof top units by specification of a special joist that can support the specific unit's reaction. The section on the design and specification of joists subjected to concentrated loads should be consulted for more detailed information on this topic. If a significant number of relatively large units are randomly dispersed on a roof the specifying professional may prefer to use KCS joists in lieu of specifying individual special joists. This may prove most effective if all the special joists are just slightly different in loading. Replacing many similar special joists with KCS joists will avoid confusion, minimize the potential for errors, and maximize the flexibility of the system. KCS joists are not designed for the bending of the concentrated loads between panel points, so if the loads are not at panel points, additional web members must be installed.

A similar option would be to use a Load/Load designated joist with an Add-Load that will cover the variable mechanical loads in that area of the roof. The Load/Load designation has the advantage that the joist can also be designed for axial loads. KCS joists and Load/Load joists are not designed for the bending of the concentrated loads between panel points, so if the loads are not at panel points, additional web members must be installed.

If a project is being fast tracked or if the specifying professional is unable to procure definitive unit load and placement information, the specifying professional may choose to resort to the zoning method to provide strength to support roof top units. In the zoning method, the specifying professional in consultation with the mechanical engineer, designates selected zones on the roof where units may be placed. Using the mechanical engineers' estimates of the number, size and weights of the anticipated mechanical units, the size, location and loading of the zones are designated on the plans. Joists are selected or specified to resist the loads placed within the zones. This is a good approach for estimating a project. If more accurate information is available Vulcraft can design for that information.

The zones should be located to provide the maximum area, while affecting the fewest number of joists and Joist Girders possible. Locating the zones near to columns will minimize the amount of flexural resistance required in the system. Zones should be located at relative high points of the roof to avoid conflicts with roof drainage. Placing a rectangular zone with at least two of the four boundary lines coincident with building frames lines will help avoid confusion and will result in zones located near columns.

For the zone approach to be successful, the joists specified within or partially within a roof zone must be able to support the units placed within the zone. This will require that any joists in the zone be able to resist a given reaction at any point along their chords (with panel point field reinforcement, if required) within the zone. Figure 5.4.1 is an illustration of the proper specification of a zone with appropriately sized KCS joist.

The use of zones requires a relatively conservative design, and the size of zones should be minimized. Roof top unit zones are particularly useful when specifying a prototype building. Savings in design time are realized if the same zones are used for each building built according to the prototype plans. The owner of the prototype buildings has the advantage of a uniform design for each building.

There is a third alternative to providing special joists at each unit or providing zones for the units. Roofs with numerous small units (reaction to a joist < 500 lbs.) may be designed to support these units at any location on the roof. The specifying professional would determine the worst-case loading of unit reaction to a joist, and use the procedure outlined in Chapter 6 to choose a joist to resist the unit load. These joists would be used throughout the roof. This procedure essentially provides a uniform collateral strength throughout the roof. The specifying professional using this procedure will quickly be able to determine if joist size selection has been appreciably affected. If the cost of providing the uniform strength throughout the roof is overly expensive, the alternatives of zoning the units or providing special joists at each unit should be investigated. When Joist Girders are used to support KCS joists the specifying professional must clearly indicate the panel point loads and the joist spacing for the girders.

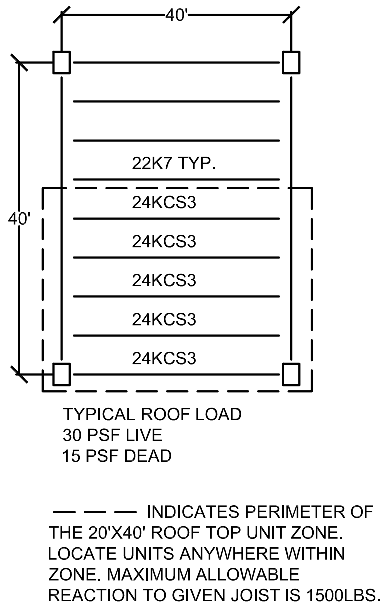



**Fig. 5.4.1 Roof Top Unit Zone**

To support mechanical loads on a joist system, the specifying professional must fill the gap between Vulcraft and the mechanical engineer by determining the load imposed on each joist and designating the required joist for each situation. The structural design drawings should indicate the location of large roof supported units. Zone locations or the loads for small randomly placed units should be indicated on the plans. The plans should also indicate the sizes of the joists. Load diagrams should be provided for special joists. The unit size and location information and the zoning information on the plans will allow the other consultants and trades to coordinate with the specifying professional. Joist designations and load diagrams will allow Vulcraft to coordinate with the specifying professional. Vulcraft should include the joist designations and load diagrams on the shop drawings for confirmation during the shop drawing review.

Providing properly sized joists and Joist Girders will ensure adequate shear and flexural strength to transfer the loads from the joists and Joist Girders to their supports once the load has been transferred into the joists. The unit reaction must be transferred into a panel point of the joist to avoid localized bending of the top chord of the joist. If the location of the unit can be controlled, the specifying professional may be able to locate the edges of the unit at a joist panel point. The locations of the joist panel points will have to be determined by Vulcraft. If this is not possible, a diagonal member will usually have to be added to transfer the reaction to a joist panel point. This detail is illustrated in Figure 5.4.2. This diagonal is usually added in the field. However, Vulcraft will add the diagonal in the shop if instructed to do so, and if the exact location, size and weld requirements of the diagonal are specified. The provision for concentrated loads is presented in Section 6.3.

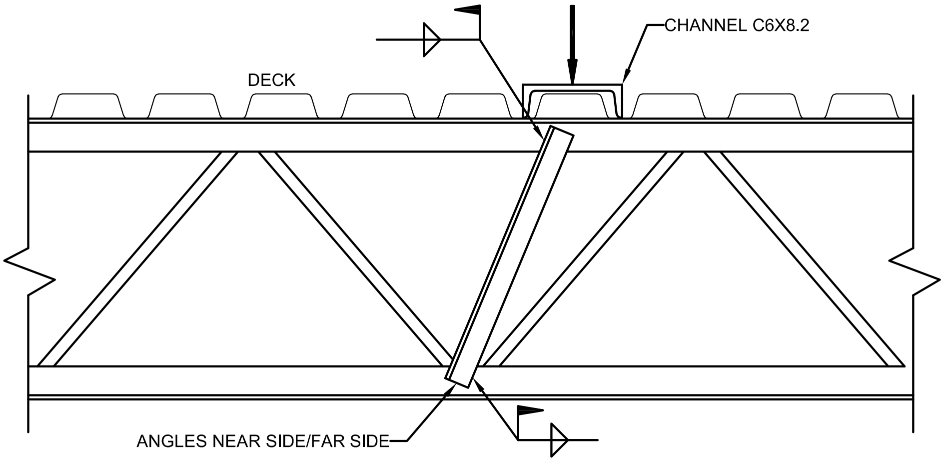



**Fig. 5.4.2 Joist Reinforcement**

The use of an inverted channel to carry the unit load between the joists is also illustrated in Figure 5.4.2.

The specifying professional should be aware that placing mechanical units on the roof may create other special considerations. In addition to their own weight, roof top units may cause other loads to occur such as snow accumulation around the unit. The 2015 IBC (by referencing ASCE 7) has the following exception for requiring drift loads at projections, "EXCEPTION: Drift loads shall not be required where the side of the roof projection is less than 15 feet (4.6 m) or the clear distance between the height of the balanced snow load, $h_b$, and the bottom of the projection (including horizontal supports) is at least 2 feet (0.61 m)."

The effect of wind blowing against the profile of the unit must also be considered. The overturning force reaction may be greater than the reaction due to the weight of the unit, or there may even be a net uplift at some support locations. Wind controlled reactions are more likely with high-profile-low-density units with large plenums.

Roof top units frequently contain moving parts such as fans or motors. It is possible that the operation of these units will cause the support structure to vibrate. Vibrating units should be mounted on isolators to separate their motion from the structure. However, the isolators may not be completely effective with large cyclone fans or compressors. The problem may be mitigated by providing support joists with a natural frequency at least fifty percent greater or fifty percent less than the operating frequency of the unit.

The lower frequency, however, will allow the support structure to vibrate during start up and shut down of the unit when the operating frequency passes through the natural frequency of the joist. The natural frequency of a simple span joist may be determined from Equation 5.4.1 or Equation 5.4.2 as applicable. Equation 5.4.1 defines the natural frequency of a joist loaded primarily by a concentrated load at mid-span. Equation 5.4.2 defines the natural frequency of a joist loaded primarily by a uniform distributed load.

$$f = 188/\sqrt{\Delta} \qquad \text{(Eq. 5.4.1)}$$

$$f = 213/\sqrt{\Delta} \qquad \text{(Eq. 5.4.2)}$$

$f$ = the natural frequency of the joist in cycles per minute

$\Delta$ = the joist deflection at mid-span in inches

Verification of the natural frequency of the joists is not a substitute for providing isolators.

Design of such isolators is the responsibility of the mechanical engineer and the equipment supplier. When checking a system for possible dynamic excitation, the specifying professional should consider the loads likely to be supported by the joist during the operation of the unit. Joists adjacent to the unit may also be susceptible to vibration.

Tall vents or stacks protruding through the roofs of buildings often require guy wires for stability. Parapets and mechanical screen posts may require bracing. An attachment to the building structure should be provided to avoid tearing the deck or roofing. It is usually desirable to attach the guy wires or braces to a vertical standard several inches above the level of the roof to avoid interference with the roofing materials. Two possible connection details are illustrated in Figure 5.4.3 and 5.4.4. All the force vectors and eccentricities of the guy wire attachment must be resolved into the support structure and the joists sized accordingly.

In Figure 5.4.3 the upper horizontal angle must be attached to the steel deck in order to transfer any forces transverse to the joists into the steel deck. Joists cannot resist any out of plane loading. For the best load path lateral braces should be attached to the transfer channel rather than the joist top or bottom chords.

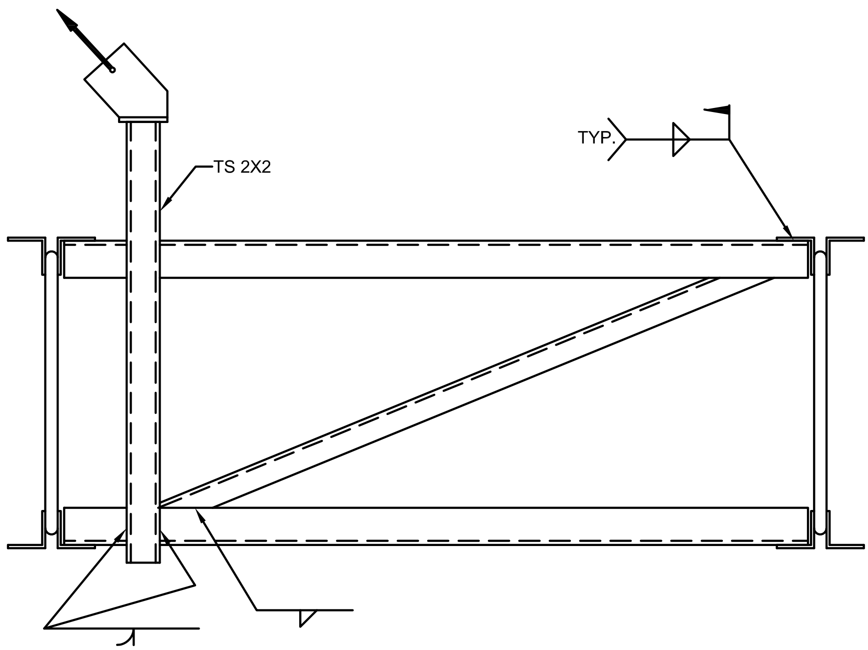



**Fig. 5.4.3 Guy Wire or Screen Post Attachment**

## 5.5 JOIST REINFORCEMENT

### Introduction

The situation often arises when new loads are introduced to an existing framing system, and the system must be evaluated with respect to the new loading. If the existing system is unable to safely support the additional loads, then the system must be reinforced. The new loading must be evaluated against the known strength of the joist elements to resist moments, shears and end reactions. If reinforcement is required, it must be designed for the joists. If the joists in place were furnished by Vulcraft, and if time permits, the specifying professional can ask for assistance from Vulcraft for the reinforcement design. Vulcraft may require a fee for their work.

One can also ask the SJI office for assistance in determining the joist type. Before contacting the SJI office it is recommended that the "Joist Investigation Form" be downloaded from the SJI Website, www.steeljoist.org.

There are several acceptable ways to evaluate existing joists for a new loading condition. Engineering judgement and experience is valuable in determining the most efficient method for each individual condition. The first step is to determine what the existing joist is and what it was designed for. Although joists are generally designed to a stress ratio of 0.95 to 1.0, sometimes the original loads were conservative. This is a good place to start the evaluation. The goal is to do as little reinforcing as possible.

The strength of the joists can be determined from load tables and the use of the standard SJI Specifications. The SJI Specifications require that for a given series of joists the web members be designed for a minimum percentage of the joist end reaction. The specifications also have requirements for the minimum strength of vertical web members, chord splices and the welding of diagonal members. The material properties and the proper interaction equations for checking the chord members may also be determined from the SJI Specifications. The Steel Joist Institute has published the "90 Year Open Web Steel Joist Construction Digest" (SJI, 2018) that is particularly useful for determining the strength of older joists. The 90-year digest contains specifications and load tables for all the series of joists published between 1928 and 2018. The digest also contains helpful chronological listings of changes in the joist design methods.

When evaluating an existing system, the size and series of joists must be determined. The best method is to refer to the construction documents for the joist size and spacing designated, and then to verify the existing construction with a field visit. Inspection of the actual installation is important. It allows the determination of the configuration and the sizes of the chords and webs. In addition, a check is made to see whether:

1. The web members are rods or crimped single angles or double angles.
2. The chords are hot rolled angles or rods or a cold formed shape.
3. The bridging is in place.
4. The joists are in good repair or have been damaged.

This information is important since the reinforcement must not only work with regard to stresses and deflections, but it must also be physically compatible with the existing construction. The reinforcement in the shape of bars, rods or angles must fit. Eccentricities must be eliminated or evaluated, and the field welder must have room to weld the pieces in position. Therefore, oversizing reinforcing members is not always a conservative design. The project site visit also allows the evaluation of the present actual loading condition on the joists.

If the specifying professional does not have access to the design drawings, then the site visit is, of course, even more essential. It may be possible to determine the joist designation from the joist tag. Each joist is supplied with a tag at one end. This tag is intended to mark that specific joist for erection purposes. Vulcraft will provide an erection plan that indicates where each joist is to be located. The joists are identified on the plan by the mark on the tag. Besides the mark number, the tag may also indicate the joist manufacturer. If the manufacturer can be identified they can be contacted to determine if they have any records of the structure. Even if the manufacturer does not have records of the project, the manufacturer may have helpful data about his previously supplied joists. If the tags have been removed, then the joist configuration, chords and diagonals may be measured. This of course does not provide the material properties of the joists. Rather than measure each element of the joist, only the chords and the end diagonal can be measured. With the capacities of these members, the specifying professional could determine the allowable end shear and moment of the joist. Then by estimating the original design loads and the time of construction, a reasonable and conservative estimate of the joist size can be made. As an alternative to measuring the joists, it may be reasonable to assume that the strength of the existing joists is being fully utilized to support the existing loads and add sufficient reinforcement to carry the entire effect of the additional loading.

## Load Distribution

The simplest way to distribute load is to span a member between two joists. The member is designed as a simple beam to span between the joists.

If distributing the load between two joists results in a joist overstress, a support system that will distribute the load between several joists can be designed. By distributing the load to several joists, the load to each individual joist may be small enough to eliminate the need for joist reinforcement. This system is analogous to a continuous beam on flexible supports. The support system must be sufficiently rigid with respect to the joists to distribute the load to the intended joists.

The engineer can create an analytical model to determine the load distributed to each joist by a continuous beam. The joists may be modeled as beam members using the appropriate moment of inertia for the joists.

Criteria applicable to beams on closely spaced elastic supports is applicable to this case. An in-depth discussion of this material may be found in Seely, F.B., and Smith, J.O., “Advanced Mechanics of Materials,” (Seely, 1962). One case of the beam on elastic supports is of interest. This is the case where the beam, which distributes the load to the joists, is stiff enough to be considered as a rigid body with respect to the supporting joists.

In lieu of using an analytical computer model the user may want to create a spreadsheet using this procedure.

The relative stiffnesses of the joists and the distribution beam is defined by the characteristic parameter, beta, as defined in Equation 5.5.1.

$$\beta = \sqrt[4]{(K/S)/4EI} \qquad \text{(Eq.5.5.1)}$$

where:

$K$ = the stiffness of the joist, kips/in.

$S$ = the spacing of the joists, in.

$E$ = the modulus of elasticity for the beam, ksi

$I$ = the moment of inertia of the beam, in.$^4$

If S is less than $\pi/4\beta$ the beam on elastic support calculations are applicable. If the spacing limit is not exceeded and the length of the beam is less than $1/\beta$, the beam may be rigid with respect to the supporting joists and the reactions to the joists may be determined by static equilibrium. In lieu of using a spreader beam below the joist, a specially designed and field fabricated truss can be placed between joists to distribute load between several joists. The chords of the truss can be attached below the joist top chord and above the bottom chord. The web members of the truss can be placed between adjacent joists and attached to the truss chords. If the truss is not located at a joist panel point, joist web reinforcement may be required.

The following example illustrates the design of a beam to distribute a concentrated load to four joists.

**Example 5.5.1 Load Distribution to Joists**

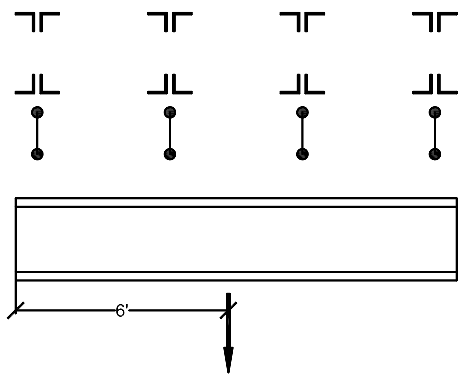



**Fig. 5.5.1 Example 5.5.1**

Size the beam shown in Figure 5.5.1 to act rigidly and determine the reactions to the joists.

**Given:**

The load is located at mid-span on the joists.

**Solution:**

Determine the stiffness of the joists:

$I_J = 26.767(W_{LL})(L^3)(10^{-6})$

For a 24K7:

$W_{LL} = 148$ plf.

$I_J = 26.767$ (148 plf)(39.67 ft)$^3$($10^{-6}$)= 247 in.$^4$

Divide $I_J$ by 1.15 to account for shear deflection.

$I_{J\,eff}$ = (247 in.$^4$)/1.15 = 215 in$^4$

$K = P/\Delta = P/(PL^3/48EI) = 48EI/L^3$

$K$ = [(48)(29,000 ksi)(215 in.$^4$)/[(39.67 ft)(12 in./ft)]$^3$ = 2.78 kips/in.

Based on Equation 5.5.1, determine the beam size necessary to distribute the load to the four joists:

Try a W16x26:

$I_x = 301$in.$^4$

$$\beta = \sqrt[4]{\frac{(2.78\text{ kips/in.})/(54\text{ in.})}{(4)(29{,}000\text{ ksi})(301\text{ in.}^4)}} = 0.0062/\text{in.}$$

Check if the spacing $S < \pi/4\beta$

$S$ = 54 in. $< \pi/4\beta$ = 127 in.

Check the length of beam < $1/\beta$:

$L$ = 13'-6" = 162.0 in.

$1/\beta = 1/(0.0062/\text{in.}) = 161.3$ in.

162 in.$\cong$ 161.3 in.

Therefore, the beam may be assumed to act as a rigid body in delivering the load to the joists.

*Solve for the reaction at each joist:*

The reaction at the joist is equal to the deflection at the joist multiplied by its stiffness.

The rigid body displacement of the beam is shown in Figure 5.5.2.

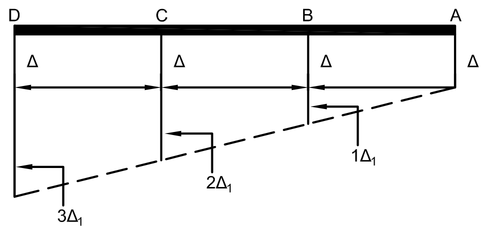



**Fig. 5.5.2 Deflected Shape of the Beam**

Based on the rigid body displacement of the beam each joist force equals the stiffness of the joist, K, times the displacement of the joist.

Determine forces in the y direction.

$\Sigma Fy = 0$

$K(\Delta+3\Delta_1) + K(\Delta+2\Delta_1) + K(\Delta+\Delta_1) + K\Delta\text{-}P = 0$

$4K\Delta + 6K\text{D}_1\text{-}P = 0$

$$\Delta_1 = \frac{P - 4K\Delta}{6K}$$

Sum the moments about point A:

$\Sigma M_A = 0$

$K(\Delta+\Delta_1)(4.5\text{ ft}) + K(\Delta+2\Delta_1)(9\text{ ft}) + K(\Delta+3\Delta_1)(13.5\text{ ft-}P(7.5\text{ ft}) = 0$

Reducing:

$27K\Delta + 63K\Delta_1\text{-}P(7.5\text{ ft}) = 0$

Substituting for $\Delta_1$ and solving:

$27K\Delta + 63K(P/6K\text{-}4K\Delta/6K) - (7.5\text{ ft})P = 0$

$27K\Delta + 10.5P - 42K\Delta - 7.5P = 0$

$\Delta = -3.0P/(-15K) = \text{P}/15K = 1\text{ kip}/[(5)(2.78\text{ kips/in.})]$

$= 0.072$ in.

$$\Delta_1 = \frac{(1\text{ kip})(-\ 4)(2.78\text{ kips/in.})(0.072\text{ in.})}{(6)(2.78\text{ kips/in.})} = 0.012\text{ in.}$$

Solving for the reactions:

$R_A$ = (2.78 kips/in.)(0.072 in.) = 0.20 kips

$R_B$ = (2.78 kips/in.)(0.072 in. + 0.012 in.) = 0.23 kips

$R_C$ = (2.78 kips/in.)[(0.072 in. + (2)(0.012 in.)] = 0.27 kips

$R_D$ = (2.78 kips/in.)[(0.072 in.+ (3)(0.012 in.)] = 0.30 kips

The maximum joist reaction is 300 pounds plus the tributary weight of the hanger beam. The joist reactions are shown in Figure 5.5.3.

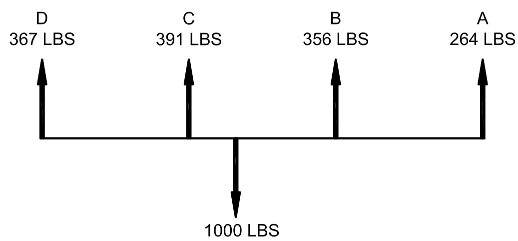



**Fig. 5.5.3 Joist Reactions**

The joist strength for this loading could now be verified.

If the support beam is not sufficiently rigid to be assumed to act as a rigid body relative to the joists the specifying professional may wish to refer to references for beams on elastic support. It can be seen from this example that a relatively stiff spreader beam is required to distribute the hanging load to joists beyond those immediately adjacent to the hanging load.

As an alternative, the specifying professional may model the beam and joist assembly using a frame analysis program.

## Adding New Joists

Once it has been determined that the existing system is inadequate, the specifying professional must decide if it is more appropriate to reinforce the existing joists or to add new joists to the system. As the following discussion indicates, there are several problems associated with adding joists to an existing system, and usually it will be more efficient to reinforce the existing members. If it is decided to add new joists to the system, then a standard joist or a special joist must be chosen to carry the new load.

Access into the structure with the new members should be considered, as well as the disruption of piping, ducts and electrical lines. If the new loading also loads the roof or floor deck, it may overstress the deck at the present joist spacing. In this situation, new joists will have to be added to reduce the span of deck. This is frequently the case when new adjacent construction causes snow accumulation on an existing building.

Typically, the top chords of the joists are laterally stabilized by attachment to the supported deck. A new joist will have to be attached to the deck, or braces will have to be provided from an adjacent stabilized top chord. When specifying the new joist with discrete point bracing, the joist supplier should be made aware of the top chord brace spacing. The bottom chord should be attached to the existing bridging.

There may be some difficulty in the installation if the camber of the new joist does not match the deflected shape of the existing joists. If the new joist is supplied with camber, it will be difficult to install this joist between the deflected deck and the joist support. If the new joist is designated as having no camber, then the space between the joist and the deck may have to be shimmed.

The most difficult problem associated with adding an additional joist is placement room. For example, if the supports are thirty feet apart, it is almost physically impossible to wedge a thirty-foot-long joist between the existing joists and slide it into place between the deck and the joist supports. One possible solution to this problem is to order a joist with one end eccentric and with extra bearing length. Both ends should be ordered with shallower seats so that the joist can be slid into place and then shimmed. Another solution is to order a joist with a bolted splice within the span. Both solutions also require that the replacement joists have no camber or reduced camber.

### Joist Reinforcement

The design of joist reinforcement can best be dealt with by considering the joist as being composed of three major components: the chords, the webs and the end seats. Each of these items can be considered and reinforced largely independent of the others. The chords determine the flexural strength of the joists. The allowable axial force in the weaker chord (top or bottom) times the effective depth of the joist is equal to the flexural strength of the joist. The strength of web diagonals determines the shear strength of the joists. The strength of the end seat determines the allowable end reaction of the joists.

The moment strength of a given joist can be determined from the standard load tables. For H joists, the moment strength is tabulated directly. For K, LH, and DLH joists, the moment strength may be determined by calculating the moment due to the allowable uniform load as given in the joist load tables. The allowable axial chord force may be determined by dividing the allowable moment by the effective joist depth. The effective depth is the total depth less the distances from the angles outstanding legs to their centroid.

The allowable axial force in the compression chord may vary along the length of the joist. Vulcraft may have connected the chords with spacers between the panel points to limit the unbraced length of the individual chord angles. The spacers may not have been used near the ends of the joist, as the required axial strength is less at the ends of the chords. The existence of spacers will have to be checked with a site visit.

The moment on the joist, due to the actual loading, may be determined as it would be for any simple span member. The actual chord force can then be determined by dividing the moment by the effective joist depth. If the actual chord force exceeds the allowable chord force, the chord must be reinforced. The reinforcement must extend beyond the point where the actual moment exceeds the allowable moment and must be fully developed beyond this theoretical cut off point. Enough weld should be provided at the panel points to transfer the horizontal force component of the web member into the chord. The spacing of the welding should also be spaced to control buckling of the reinforcement between the intermittent welds. Complete uniform welding of the reinforcement to the joist chord is usually not required.

Large amounts of welding should be avoided, and the amount of weld applied to a given joist element should be carefully controlled. If excessive amounts of welding are required, or if the dead load stresses are high, members should be shored while the member is being reinforced. AWS D1.1 “Structural Welding Code” (AWS, 2015) states that “the specifying professional shall determine the extent to which a member will be permitted to carry loads while heating, welding, or thermal cutting is performed.” If the specifying professional determines that the existing stresses need not be relieved prior to reinforcement, the reinforcement design should account for the existing stress in the members.

To evaluate the joist web, it is necessary to draw two shear diagrams. The allowable shear diagram should be drawn to scale, then the actual shear diagram should be superimposed (to scale) over the allowable shear diagram. The portions of the actual shear diagram that fall outside of the allowable shear diagram indicate locations of the joist diagonals that require reinforcement. In addition, to causing locations of high shear, the occurrence of large concentrated loads on joists may also cause force reversals in some of the diagonal members. If the point of zero shear on the actual moment diagram deviates significantly from the center of the joist, the diagonals in this area will shift from tension into compression or the reverse. Diagonals that have shifted from tension into compression may require reinforcement.

The allowable shear diagram can be constructed from information derived from the joist load tables and the SJI Specifications. The maximum allowable end shear is equal to the allowable uniform load times half the joist span for the given joist. The allowable shear at the center of the joist is a percentage of this value. The correct percentage is given in the SJI Specifications for the series of joist being considered. K-Series joists are designed for a centerline shear of one quarter of the maximum end shear.

Round bar and single angle web members are usually reinforced by adding an angle to each side of the web. Double angle web members may be reinforced by adding rods or bars to the angles to increase their area. The welds connecting the diagonal reinforcement to the chords must also be designed or verified.

The joist end seat may require reinforcement if the actual shear diagram is outside of the allowable shear diagram at the support. The end seat can be reinforced by adding vertical plates between the joist bearing seat and the outstanding leg of the top chord.

After the joists have been reinforced, the reinforcement should be inspected. The inspection should verify the size and location of reinforcement is as was specified. The reinforcing members should be in line from work point to work point. The welding should be visually inspected. Other methods of weld inspection are impractical (and not appropriate) for the given configuration of joist elements and welds.

The following example illustrates the principals involved in the reinforcement of joists. Note that the reinforced joist has considerably more strength than is required for the new loading condition. Given the unknowns associated with the reinforcement of joists, some conservatism seems justified. The added strength can be acquired at little cost, since the incremental cost of material in the reinforcement of joists is negligible. The largest portion of cost for this type of work is for set up and labor.

The reader is referred to the SJI TECHNICAL DIGEST #12, "Evaluation and Modification of Open Web Steel Joists and Joist Girders," for additional discussion and examples for joist and Joist Girder evaluation and modification.

### Example 5.5.2 Joist Reinforcement

Reinforce a joist to support the uniform load and a new hanging load of 2,000 lbs. located 10' from the left end.

**Given:**

From existing plans, the joist is a 20K7.

Uniform applied load = 275 plf

Length = 33 feet

The top chord angle dimensions were field measured as 1-3/4x1-3/4x0.156 inches.

For the two angles:

$A \cong 1.04$ in.$^2$

$I_x \cong 0.306$ in.$^4$

The distance between panel points is 24 inches according to field measurements.

### Solution:

Determine the chord strength:

Determine the end reactions:

$R_L$ = (0.275 kips/ft)(33 ft)/2 + 2(23 ft/(33 ft) = 5.93 kips

$R_R$ = (0.275 kips/ft)(33 ft)/2 + 2(10 ft)/(33 ft) = 5.14 kips

Determine the maximum moment:

$M_{max} = (R_R)(x) - (w)(x^2/2)$

where

$x$ = 5.14 kips/0.275 kips/ft = 18.70 ft.

$M_{max}$ = (5.14 kips)(18.7 ft) - (0.275 kips/ft)(18.7 ft)$^2$/2

= 48.1 kip-ft

$M_r$ = 577 kip-in.

Determine the allowable moment:

From SJI load tables $W_{allow}$ = 309 plf.

$M_a$ = (0.309 kips/ft)(33 ft)$^2$/8 = 42.1 ft.- kips. = 505 kip-in.

577 kip-in. > 505 kip-in.

Chord reinforcement is required.

It should be noted that if the joist designation is not known then the calculation of the chord strength is more complex. The chord strength depends upon whether the chord is fully effective, i.e. $Q$ = 1.0, and the number of battens (plugs) between the chord angles for control of chord buckling about the z-axis.

Determine the amount of flexural reinforcement required:

The joist will be shored prior to reinforcement, so pre-stress need not be considered.

The required additional chord force equals $M_r$ - $M_a$ divided by the effective depth of the joist. The additional area can be calculated by dividing the additional required chord force by the effective depth of the joist.

Additional required chord force = (577 in.- kips - 505 in.- kips)/19.0 in. = 3.79 kips

Where the effective depth = 20 in.- 2$y_{bar}$ of the chords.

$y_{bar}$ of both chords is taken as 0.5 in.

Try adding (2)¾ in. diameter rods to each chord, $F_y$ = 50 ksi

$A$ = 2(0.442 in.$^2$) = 0.884 in.$^2$

The adding of the rods reduces the radius of gyration of the top chord. This effect is primarily about the x-axis.

Check compression chord after reinforcement:

Required chord force, $P_r = M_r/d$ = 577 in.- kips /19.0 in. = 30.4 kips

Determine the allowable stress for the reinforced top chord:

$I_x$ for the chord angles = 0.306 in.$^4$ (given)

$A$ = 1.04 in.$^2$ + 0.884 in.$^2$ = 1.92 in.$^2$

$$r_x = \sqrt{\frac{0.306 \text{ in.}^4}{1.92 \text{ in.}^2}} = 0.40 \text{ in.}$$

$L_c/r_x = KL/r_x$ = 1.0(24 in.)/0.40 in. = 60

$F_{cr}/\Omega$ = 23 ksi AISC Manual Table (4-14)

$P_a = (F_{cr}/\Omega)A$ = (23 ksi)(1.92 in.$^2$) = 44.2 kips

44.2 kips > 30.4 kips **o.k.**

Calculate cut off points for the chord reinforcement:

Locate the distance ($x$) from the right end:

$M_a$ = (505 kip-in.)/12 = 42.1 ft - kips

$42.1 = 5.14x - 0.275x^2/2$

$x^2 - 37.4x + 306 = 0$

$x = 12.1$ feet.

Locate the distance ($x$) from the left end:

$42.1 = 5.93x - 0.275x^2/2$ (for $x < 10$ ft, $x^2 - 43.1x + 306 = 0$

x = 8.95 feet.

The chord reinforcement must be fully developed at these locations.

Determine the welding required for the chord reinforcement:

Per AWS the effective throat of a flare bevel weld is $0.3125r$, where $r$ is the radius of the curved member.

With the ¾-inch rod the allowable weld force equals 2.46 kips/in.

Required weld at the ends of the ¾ in. reinforcing rods:

Required length of flare bevel weld = 13.3 kips/2.46 kips/in. = 5.41 inches

Provide 6″ of flare bevel weld at the ends of each rod.

Required weld along the length of the ¾ in. rods:

Rod force:

$$\frac{R_n}{\Omega} = \frac{F_y}{\Omega} A = \left(\frac{50 \text{ ksi}}{1.67}\right)\left(0.442 \text{ in.}^2\right) = 13.3 \text{ kips}$$

$I_{joist} = 26.767(W_{LL})(L^3)(10^{-6})$

$I_{joist} = 26.767(181 \text{ plf})(33 \text{ ft}-0.33 \text{ ft})^3(10^{-6}) = 169 \text{ in.}^4$

$I_{eff} = 169 \text{ in.}^4/1.15 = 147 \text{ in.}^4$

Required shear flow, $v$:

$v = VQ/I$

Conservatively use the left end reaction for $V$:

$v = (5.93 \text{ kips})(0.442 \text{ in.}^2)(19.0 \text{ in.}/2)/147 \text{ in.}^4 = 0.17$ kips/in./rod

The shear flow is introduced into the chord at each panel point. Using the panel point spacing of 24 inches and the strength of the flare bevel weld of 2.46 kips/in. on a ¾ in. diameter rod, determine the length of flare bevel weld required at each panel point.

Length of weld req'd = (0.17 kips/in.)(24 in.)/2.46 kips/in. = 1.66 inches

Provide 2 inches of flare bevel weld at each panel point. Also, additional weld between each panel point to control buckling of the rods.

For the compression chord, limit $L/r$ of the rod to .75 of the $L/r$ of the reinforced chord:

Radius of gyration for ¾ in. round rod = 0.1875 in.

Solve for the maximum spacing of welds:

$L/r_{rod} = 0.75L/r_{chord}$

Maximum weld spacing = 0.75(60)(0.1875 in.) = 8.4 in.

Provide 2 in. of weld at 8 inches on center between the panel points.

Check the web strength:

$V_R = wL/2$

$V_R = (0.309 \text{ kips/ft})(33 \text{ ft.})/2 = 5.1 \text{ kips}$

Minimum shear per SJI = $V_R/4 = V_R/4 = 1.27$ kips

Construct the allowable and actual shear diagram.

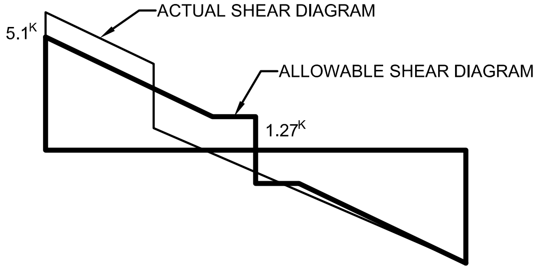



**Fig. 5.5.4 Shear Diagram**

From inspection of the shear diagram, it can be seen that the diagonals from the left end to the load require reinforcement.

Determine web reinforcement:

Field measurements have provided the panel point locations as shown in Figure 5.5.5.

Conservatively add diagonal angles along the web members to carry the entire shear force to each side of the chords.

Use A36 reinforcing angles.

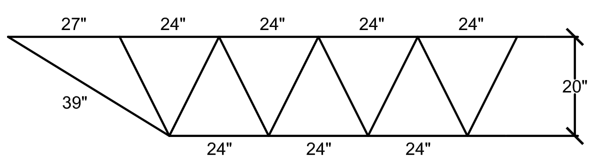



**Fig. 5.5.5 Joist Measurements**

Determine the forces in the diagonals:

Tension in the end bar; $T = 12.8$ kips

Maximum compression rod; $C = 5.96$ kips

Tension diagonal:

$A_{req'd} = 12.8 \text{ kips}/(36 \text{ ksi}/1.67) = 0.58 \text{ in.}^2$

Use 2 L 1-1/4x1-1/4 x 3/16, $A = 0.868 \text{ in.}^2$

Length of 3/16″ weld required = 12.8 kips/2.78 kips/in.= 4.6 inches

Use 3″ of 3/16″ fillet on each angle.

Compression diagonal:

Try 2 L 1-1/4x11/4 x 3/16

$A_{req'd}$ = 0.868 in.$^2$

$r_x$ = 0.377 in.$^2$ (for two angles, battens are required between the angles)

$L_c/r_x$ = 38.0 in./0.377 in. = 101

$F_{cr}/\Omega$ = 12.4 ksi.

$P_{cr}/\Omega$ =($F_{cr}/\Omega$)(A)

$P_{cr}/\Omega$ = (12.4 ksi)(0.868 in.$^2$) = 10.8 kips **o.k**.

Reinforce the end seats in order to attach the new end diagonals. Try adding 0.25" x 2"x 5" long plates to each side of the seat (See Figure 5.5.8).

Check plate shear stresses:

From statics, the horizontal force component in the end diagonals equals 11.5 kips. The vertical force component equals 5.6 kips. Thus, each side plate receives a horizontal force, $H$ = 11.5 kips/2 = 5.75 kips and a vertical force $V$ = 5.6 kips/2 = 2.8 kips.

Assume that the horizontal force component is resisted by the weld between the plate and the top chord. The stress in the 3/16 in. x 5 in. long weld equals 5.75 kips/[(0.707)(0.1875 in.)(5 in.)] = 8.68 ksi.

Allowable shear stress:

$R_n/\Omega = 0.6F_{EXX}/2.0$ = (0.6)(70 ksi)/2.0 = 21 ksi

8.68 ksi < 21 ksi **o.k.**

The vertical force component is resisted by the weld between the plate and the seat angle.

The weld stress equals 2.8 kips/(0.707)(0.1875 in.)(5 in.) = 4.2 ksi < 21 ksi. **o.k**

Since the weld stresses are low it is assumed that if the diagonal is not positioned exactly at the weld group centroid the resulting eccentricity of load will not overstress the welds. It can also be seen that the top chord will not be overstressed locally at the weld location since the chord thickness is greater than the weld throat thickness.

The required reinforcement is illustrated in Figures 5.5.6, 5.5.7 and 5.5.8.

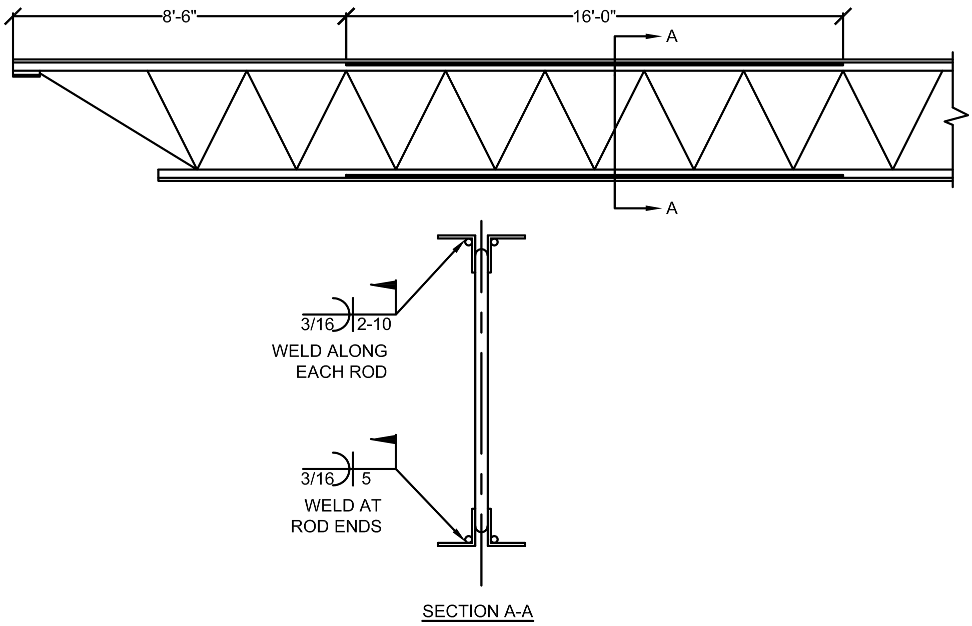



**Fig. 5.5.6 Joist Chord Reinforcement**

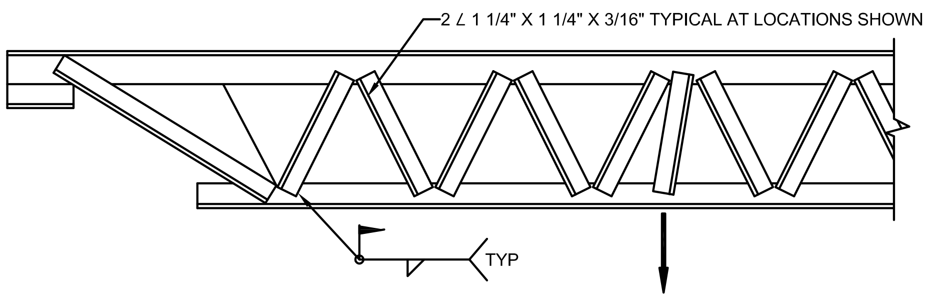



**Fig. 5.5.7 Joist Diagonal Reinforcement**

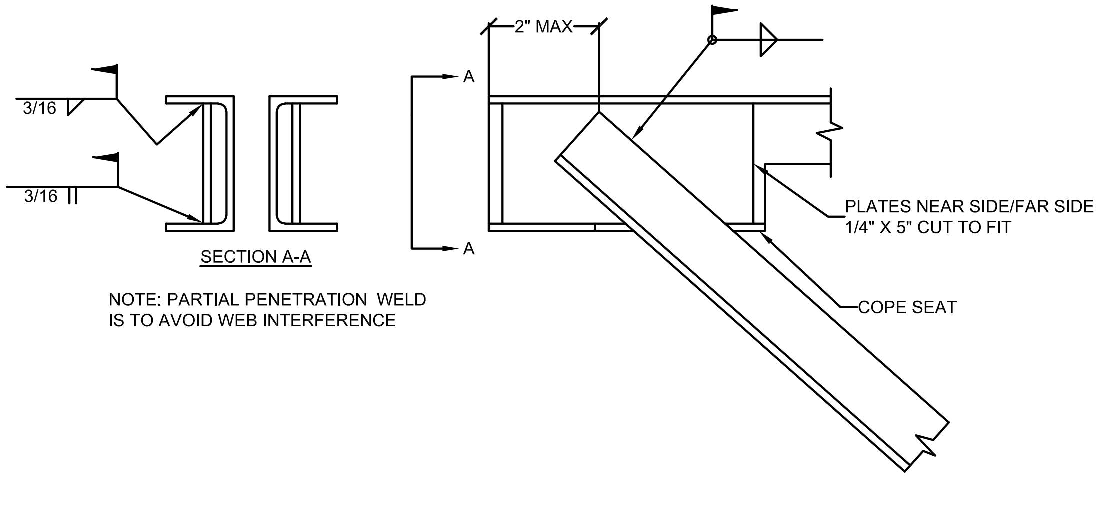



**Fig. 5.5.8 Joist End Seat Reinforcement**

## 5.6 FLOOR VIBRATIONS

All elevated floor systems respond to pedestrian traffic or other activities. Vibration of the floor is one response. The magnitude and duration of the vibration may vary from one floor system to another. The occupants may not be able to perceive any vibrations, or the vibrations may be so severe as to be disruptive to the occupants. Most floors may be considered as serviceable with respect to the perceptibility of vibrations. The criterion for determining the serviceability of the floors is based on whether the occupants are annoyed by floor vibrations. This criterion is, rather nebulous, and the design of a floor support system that meets this requirement must be based on the sound judgment of a qualified specifying professional using researched and documented design techniques.

In general, floor vibration considerations can be grouped into two categories: vibrations due to rhythmic or repeating excitation and vibrations due to transient vibrations.

The human perception of transient floor vibration relates to the frequency, amplitude and duration of the vibration transmitted through the floor. The related structural characteristics are the natural frequency, stiffness and the amount of damping available in the floor system. The frequency and amplitude of the vibration define the acceleration of motion that is felt by the occupants. At lower frequencies a higher amplitude may be tolerated by humans without discomfort. At higher frequencies a lower range of amplitude is more easily perceived by humans. Damping defines the rate of decay of the amplitude of vibration. A system has 100% of critical damping if the initial displacement is not repeated. If a floor system has a relatively large amount of damping, the magnitude of motion may quickly be reduced to an imperceptible amount. In this case, vibrations are not perceived by the occupants. There is a certain amount of damping inherent in a steel joist and concrete floor system. Additional damping is provided by elements supported by or attached to the floor system such as ducts, ceilings, partitions, furnishings and even people.

The current research about design of steel framed floor systems subjected to transient vibrations is contained in the AISC Design Guide No. 11 “Vibrations of Steel-Framed Structural Systems Due to Human Activity” (AISC, 2016b). The Design Guide provides:

- A method of determining the natural frequency of the floor system.
- Recommendations for the amount of damping that could reasonably be anticipated for different building types.
- Recommendations for an acceptability criterion based on a maximum acceptable acceleration for different occupancies.

The Steel Joist Institute also publishes SJI Technical Digest 5, ‘Vibration of Steel Joist – Concrete Slab Floors (SJI, 2015b), which provides further guidance and clarification concerning this specific floor framing system. Vulcraft provides an online design tool (www.vulcraft.com/design-tools/vibration-analysis-tool) to help the specifier to analyze for transient (walking) vibrations in order to determine joist specification information, layout, concrete thickness and other details needed to ensure that the end owner will get a concrete-slab-on-joists system that meets their expectations. This tool includes recommendations from both AISC DG 11 and SJI TD 5 as well as significant guidance for the specifier in economical ways to ensure vibration is properly addressed. When properly designed both composite and non-composite floor joists provide satisfactory performance relative to vibration behavior.

Repeating forcing functions occur in gymnasiums or aerobics areas where rhythmic exercises occur, or in large shopping centers or airport concourses where people walk long distances at a steady pace. If a floor area is subjected to a repeating forcing function, the designer must not base his analysis of floor serviceability on criteria that include the effects of damping. This is because the beneficial effects of damping are nullified by the repetitive loading. If a forcing function continues a rhythmic basis that is near the natural frequency of the framing members, the function will continue to be amplified as the motion is successively reintroduced prior to being damped out. In this type of situation, it is desirable to provide framing members of a natural frequency that is higher than the frequency of the forcing function. Also, the amplitude of motion imparted by the footfalls should be limited. The AISC Design Guide 11 should be consulted for guidance as to the acceptable ranges of amplitudes and frequencies for floor systems subjected to rhythmic excitation.

Problems in steel joist and concrete floor systems are most common in floor systems with closely spaced joists that are 28 feet long and support a relatively thin (2.5 inches thick) concrete deck. The dynamic characteristics of joists improve with longer and shorter spans. Increasing the thickness of the deck (with more concrete) also improves the behavior of the floor system. The heavier deck increases the mass of the system and increases the number of participating effective joists. For a given span, increasing the mass will improve the behavior of the floor. Simply increasing the size of the joist is not as effective or economical means of improving the floor's behavior. The authors have found that using a relatively heavy floor deck with floor joists spaced 4 to 5 feet apart creates a very economical floor system. The cost of this system compares favorably with systems using more closely spaced joists, and floor vibrations are greatly decreased.

The designer is cautioned against taking a "cookbook" approach to the analysis of a floor's vibration characteristics. An estimate of the amount of damping present within the joists effective width and the mass of the system while in use should be based on experienced judgment. The specifying professional should consider the possibility of the primary beams or girders as contributing to the floor's vibrations. Also, the areas around floor openings should be considered. The level of perceptibility of floor vibrations (within acceptable limits) defines the quality of the floor system. It is not the responsibility of the joist supplier to determine the requirements of the building use and specify a floor that meets those requirements. If requested, Vulcraft will provide technical assistance to aid the specifying professional making the required decisions about span direction, member size, spacing and floor thickness.

In addition to human perceptibility considerations, it is possible that floor vibrations may affect the functioning of equipment. Lab equipment such as electron microscopes or medical equipment may be sensitive to floor motion. Properly designing a floor to serviceably support such equipment requires coordination with the equipment supplier. Guidance for the design of floors supporting sensitive equipment can be found in Design Guide 11. It may be necessary for some equipment to be placed on the base floor supported by the soil. Vibrating equipment should be isolated so as not to impart troublesome vibrations into the system. The design of joists supporting vibrating equipment is discussed in the section on roof top units.

## 5.7 SPANDREL SYSTEMS

The design of the framing along the building perimeter warrants special attention, because it is at the interface of the building frame with the exterior wall. The principal areas of concern are:

1. Proper design and detailing of the projected framing from the center line of the perimeter framing to the building edge
2. Deflection control of the perimeter framing
3. Control of the dimensional tolerances in the building perimeter construction

In only rare instances would there be no projection of the building edge beyond the center line of the perimeter framing, so some sort of cantilever is almost always required. On the building sides which are perpendicular to the joist framing, the standard approach is to extend the joist end beyond the center line of the perimeter framing. In ascending order of strength, extended ends are:

1. Extensions of the top chord angles
2. Extensions of the standard joist end
3. Extension of a specially designed extra depth end

4. Full depth extension of the joist, resulting in a bottom bearing joist

*Top Chord Extensions (TCXs):*

The standard details of extended ends vary among manufacturers. SJI has tabulated uniform capacities vs. lengths for extensions on K-Series joists using standard designations, namely S1 through S12 which are extended top chords and R1 through R12, which are extended ends. These tables are provided under the title of “Load Tables LRFD/ASD – TCX” in the Vulcraft Steel Joist & Joist Girder Systems Manual (Vulcraft, 2017c). The allowable loads decrease as cantilever length increases. The maximum load is 550 lbs. per foot which matches the maximum load per foot for K-Series joists. These standard load tables apply to uniform loads only and not concentrated loads. However, since standardized section moduli and moments of inertia are given, standard ends with uniform and concentrated loads can be evaluated. The digit following the “R” and “S” corresponds to the digit following the “K” in the standard K-Series joist designation. The designer should not designate an extension where the extension digit exceeds the joist digit. If this limitation cannot be met due to design requirements, then the manufacturer must increase the joist chord size and in most cases an end depth greater than the standard 2½" will be required. The table can be used as a guide in order to understand the capabilities of 2½" deep extensions relative to joist size. Contact Vulcraft for the most economical design and bearing depth. Loads on extended ends must be provided on the construction documents by the specifying professional. Also, the deflection limits and bracing requirements must be given. Another reason that the extended ends must be considered at the time the construction documents are prepared is that the depth of the end must be set so that the elevation of the perimeter Joist Girder can be established.

The joists located on the column center lines pose a problem in floor construction. For roofs the extension can pass over the top of the column. Since the columns interrupt the joists in floors, two methods can be used to solve this condition. Either a cantilevered bracket can be detailed from the side of column or the joist extension on either side of the column can be designed for additional concentrated load by using the perimeter element to carry a small header spanning between the column and the perimeter element.

*Cantilever Extensions:*

The extension from framing centerline to building perimeter on the ends parallel to the joist framing is not solved as easily as the perpendicular condition. In general, three approaches are available: 1) cantilevering the deck, 2) cantilevering a bent metal plate and 3) extending outrigger framing which is perpendicular to the edge.

Where possible, the best approach is to cantilever the deck. Based on the recommendations of the Steel Deck Institute, roof decks (A, F, B) can be cantilevered in the range of 1'-0" to 2'-10" depending on the type and gage. Such cantilevers are not intended to carry significant downward concentrated loads but can be used as a lateral tie back point for the wall system when appropriate, as in the case of metal panel walls supported on intermediate girts. It is also possible to cantilever floor deck. For short cantilevers with minimal loads, sheet metal pour stops can be used, but for overhangs of any appreciable dimension, top reinforcement in the concrete slab should be provided. The Steel Deck Institute has published a “Pour Stop Selection Table” (SDI, 2017e) which gives design thicknesses for various combinations of slab depth and overhang. This table does not provide live load strength, nor does it consider the potential for rotation or deformation of the perimeter steel.

Cantilevering the steel deck and concrete can be done when a pour stop would not be adequate. In this case, the steel deck must be checked for its adequacy as a cantilevered form, and the concrete and steel section must be checked as a cantilever. Top reinforcement in the concrete will generally be required for the negative moments over the perimeter framing. If there is a concentrated load at the slab edge, attention must be given to the development of the reinforcement and may require concrete anchors welded to the edge steel and lapped with the negative moment reinforcement. In most construction, reinforced cantilevered concrete of the building ends will be a marked departure from the remainder of the deck so field follow up will be needed to ensure that the requirements are in fact executed.

Cantilevered bent plates are sometimes provided at the end edges of roofs and floors. In floors, they resemble the pour stops discussed above. In both floors and roofs, they rely on the torsional strength and stiffness of the perimeter framing. Since these perimeter members are usually light for reasons of load, they usually have low torsional strength or stiffness.

The use of cantilevered outriggers is recommended where heavy concentrated loads are present at the building perimeter. Vulcraft's standard outriggers are available and, of course, custom designed outriggers can be used. The use of outriggers necessitates reducing the top elevation of the joist on the exterior column line below that of the other joist in that bay so the outriggers can cantilever over and be supported by them. If the deck can span from the first interior joist to a parallel edge member at the outrigger ends, there is no need to turn the deck between outriggers. However, turning the deck to span between outriggers can be easily accomplished if required. Vulcraft's standard details show the "in board" end of outriggers welded to the underside of the first interior joist. This may or may not be appropriate in a given situation depending on loads. This detail should be used if feasible.

*Wall Support Systems:*

In addition to supporting the dead and live loads from the center line of perimeter framing to the building edge, the cantilevered perimeter must frequently support the exterior wall. In general, there are three conditions of support for the exterior wall.

1. It can be supported on the foundation and tied back to the structure for lateral support and stability.
2. It can be anchored to the columns for vertical and lateral support.
3. It can be supported continuously along the building edge.

In the first two situations there is no gravity load from the wall on the cantilever. The only requirement is that the perimeter carry the lateral loads, and the vertical deflection be compatible with the roof/wall or the floor/wall joint. Structures which carry the gravity load of the exterior wall along the cantilevered edge require greater attention in the design. First, the cantilevered perimeter receives a potentially significant concentrated load at the end and, depending on the system, the wall may not load all the extensions equally. Secondly, deflection of the cantilevered edge and the perimeter framing play an important part in the proper performance of the exterior wall. Deflection at each story must be limited to prevent inadvertent loading of the exterior walls below. The deflection should be limited so that the cladding supplier's standard story to story relief joint can be used and can function properly. Deflection must be restricted so that the building perimeter is relatively stable as the load is added during the erection of the exterior wall. This is especially true in the case of relatively heavy exterior walls such as masonry, stone or precast. The following criteria have been proposed by (Fisher, J.M. and West, M.A., 2019) for the support of floor and roof edge supported exterior walls. These limits would apply to the total of cantilever deflection and deflection along the perimeter framing.

1. Deflection due to dead load prior to setting exterior wall: 3/8" max
2. Deflection due to dead loads and weight of exterior wall: span over 480 to a maximum of ⅝"
3. Deflection due to dead load and the weight of exterior wall when the exterior wall weight exceeds 25% of the total dead load: span over 600 to a maximum of ⅜"
4. Deflection due to live loads: span over 360 to a maximum of ¼" to ½" depending on details

The cantilevered edge represents an important interface between the structural frame and a principle architectural component. These two systems are designed by separate disciplines and are installed by different trades. Because of this, care must be taken to coordinate details, dimensions and especially tolerances. Control in detailing, fabrication and erection must allow for in and out deviations in the edge and vertical variations from the idealized floor elevation. Also, the vertical face of pour stops, and edge angles must be truly vertical for systems which are attached to the face of the edge. In general, the tolerances to which edges pieces must be set are stricter than that of structural material. The tolerances should be clearly set forth on the construction documents as should the expected deflections. Where extended ends and outriggers are used, the documents should clearly state the dimension and associated tolerance for the distance from end of joist/outrigger to the face of the exterior edge. Also, provision should be made to accommodate the embedded anchors and fasteners required to secure the wall system. These are sometimes cast into the top of the slab or are sometimes fastened to the face of the edge. Coordination and complete construction documents are required.

## 5.8 PONDING

The SJI "Standard Specification for Steel Joists, K-Series, LH-Series, DLH-Series and For Joist Girders" states in paragraph **5.11 PONDING**, "The ponding investigation shall be performed by the specifying professional."

Rainwater causes the deflection of a roof system, which in turn increases the volumetric capacity of the roof. Additional water is retained which in turn causes additional deflection and volumetric capacity. The process is iterative. The purpose of a ponding check is to ensure that convergence occurs, i.e. that an equilibrium state is reached for the incremental loading and the incremental deflection. Also stress at equilibrium must not be excessive.

The 2015 IBC Code **Section 1608.3** references ASCE 7 and states, "Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Section 7.11 of ASCE 7." The IBC also makes the following reference in **Section 1611.2**, "Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Section 8.4 of ASCE 7."

ASCE 7-16 requires in **Section 7.11** "Susceptible bays shall be designed to preclude ponding instability. Roof deflections caused by full snow loads shall be evaluated when determining the likelihood of ponding instability (see Section 8.4)." Section 8.4 states, "Susceptible bays shall be investigated by structural analysis to ensure that they possess adequate stiffness to preclude progressive deflection (i.e., instability) and adequate strength to resist the additional ponding load. Any of the following conditions shall be deemed to create susceptible bays: (1) bays with a roof slope less than 1/4 in: per foot (1.19°) when the secondary members are perpendicular to the free draining edge, (2) bays with a roof slope less than 1 in. per foot (4.76°) when the secondary members are parallel to the free draining edge, (3) bays with a roof slope of 1 in. per foot (4.76°) and a span to spacing ratio for the secondary members greater than 16 when the secondary members are parallel to the free draining edge, or (4) bays on which water accumulates (in whole or in part) when the primary drain system is blocked but the secondary drain system is functional. The larger of the snow load or the rain load equal to the design condition for a blocked primary drain system shall be used in this analysis."

Other references to ponding instability can be found in the SJI Technical Digest 3, "Structural Design of Steel Joist Roofs to Resist Ponding Loads." (SJI, 2018a)

Ponding as a structural design phenomenon is of concern for two reasons:

1. The loading is water which can fill and conform to a deflected roof surface.
2. The source of load (water) is uncontrollable, i.e. nature.

When water can accumulate on a structural system due to impoundment or restriction in drainage, ponding must be checked. Reasons for the accumulation can be:

1. Dead load deflections of members in roofs designed to be flat
2. Deflections of members which places points in their spans below their end points
3. Deflections of bays supporting mechanical units
4. Members installed with inverted cambers

5. Blocked roof drains
6. Parapets without scuppers
7. Parapets with blocked scuppers
8. Intentional impoundment of water as part of a controlled flow roof drain design
9. Low slope roofs which allow water to accumulate due to the hydraulic gradient

The surest way to avoid a ponding collapse is to construct a roof with enough slope and free drainage, so that water never accumulates. What is enough slope and what constitutes enough drainage? Roof slopes varying from 1/8 in./ft. to 1/2 in./ft. have been used successfully in the past, but it cannot be stated that in all cases such slopes prevent ponding collapse. Rational analysis to answer these questions requires knowledge of both the structural and hydrological characteristics of the roof. Roof slope, stiffness and strength of the members supporting the roof membrane, as well as the location and size of drains are all important in avoiding ponding instability.

It should be noted that the 2015 IBC requires that the roof slope shall not be less than ¼ in. per foot. In **Section 1507.10.1 Slope.** "Built-up roofs shall have a design slope of not less than one-fourth unit vertical in 12 units horizontal (2-percent slope) for drainage, except for coal-tar built-up roofs that shall have a design slope of not less than one-eighth unit vertical in 12 units horizontal (1-percent slope)."

From Technical Digest 3: "The recommended general procedure for roof design for ponding is as follows. First, select a joist system to carry the primary design loads with the use of the SJI Load Tables (SJI, 2015a). Then, check the adequacy of the design for ponding. If the joist system is determined to be adequate, no further ponding checks are required, and the design can proceed. If the joist system is determined to be inadequate for ponding, either the stiffness, strength or both, the system should be increased. The most efficient method will vary based on the specific loading and roof configuration. This can be accomplished by any of the following or a combination of the following:

- Decreasing the joist spacing
- Increasing the joist size for the original spacing
- Increasing the joist depth
- Increasing the Joist Girder depth
- Increasing the Joist Girder panel point load

The AISC Specification ponding provisions have a long history of successful use in design practice. In addition to the assumption that the entire bay is covered by water, several potentially important effects are neglected:

The un-deformed shape of the roof is perfectly flat, even though a minimum slope is required by the IBC (IBC, 2015) and steel joists and Joist Girders are typically constructed with camber. The method has been adapted to account for low sloped roofs, but to the authors' knowledge no comprehensive study has been performed to evaluate the safety or accuracy of these adaptations.

1. The joist on-stiff-supports method can account for camber, but not sloped joists.
2. The method was originally developed for roof systems with solid web steel beams and girders where a check of the maximum moment is sufficient to assess strength. Open

web steel joists and Joist Girders are typically designed for uniform loads. Accordingly, their shear and moment capacity can vary along their length, requiring strength to be checked along the entire length. This is especially important since, a) the maximum moment experienced under ponding conditions may not occur at mid-span, where the moment capacity is at a maximum, and b) shear reversals near mid-span can occur under ponding conditions causing web members that are designed for tension to be subjected to compression.

3. When joists and Joist Girders which have been designed for uniform loads are subjected to non-uniform loads, additional potential limit states arise. For joists, a region of high applied distributed load can cause bending failure between the panel points. For Joist Girders, high applied panel point loads can cause failure of web verticals.

A direct analysis method can account for all relevant effects in which the load due to impounded water is computed directly. Such an analysis is implemented within the **SJI Roof Bay Analysis Tool** which is available on the SJI website (www.steeljoist.org) under the Design Tools tab.

*SJI Roof Bay Analysis Tool:*

The SJI Roof Bay Analysis Tool utilizes a direct analysis method in which the applied loads are based on the deformed shape.

The SJI Roof Bay Analysis Tool is an excel workbook that can be used as a design aid for estimating purposes and the selection of roof framing. It performs analysis and design of a rectangular bay consisting of four columns, two Joist Girders and several joists. General bay design (not considering the effects of ponding) is performed in the “Roof Bay Analysis” sheet. Ponding analyses are performed in the “Ponding Analysis” sheet. Notes and instructions regarding the ponding analysis are given in the “Ponding Instructions” sheet and analysis results formatted for printing are given in the “Ponding Load Results” sheet.

Results from the analysis are presented in data tables. The first table displays the maximum shear and moment for each joist as well as a strength check. The total applied loads in the ponded configuration are non-uniform as height of water varies with the deflection of the bay. Equivalent uniform distributed loads are computed through comparisons of the computed moment and shears to the available moment and shear envelopes for the standard designated joist. They are determined through a point-by-point comparison as the minimum specified capacity that results in the available strength of the joist equaling or exceeding the required strength at each point along the length of the joist. Equivalent loads are computed separately for moment and shear to readily identify which controls. The strength ratio is the ratio between the larger equivalent load (either moment or shear) and the specified load capacity for the standard designated joist from the SJI Load Tables.

The fourth table displays the joist reactions, panel point loads, and a strength check for each Joist Girder. Equivalent uniform panel point loads are computed through comparisons of the computed moment and shears to the available moment and shear envelopes for the Joist Girder. They are determined through a point-by-point comparison as the minimum specified capacity that results in the available strength of the Joist Girder equaling or exceeding the required strength at each point along the length of the Joist Girder. Equivalent loads are computed separately for moment and shear to readily identify which controls. The strength ratio is the ratio between the larger equivalent load (either moment or shear) and the load capacity of the Joist Girder as determined from its designation.

*Practical Notes for Roof Design for Ponding (From SJI Technical Digest 3):*

1. It is important to have a rational strategy for addressing ponding regardless of the specific methodology employed.
2. Sometimes the best strategy is to eliminate susceptible bays by sloping roof members, or using tapered insulation or sloping fill.
3. Counteract the ponding mechanism by providing upward camber in the joists, provided that drains are installed near columns, see FM Global 1-54 (FM, various dates).
4. When designing roofs with low slopes, parallel chord joists with end supports at different elevations are more economical than providing pitch into the joist top chords. The web system of a non-parallel chord joist and the joist as a whole is more expensive to manufacture.

The specifying professional should also be aware that the moments of inertia used in this tool are based on estimated values using the SJI formulas. To insure both the joists and the Joist Girders are designed with adequate stiffness by Vulcraft, it is recommended that the gross moment of inertia for both the joists and the Joists Girders be specified on the plans as the minimum moment of inertia for those members. The gross moment of inertia $I_g = 1.15I_{eff}$; where $I_{eff}$ is the value used by the Ponding Analysis tab in the tool.

It is recommended the specifying professional use at least the standard SJI camber. Specifying camber less than standard SJI camber will have additional costs for the special case, in addition the decreased camber can lead to higher water loading. If the specifying professional wishes to use camber greater than standard camber, it is recommended the camber be at least 1.5 times standard camber for the span, due to fabrication tolerances for camber. The specifying professional should also be careful not to specify too large a camber, since it could cause additional flat areas in the roof.

Finally, it is the responsibility of the building owner to properly maintain the drainage system so that it will function properly.

The reader is encouraged to examine the SJI Technical Digest No. 3 in detail. Using the SJI ponding tool can provide joist and Joist Girder roof systems with the required strength and stiffness to prevent ponding collapses in roof systems when code compliant drainage is provided by the design professional. Most ponding collapses are due to maintenance issues.

## 5.9 FIRE RESISTANCE

The use of steel joists and Joist Girders in buildings frequently requires the use of fire rated systems of construction. The fire ratings of systems are expressed in hours ranging from one to four hours. The required rating for a roof-ceiling or floor-ceiling assembly in a building is established by the Building Code and is a function of the building's occupancy, area and height. The ratings are to be met using rated assemblies' meeting or exceeding the required rating. Such assemblies typically consist of steel deck, structural members, fireproofing protection and other appropriate materials. Assemblies are constructed and tested according to the methods and acceptance criteria described in ASTM Standard E119, "Standard Test Methods for Fire Tests of Building Construction and Materials" (ASTM, 2018).

The purpose of fire tests is to establish the relative performance of different assemblies under identical laboratory test conditions. Most fire tests over the years have been conducted by Underwriters Laboratories Inc. and descriptions of the rated assemblies are compiled in the UL "Fire Resistance Directory." It is important when applying rated assemblies to a project that all the features of the rated assembly be carried over into the design, or it will not be valid and can delay the issuance of a Building Permit. Common problems associated with this need to strictly follow the requirements of a given assembly are:

1. Increased insulation thickness over the steel roof deck: This could result in greater heat buildup below the assembly and invalidate the fire rating. Certain roof ceiling assemblies in the "Fire Resistance Directory" permit an unlimited thickness for the roof insulation.
2. Substitution of different manufacturers of assembly components
3. Substitution of different products
4. Use of lighter, smaller structural members: The assemblies state the minimum size member. A lighter member may result in reaching the maximum temperature limitation faster because of its smaller mass
5. Use of assemblies which are out of date: Many assemblies contain J-Series joists which are no longer produced. If no K-Series joist is included as an alternate in the assembly, a K-Series joist may be substituted in accordance with the design information section of the "Fire Resistance Directory."

ASTM E119 divides all construction into two classifications based on two conditions of support: restrained and unrestrained. A guide determining the applicability of these classifications is given in Appendix X3 and Table X3.1 at the end of ASTM E119. The listings appropriate to steel joist and Joist Girder framing are:

1. Wall bearing:
   a. Single spans and simply supported end spans of multiple bays with steel joists supporting concrete slab, precast units or steel decking: unrestrained
   b. Interior spans of multiple bays with steel joists supporting continuous concrete slab: restrained
   c. Interior spans of multiple bays with steel joists supporting precast units or steel decking: unrestrained
2. Steel framing:
   a. All types of prefabricated floor and roof systems where the structural members are secured to the framing members and the potential thermal expansion of the floor or roof system is resisted by the framing system or the adjoining floor or roof construction: restrained

The fireproofing protection for floor ceiling and roof ceiling assemblies using steel joists and Joist Girders can be achieved in two ways:

1. Direct application of an insulation material such as a cementitious mixture or sprayed fiber product
2. Installation of a continuous barrier membrane such as a suspended acoustical tile or gypsum board system beneath the framing

The use of a membrane protection permits the use of mechanical components such as air ducts in the plenum area above the membrane. The mechanical systems can be attached directly to the structural framing and can run through the structure as needed. Membrane systems require care and detail in their installation. Since they may be used effectively in certain situations, they should always be considered in the design process.

It should be noted that the determination of the building fire classification, the required assembly fire ratings and the specification of the fireproofing protection and materials are generally within the Scope of the Architect's services. Close coordination with the specifying professional is necessary in order to produce a structural design compatible with the architect's specifications.

Items of structural concern would include:

1. Providing structural member connections and details consistent with the classification of restraint
2. Providing for minimum required structural member sizes and depths
3. Use of appropriate structural member tensile stresses
4. Use of required steel deck profiles and thicknesses
5. Use of appropriate concrete strengths, densities and thicknesses
6. Accounting for the weight of the fireproofing protection system in the design
7. Accounting for the depth of the fireproofing protection system in the overall structural design

A further discussion of fire resistance ratings using steel joists and Joist Girders is presented in the Vulcraft Manual entitled "Steel Joists and Joist Girders." This information includes various types of assemblies and specific fire resistance design numbers published by Underwriters Laboratories Inc.

## 5.10 SPECIAL SITUATIONS

There are several special situations that confront the specifier of joist and Joist Girder buildings. This section offers a few brief comments regarding each.

### Camber

The Steel Joist Institute Specification tabulates the approximate camber for K, LH and DLH joists and Joist Girders. Vulcraft does not typically camber joists where the top chord is pitched two inches or more per foot, because defections are minimal for such joists.

The Steel Joist Institute Specification tabulates the approximate camber for K, LH and DLH joists and Joist Girders. Camber is solely based on length and ranges from ¼ in. for a 20 foot joist, to 4 ¼ in. for a 100 foot joist. For joists over 100 feet, the camber is span/300. All parallel chord joists receive standard camber unless noted otherwise. Joists can be cambered to specified requirements, but this is expensive and should be avoided. In cases where it is desired to camber joists for dead load standard SJI camber will be used by Vulcraft. Joist can also be built with no camber when the circumstance warrants. Vulcraft does not typically camber joists where the top chord is pitched two inches or more per foot, because defections are minimal for such joists.

Erection and detailing problems can occur with LH, DLH joists because these joists have a significant amount of camber. For example, if the deck is to be connected to a shear wall at the end of the building and a joist is placed next to the end wall, then allowance must be made for the camber in the edge joist in order to connect the deck to the wall system. If proper details are not provided, the diaphragm may not be able to be connected and field adjustments may be required. In those cases where the edge joist is eliminated from the end wall, the deck can often be pushed down flat on an end wall support unless the camber is such that the bending in the deck would be so severe as to buckle the deck. If the joist has significant camber, it may be necessary to provide simple span pieces of deck between the wall and the first joist. This can present an additional problem since the sharp edge of the deck will stick up. The edge should be covered with a sheet metal cap to protect the roofing materials. Alternatively, the designer can specify a special camber for the end joists allowing for a gradual transition in elevation. Consideration must also be made for the anticipated live load deflection which may take place under a full design load condition possibly in the future. Keep in mind that special camber will increase the cost of the joist since it must be set up differently to achieve the special camber specified.

The specifying professional must pay attention to different length joists that are parallel to one another. These joists will have different standard cambers, which can cause detailing and deck installation problems.

### X-Bridging at Walls

It is good practice to eliminate X-bridging between the last joist and an end wall and substitute horizontal bridging at this location. This will allow the joist and the end wall to deflect independently. This situation occurs with hard wall systems where the joist deflects, and the end wall does not. In these situations, the X-bridging may tear out of the wall or its attachment to the wall may fail because the X-bridging will act like a vertical support and will attempt to carry the joist vertical load. If damage occurs, the bridging will no longer be effective.

### Cutting Bridging

One of the primary purposes of bridging is to stabilize the joists so that the joists can support construction equipment and workers prior to the placement of the decking. The bridging also serves to hold the joists in the position shown on the plans. In addition, as previously mentioned, the bridging also stabilizes the joists when standing seam roofs are used. Bottom chord bridging also braces the bottom chord for wind uplift and provides lateral bracing for the joist diagonals (in combination with the bottom chord). This function is often overlooked. The need for this bracing is obvious if one stops to consider that the compression diagonals within the joist are designed as individual columns with an effective length factor of 1.0 or less. As such, the ends of the columns must be braced out of plane. The bridging and inherent flexural stiffness of the bottom chord provide this bracing. Because of the required function of the bottom chord bridging, it cannot be cut or omitted after the deck is in place. If the bridging must be interrupted due to deck penetration, the bridging on either side of the penetration must be "tied off." Under most circumstances, X-bridging can be used on either side of the penetration to tie off the bridging.

Because the cost of placing bridging in the field is quite expensive, the designer should not over specify the number of rows of bridging unless it is necessary for some reason to do so.

A special case involving bridging is relative to bridging for composite joists. The need for bridging for Vulcraft's CJ-and Ecospan® E-Series floor joists was studied by Ziemian (2018). The results of this study indicated that the bridging can be removed from Vulcraft's CJ-and Ecospan® E-Series floor joists once the concrete has reached its 7-day compressive strength. Ziemian explains that, "with the shear connectors anchored by 7 day or older concrete and the steel deck, rotation of the top chord will be restrained. Attachment of the joist webs to this restrained top chord and catenary action within the bottom chord provides increased lateral stability for the joist, thereby safely permitting the removal of the joist bridging."

Ziemian also indicates in the white paper, "that bridging not be permanently removed from Vulcraft CJ and Ecospan E-Series joists with span / depth ratios less than 1.125 without additional full-scale testing and/or finite element modeling."

In rare cases where the aforementioned joists are subjected to forces which would cause the bottom chords to be subjected to compression the bridging cannot be removed.

### Joists Spanning Parallel with Standing Seam Roof Spans

On occasion it is necessary to provide a standing seam roof on a joist system in which the joists span in the same direction as the major corrugations of the standing seam roof. In these cases, a subpurlin system can be used to support the standing seam roof. The subpurlin is generally a light gage hat section spaced 5'-0" on center. Any economical joist spacing may be used, but the subpurlin system must be designed to span the distance between the joists. The reactions from the subpurlins and their locations on the joists must be specified by the specifying professional of record on the structural drawings to Vulcraft. If the panel points on the joists cannot be spaced so that the subpurlin reactions are applied at panel points, the top chord of the joists must be designed for the concentrated loads delivered by the subpurlins. The designer should be careful in using this system if UL or FM uplift requirements have been specified. The subpurlin system may not have been tested for uplift, thus guarantees cannot be made regarding the uplift performance of such systems.

### Tilted Joists

Joists are often supported in a manner such that the vertical axes of the joist are tilted with respect to the ground. This should be avoided if feasible due to several issues as discussed below. If a significant tilt exists, i.e. greater than 2 on 12, consideration should be given to the downhill component of loads. This load component can be resisted either by designing the roof diaphragm system to resist the loads or by designing a horizontal truss system to support the bridging loads. The specifying professional is responsible for the force resisting system.

### Folding Partitions

The specifier should be aware that the dust skirts on most folding partitions have a maximum deflection allowance of one inch. If the roof system deflects under the weight of snow or other live load more than one inch, the partition will become inoperable. The specifying professional must provide Vulcraft the deflection criteria and indicate the loading to be considered for the design of the joist supporting the partition.

Deflection restrictions can add substantial cost to the joists and Joist Girders that support the partition. The specifying professional should be aware that the partition installer will adjust the partition off the floor and should raise it to the proper height per installation instructions. Therefore, it may be possible to consider only a portion of the partition load in addition to the loads superimposed after the partition installation when specifying the special deflection criteria. The amount of partition loading, if any, to be considered in the special deflection criteria will depend on the partition model being used, the manufacturer’s installation instructions, and how the partition is supported from the structure.

Some partitions have optional dust skirts that allow for larger deflections, possibly as much as four inches. Use of dust skirts with higher deflection tolerances can result in significant savings in the joist and Joist Girder costs, due to the lower deflection restrictions.

The judgment of the specifier regarding what loads to consider, the effects of deflection on adjacent and supported components, is vital.

Also, some partitions require a maximum slope of ⅛ in. per 10 feet in order to operate. This must also be checked under required load conditions. If the building designer is not sure whether the deflection criterion can be met with steel joists, he should contact Vulcraft for assistance. In addition, because the folding door is a movable load, high shear and possibly chord bending will occur on the joist or Joist Girder when the weight of the entire partition is moved to one end of the supporting element. Therefore, not only should the deflection criterion be established, but also a loading diagram should be provided to Vulcraft so that the joists can be properly designed. This is another situation where the KCS joist can be specified.

### Seat Depth Changes

On occasion, specifying professionals may forget that there is a difference in seat heights between the various types of joists, i.e. K-Series and LH-Series, and elevation problems will occur at the member on which both are supported. The designer may provide a special raised portion on the support beam or Joist Girder to raise the K-Series seats so that the top of the chord is at the same elevation. However, it is usually less expensive to specify a 5-inch seat on one end of the K-Series joists. In addition, if special requirements require a four inch or some other unique seat height, this can be supplied by Vulcraft so long as the special seats are specified and the special seats are deeper than the standard, (e.g. a four-inch seat on a LH-Series joist should not be specified).

### Unequal Joist Reactions on Joist Girders or Beams

Proper design of details can reduce or even eliminate twisting forces on a Joist Girder or beam under the action of unequal joist end reactions applied from each side of the girder. When joists frame into the supporting member from one side only (such as with a typical perimeter condition) it is desirable to center the reaction point over the center of the support. This can be achieved by increasing the clear bearing length by increasing the seat depth. Specifying an extended end does not accomplish this because the reaction point is not moved. A good rule of thumb is to expect to gain one inch of clear bearing for each inch increase in bearing depth. Vulcraft should be aware of the intent so that they will provide this added clear bearing when detailing the joists.

A similar situation occurs when large joists bear on one side of the support and small joists bear on the other. Again, one can strive to get the reactions over the support centerline by increasing bearing depth and staggering or offsetting opposing joists (usually by 6 in.) so that each may extend beyond the centerline. If, for some reason, the joists cannot be staggered or the seats cannot be heightened, the induced torsion can be resisted by adding braces from the joist to the bottom chord of the Joist Girder (or bottom beam flanges) on the side of the larger joists only. The braces must be designed and specified on the structural drawings by the building designer.

In summary, roof systems with Joist Girders should be configured to eliminate the possible torsion in the Joist Girder. Roof systems with beams can be configured to eliminate the torsion, or the beams can be designed for the torsion and allowed to twist.

Since Vulcraft may not know the full intent, the designer must designate the offsets and increased seat depths on the structural drawings.

### Weld Size

The sizes of the chord members of joists and Joist Girders are not known by the specifier until the members are designed by Vulcraft. The designer may call for a 5/16 in. fillet weld on the edge of a Joist Girder chord and the Joist Girder chord supplied is only 3/16 in. thick, thus the weld cannot be made. The specifier should attempt to use 1/8 in. fillet welds whenever possible to eliminate this potential problem. The specifier can contact Vulcraft early in the design process if exact sizes need to be known so that special weld requirements can be determined. Vulcraft can then provide oversized members to meet specified weld requirements. If the specified welds are not actually required, this can add significantly to the cost of the project. This is an area where good communication between the specifying professional/detailer and supplier is important.

### Expandable Walls

A situation often occurs where joists are placed on a perimeter Joist Girder when the building must have expansion capabilities, i.e. a joist will be added to the same edge girder in the future. This problem is similar to the unequal reaction on girders, in that the joists should be placed over the centerline of the perimeter member. The future joists must then be offset from the existing joists. Again, Vulcraft must be informed of the designer’s intent so that the Joist Girder’s top chord can be designed for the eccentric loading of the future joists. As was also mentioned in the unequal reaction section, the bottom chords can be extended to eliminate the twist in the girder. The third option is to indicate that the tail of the seats on the joists can be cut off later to allow room for the new joists to be placed on the perimeter member. Vulcraft must also be aware of this situation so that the joist seat can be designed for present and future conditions.

### Expansion Joints

Several situations arise with respect to expansion joints. Obviously, bridging cannot be extended though an expansion joint. At expansion joint locations, it is recommended that a row of X-bridging be placed on each side of the expansion joint so that the forces which accumulate in the bottom bridging line can be transferred up to the top chord of the joists and into the roof or floor diaphragm, or into another bracing system that may be present at the top chord.

Another situation that arises is how to allow the joists to slide on the Joist Girders at an expansion joint. Some design professionals specify the use of Teflon pads placed on the joist seats to facilitate the sliding of these joists. Holes are often specified in the top chord of the Joist Girder so that bolts can be placed connecting the joists to the top chord and preventing the joists from sliding off the top chord of the Joist Girder. One side of the Joist Girder must be stabilized by firmly bolting or welding the joists to the top chord. In addition, uplift braces can be extended from the joists only on one side of the Joist Girder. In lieu of providing holes, a separate plate has been used to allow the expansion and contraction to occur. A detail of such a plate is shown in Figure 5.10.1.

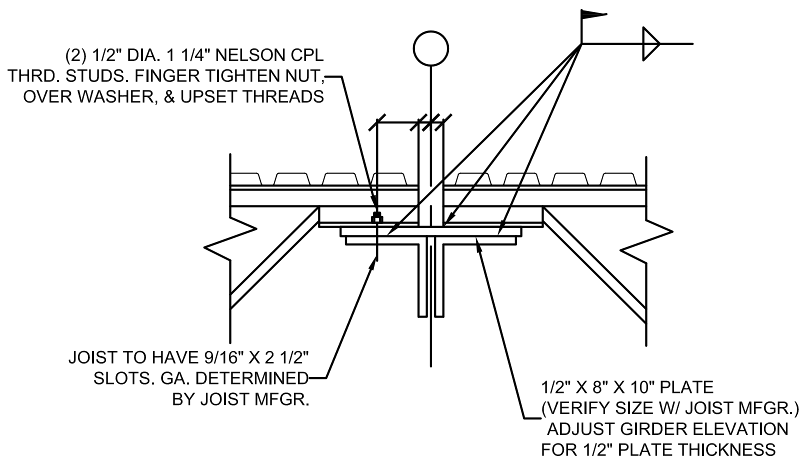



**Fig. 5.10.1 Expansion Joint**

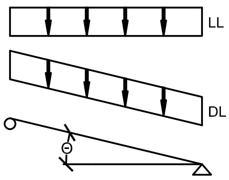
## Sloping Joists

Currently SJI Specifications do not address joists that are to be used at a slope rate greater than ½ inch per foot. Due to a lack of information, designers currently have no easy means for the proper selection of sloped joists. Some of the commonly encountered mistakes made with sloped joist designations include:

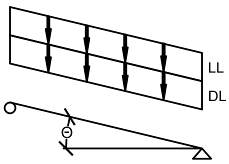
1. The use of horizontal projection as the span
2. The inconsistency in how loads are being applied to sloped joists
3. The effect of the load component parallel to the chords of the joists: This is not an issue when Vulcraft is designing the joist, since the load will be broken into components based on the slope of the joist.

Joists are specified by using their actual length and the load normal to the joist as the values that are used in the SJI load table. The dead and live loads for a roof system are typically oriented on two different axes.

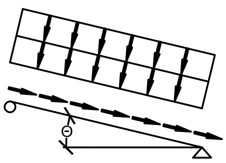
The live load is applied over the plan length of the member and the dead load is applied over the slope length. See Figure 5.10.2.




Loads applied by nature.




Loads as adjusted (LL x cosθ).




Loads adjusted and divided into components.

**Fig. 5.10.2 Sloping Joists**

To orient both loadings to the same axis, multiply the live load by the cos θ.

This method produces several benefits in that it:

1. Eliminates the need for additional load tables
2. Ensures the joist will be designed for the moment strength for which it was specified
3. Considers the actual joist length during selection, preventing over span conditions
4. Provides a standard procedure compatible with current SJI load tables

### Example 5.10.1 Sloping Joists

Determine the joist to be specified for the following:

**Given:**

Roof slope = 6:12
$LL$ = 14 psf
$DL$ = 22 psf
Plan dimension of bay:
$L_p$ = 39'-0"
Joist spacing = 5'-0"

**Solution:**

$\theta = \tan^{-1}(6/12) = 26.6°$
$LL\cos^2\theta$ (joist space) = 56 plf
$DL\cos\theta$ (joist space) = 99 plf
Actual joist length, $L_s$ = 43'-7"

The Steel Joist Institute Specifications for short span joists limits the length of joists to 24 times the joist depth. Therefore, the minimum joist depth for this situation is 22 inches.

Enter the Economical Joist Guide using: $TL$ = 155 plf and $LL$ = 56 psf and joist length = 43'-7".

Specify a 22K5 for the 44'-0" span:

Allowable uniform total load = 157 psf

Live load that produces deflection of L/360 = 76 plf

Alternatively, a load/load joists can be specified, i.e. 22K157/56.

In addition, Vulcraft will need to design this joist for the effects of the load parallel to the joist. This load would be:

$$[(LL\cos\theta) + DL]\sin\theta = 77 \text{ plf}$$

This load will be applied as an additional top chord axial force in the joist by Vulcraft.

## Splices

Long span joists are spliced when required for shipping or handling. Different states have specific laws about truck lengths of loads. However, in general, any joist over 100' will need to have field splice. Per SJI Technical Digest 9, "Handling and Erection of Steel Joists and Joist Girders"(SJI, 2008b), it is the erector's responsibility to "match mates." Stated in SJI Technical Digest 9, "joist mates will be marked '1A'and '1B' or 'A1' and 'A2' or some similar marking to indicate mates. Two dissimilar mates will not fit together properly and will result in wildly varying camber from joist to joist. To facilitate the erector's work in matching mates, Vulcraft identifies each spliced joist with a separate mark and each half with its own tag.

An example of a bolted field splice is shown in Figure 5.10.3

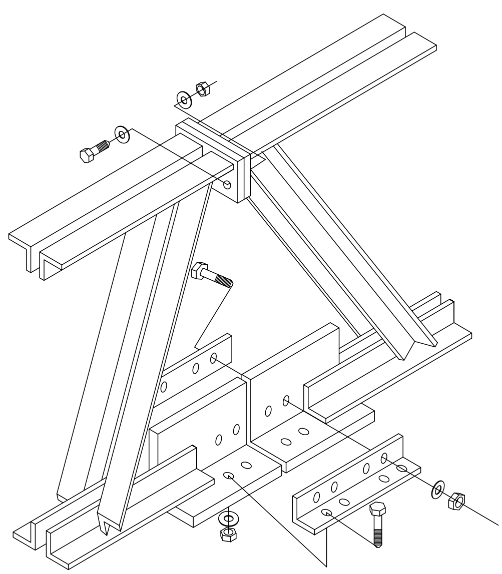


**Fig. 5.10.3 Field Splice**

# Chapter 6

# Specification of Components

## 6.1 INTRODUCTION

The purpose of this chapter is to discuss the proper specification for joists and Joist Girders for loads, deflections, bridging, geometric requirements and other criteria, and how to efficiently designate these requirements on the design drawings. The specification of loads for the design of joists and Joist Girders is the responsibility of the specifying professional. The loading must be based on building code requirements, building use requirements and the specifying professional's judgment. The extent of information required on the structural drawings is summarized in Chapter 8.

The specifying professional should consider the information required as consisting of two portions.

The first portion involves the specification of the required design criteria, including all loads, for the design the joists or Joist Girders.

The second portion relates to the design of joist and Joist Girder connection details. The connection details are usually indicated on the structural drawings. It is the responsibility of the specifying professional to indicate the type of attachment for each joist and Joist Girder. Connections relative to uplift and lateral forces must be carefully examined and detailed. As illustrated in Chapters 4 and 7, the details used to transfer axial forces into and out of joist or Joist Girder chords has a considerable impact on their design.

### Building Codes

The International Building Code (IBC) is the governing code for construction of buildings in the United States. The National Building Code of Canada is the governing code in Canada. These codes provide consistent policies and practices in the construction of buildings. Codes are updated and re-published periodically. It is important that the specifier indicate which edition of the code is used on a given project. Changes in the building codes may affect the design, load combinations, approval, use and the manufacture of open web steel joists.

The Steel Joist Institute (SJI) publishes a Specification and Code of Standard Practice, which defines how open web steel joist components and accessories are to be specified and designed. The SJI Specification is referenced in the IBC. In addition to the SJI Specification, the IBC defines policies and practices for the approval, submittals and certification of open web steel joist products. Vulcraft products meet or exceed the SJI Specification requirements.

Since Vulcraft products meet SJI Specifications, and the specifications are included in the IBC, these products are considered a "deferred submittal." In so doing, building officials do not need to approve these products prior to construction.

Vulcraft keeps current on SJI certifications, in order to eliminate the need for product inspections, and may include the International Accreditation Service (IAS, 2019) in the western US, and local jurisdictions, where required. Additional 3rd party special inspections can be performed in accordance with the IBC if done at the manufacturing facility.

### Loading

The IBC specifies standard load combinations, which are to be used for the design of structural members. The IBC requires that the contract documents include the method of design (ASD or LRFD) and the designated load combinations to be specified e.g. "Basic Load Combinations." If additional load combinations are to be checked, they must be specifically noted. The specifier must define the loads both uniform and concentrated as either: Dead, Dead Collateral, Live, Roof Live, Snow, Minimum Snow, Rain, Uplift, Wind or Seismic. When specifying the loads, it must be clearly noted whether the loads are nominal loads, ASD design loads or LRFD design loads. For example, ASD design wind load should be shown as 0.6W, and ASD design seismic loads should be shown as 0.7E. An LRFD design wind load should be shown as 1.0W, and an LRFD design seismic load should be shown as 1.0E. The specifying professional can always call out ASD or LRFD after a load to help clarify what type of load is being presented. This allows Vulcraft to properly configure the load combinations. For example, if a wind load is presented as strength level W load on the plans, for an ASD joist design Vulcraft will apply the 0.6 factor for the load combinations. The IBC also requires the $S_{ds}$ factor to be included in the contract documents. This is important for the joists and Joist Girders because the $S_{ds}$ factor impacts the load combinations with seismic loads.

### Deflection

Deflection limitations are serviceability requirements for the structure. Standard deflection limits for steel structures are listed in the IBC and in the SJI 100 specification. Deflection limits are defined for roof, floor and partitions. Typical roof live load deflection limits are L/180 or L/240; whereas, the typical floor live load deflection limit is L/360. L is taken as the span of the member. Total load deflection limits for steel structural members including open web steel joists and Joist Girders are not required per the IBC (see footnotes in the IBC deflection limit table) nor per SJI 100. Consequently, total load deflection limits should not be specified for open web steel joists or Joist Girders. In cases where a total load deflection limit is specified in accordance with the IBC the deflection limit will only be checked for live load.

Keep in mind that open web steel joists and Joist Girders are fabricated with built-in camber as a standard. The SJI 100 specification provides approximate cambers based on top chord lengths for the specifying professional's reference and consideration. The camber will exceed or offset a percentage of the dead load deflection. This additionally reinforces the reasoning why a total load deflection limit should not be specified.

For movable or stationary partitions, or for other situations where specific deflections are required, a load diagram showing the loads to be considered and the maximum deflection limit (preferably in inches) must be shown for each joist or Joist Girder. See Section 5.10 of this book for more information on **Folding Partitions**. In cases where there are sensitive serviceability requirements, a minimum Moment of Inertia should be specified rather than a deflection limit.

## 6.2 JOISTS SUBJECTED TO UNIFORM GRAVITY LOADS AND UPLIFT LOADS

### Joist Selection

For joists subjected to a uniform gravity load, the joist designation can be selected directly from the load tables in the Vulcraft Manual. To determine the load per lineal foot applied to the joist, the specifying professional multiplies the total load, including joist self-weight (psf), times the tributary width supported by the joist. The tributary width is one half the spacing to each adjacent support. The effects of deck continuity are neglected. A joist is then selected from the appropriate SJI load tables with sufficient capacity to resist the applied uniform load and to meet the project deflection criteria. The load tables contain the allowable total uniform load and the allowable live load that an individual joist can support for a given span. The red colored tabulated load indicates the load that causes a deflection of L/360. If a different deflection is acceptable or required, the deflection check may be made by ratio. For instance, if the live load deflection criterion is L/240, red numbers in the tables would be multiplied by 360/240.

Vulcraft's Manual contains an economical joist guide that is helpful to the designer in making a least cost joist selection. This table does not account for weight of bridging. Under certain conditions a heavier joist with less bridging, may result in less cost. The economical joist guide considers only downward uniform loads. For joists with additional loads (wind, seismic) a slightly deeper joist may be more economical depending on the type of load and its magnitude. The standard joist designation does not provide the breakdown of dead and live loads on a joist. For joists, with loads in addition to downward uniform loads, a breakdown of the uniform loads must be provided. As an alternate, the Load/Load Joist designation can be used (see the Load/Load Joist Designation section below).

In the load tables, the top chord of the joist is taken as continuously braced against out of plane buckling, typically by steel deck attachments. If this is not the case, the chord bracing which is available should be indicated on the drawings.

See Section 6.3 for discussions regarding KCS joist designation.

### Load/Load Joist Designation

Uniform load per foot designated joists (Load/Load designation) are an alternative to the standard SJI K, LH and DLH-Series joists. They are used when the specifying professional wants to optimize the joist cost. Using the Load/Load Designation provides the breakdown of loading for the required building code load combinations. This is particularly useful when wind and or seismic loads are required for the joist design. The designation takes the following form:

24 K 240/128

- Live Load (plf)
- Total Load (plf) (Dead + Live)
- K-Series SJI Specs to be Used in Design
- Depth (Inches)

30 LH 224/128

- Live Load (plf)
- Total Load (plf) (Dead + Live)
- LH-Series SJI Specs to be Used in Design
- Depth (Inches)

Vulcraft will design the joist for only the loads indicated. If additional capacity is required for future loading, the specifying professional may want to consider specifying the joist be designed for an Add-Load (see Concentrated Load section below). When Load/Load Designation is used the specifying professional must provide the deflection criteria on the construction documents. For example, in the joist notes on the structural plans the following could be specified, "Joists to be designed for a Live Load Deflection ≤ L/240."

The Load/Load Designation has an additional benefit for LH and DLH-Series joists. Additional joist depths can be specified when the Load/Load designation is used. These are depths in between the standard designation depths. It is preferred that the depth be in full inch increments. If a fractional depth is desired, contact Vulcraft for limitations. To illustrate, with standard designations, a 28LH06 or a 32LH06 can be specified with the Load/Load designation of 29LH, 30LH or 31LH. The additional depth options can be beneficial on projects where the specifying professional wants to maximize the joist depth based on the project limitations, like building clear height requirements.

The standard load tables provided in Vulcraft's manual, "Steel Joist & Joist Girder Systems" (Vulcraft, 2017c), can be used as a guide. The tables can be used to estimate the self-weight of the joists for the roof loading. The final weight of the joist may even be less than estimated by the table because Vulcraft will optimize the design for the exact loading provided.

### SJI Standard Weight Tables for LOAD/LOAD LH-Series Joists

The SJI Standard Weight Tables for Load/Load LH-Series joist Tables are used when the required load per lineal foot of joist exceeds the loads listed in the standard LH-Series SJI Tables. These higher loads are common for floor joists or roof joists with concrete or other heavy loads. The weight tables are a tool to assist the specifying professional in the preliminary design, and as an estimate for joists with high loading requirements. See the preceding section for the Load/Load designation format and additional information.

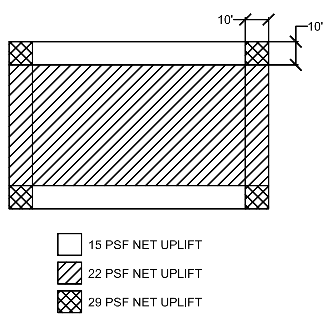
The specifying professional must verify the required joist seat depth with these heavy loads. Joist depths and loads to the left of the heavy black line use the standard LH joist seat depth of 5.0 inches. Joists to the right of the dark black lines in the tables require 7.5 in. deep bearing seats. If the table indicates that a seat deeper than 5.0 inches is required, the joist seat depth should either be called out, or specifically detailed on the construction documents.

### Joists with Wind Uplift

It is the responsibility of the specifying professional to accurately communicate the required "net" wind uplift. Uplift forces can cause stress reversal in the web members and chord forces. Since the bottom chord may be in compression and the steel deck is not present to prevent out of plane buckling, the bridging must be designed to provide this lateral support. The gross uplift load "W" on the joists is typically the Components and Cladding Wind Load per ASCE7, based on the tributary area of the joist. The "net" uplift is determined from the building code gross wind uplift loads minus the appropriate dead loads present. The "net" uplift is the result of the 0.6D+0.6W load combination for ASD and the 0.9D+1.0W load combination for LRFD, where W is the negative pressure for loads away from the roof surface. The plans should note if the "net" uplift is an ASD design load or an LRFD design load.

Care must be exercised when collateral loads are included in the design requirements for the project. Depending on what the collateral loads are, it may be appropriate to use only a small portion or none of the collateral load to resist the wind uplift loads. Therefore, the specifying professional should determine the "net" uplift loads. If the specifying professional needs to specify the gross uplift for the joists, it should be noted if the gross uplift is an LRFD or ASD design load. In addition, the specifying professional must note the dead load "D" that can be used to resist the wind uplift loads.

For small projects it is not uncommon to use the worst-case "net" uplift design load for all joists. This may be accomplished with a note on the drawings such as "Design and furnish joists and bridging for a net design wind uplift of 15 psf - ASD." For large projects it is common to provide a "net" design wind uplift load diagram, so the joists in the middle of the building do not need to be designed for the higher perimeter and corner wind loads. This is illustrated in Figure 6.2.1.




(Net Uplift is result of 0.6D+0.6W Basic Load Combination)

**Fig. 6.2.1 Joist Net Wind Uplift Load Diagram (ASD Design Loads)**

Refer to the SJI Specification Section 5.12 and the SJI "Code of Standard Practice," (SJI, 2017a) Section 2.10 for further information on wind uplift design. The Vulcraft Manual contains a table indicating approximate "Net Uplift Reaction (kips)" as a guidance as to whether the normal 2-½ inch bearing seat will accommodate the uplift.

Chapter 5 of SJI Technical Digest 6, "Structural Design of Steel Joists to Resist Uplift Loads," provides guidance on welded anchorage and bolted anchorage. It is the responsibility of the specifying professional to design the attachment of the joist and Joist Girder seats to the supporting structure. Details must be shown on the structural drawings so that Vulcraft can properly design the seat angles for the specified uplift.

The seat depth may often be determined by comparison with joists listed in the standard joist tables or by contacting Vulcraft.

## 6.3 JOISTS WITH CONCENTRATED LOADS

Specifying open web steel joists subjected to uniform loads was discussed in the previous section. When a joist must support concentrated loads, the specifying professional must furnish additional information on the construction documents so that the proper loads are included in the joist design. Items are discussed in this section that the specifying professional should know about concentrated loads and options for properly specifying those loads.

### Prevention of Torsion on Joist Chords

Concentrated loads must be connected in a manner not to cause torsion in the joist chords. In other words, the load must be applied to both chords without eccentricity on the two chord angles. The specifying professional should indicate this requirement in the construction documents. For example, a note indicating, "Manufacturers supplying framing clamp systems or suspension clamp systems for use with hanging loads shall design such systems in a manner not to cause torsion in the joist chords."

### 100 Pound Concentrated Loads

The following statement is included in the SJI 44th Edition Catalog (SJI, 2017b). "For nominal concentrated loads between panel points, which have been accounted for in the specified uniform design loads, a "strut" to transfer the load to a panel point on the opposite chord shall not be required, provided the sum of the concentrated loads within a chord panel does not exceed 100 pounds and the attachments are concentric to the chord."

### Concentrated Loads Greater Than 100 Pounds

Indicated in the SJI 44th Catalog in the section entitled CONCENTRATED LOADS AT JOIST CHORDS is the statement that "concentrated loads in excess of 100 pounds, or which do not meet the criteria outlined above, must be applied at joist panel points, or field strut members must be added as shown in the detail above." Note, the detail depicts the added strut positioned from the concentrated load to the nearest panel point on the opposite chord. An example of this detail is provided in Figure 6.3.4 later in this section.

When exact dimensional locations for concentrated loads are provided by the specifying professional and the magnitude and dimension are shown on the final Vulcraft joist layout drawing, Vulcraft will design the joist for the loads and load locations provided. If not, the joist erector, or the trade applying the load, will need to provide additional web members. Coordination with Vulcraft is encouraged when unclear.

### Add-Load and Bend-Check Loads

Add-Loads and Bend-Check loads are tools for dealing with concentrated loads. The SJI Code of Standard Practice provides definitions for both. An Add-Load is "a single vertical concentrated load that occurs at any one panel point along the joist chord. This load is in addition to any other gravity loads specified." A Bend-Check Load is "a vertical concentrated load used to design the joist chord for the additional bending stresses resulting from this load being applied at any location between the joist panel points. This load shall already be accounted for in the specified joist designation load, uniform load, or Add-Load, and is used only for the additional bending check in the chord and does not contribute to the overall axial forces within the joist." Bend-Check Loads can be specified for the top chord, the bottom chord or for both. The magnitude of the top chord and bottom chord Bend-Check do not have to be the same. Add-Loads and

Bend-Check Loads can be used in conjunction or separate, depending on the concentrated load type and design method used to specify the load. See section "Specifying Concentrated Loads for Joist Design" below for additional information (especially Option 3).

Add-Loads are a good way to build in extra capacity for joists. They allow a future load to be installed at any location along the joist, since the shear and moment envelope are covered by the Add-Load. If the future load does not occur at a panel point, field installed web members can be installed to transfer the existing panel point on the opposite chord. Using an Add-Load will also make evaluating a joist when the future load is installed easier than if additional uniform loads were added to the designation to provide additional capacity.

### Traveling Loads

For a traveling load with no specific location, specifying the traveling load as an Add-Load is often the best option. This allows Vulcraft to design for the worst-case for both the shear and bending moment. If the traveling load occurs between panel points, there are two common options. The first is for the construction documents to specify that field installed webs be installed at the final load location. The second is to require the joist to be designed for a Bend-Check load equal to the traveling load. This way the Bend-Check covers localized bending between panel points and the Add-Load covers global shear and bending on the joist.

In some cases, a traveling load will only be supported by the top chord of a joist or only by the bottom chord. The specifying professional in this case, could specify the specific chord (top or bottom) be designed for a traveling load at any panel point. If the load can occur between panel points, either field installed webs should be called out or a Bend-Check Load required on the specific chord (top or bottom). For example, if a joist is to have a 300 lb. High Volume Low Speed (HVLS) fan hung from the bottom chord of the joist, but the final location is not known and may be between panel points, the specifying professional could call out the joist to be designed for a 300 pound traveling load at any bottom chord panel point and a 300 lb. Bend-Check load on the bottom chord.

### KCS Joists

A versatile alternative to requiring special joists and selecting standard joists for resisting concentrated loads is the use of KCS joists. The KCS joist is an SJI standard design. It has a constant shear capacity and a constant moment capacity throughout its length. All KCS joist diagonals, except the end diagonals, are designed for 100 percent stress reversal. The end diagonals are designed for tension only (except when "net" uplift loading occurs), because stress reversal will never occur under gravity loading. The load tables for KCS joists list the shear and moment capacity of each KCS joist. The selection of a KCS joist is analogous to selecting a wide flange beam. The specifying professional calculates the maximum moment and shear imposed and selects the appropriate joist. If the concentrated load does not fall at a panel point, the designer must account for chord bending. KCS joists can also be designed for "net" wind uplift loading. The uplift loading is specified on the plans the same way it would be for any other joist.

An important limitation for which the specifying professional should be aware is that KCS joists cannot be designed for additional axial loads. The KCS joists do not define the loading breakdown, so the load combinations cannot be designed with additional axial loads being applied to the joist. An alternate, is to specify a Load/Load designated joist with an Add-Load to cover the special loads on the joist and the required axial load. That way Vulcraft can properly design the joist for all the required load combinations.

### Specifying Concentrated Loads for Joist Design

The SJI Code of Standard Practice, Section 2.4 – "Specifying Design Loads" suggests five options for the specifying professional to specify concentrated loads on open web steel joists. Using the options allows the estimator to price the joists and Vulcraft to design the joists in accordance with the latest SJI Standard Specifications. The most cost-effective option depends on the type of load, load magnitude, number of loads and if the location of the loads is known or unknown. The five options from the SJI Code of Standard Practice are paraphrased and modified to reflect best design procedures as follows:

Option 1: Select a joist designation from the Standard Load Table, or specify a joist using a load/load designation, which has been determined to be adequate for all design loads. The shear and moment envelope resulting from the selected uniform load shall meet the actual shear and moment requirements.

Option 2: Select a joist designation from the Standard Load Table, or specify a joist using a load/load designation for the uniform loads on the joist. In addition, provide the load and location of any additional loads on the structural plan with a note stating, "Joist manufacturer shall design joists for additional loads at locations shown."

Option 3: For additional point loads for which the exact locations are not known, for incidental loads or a combination of the two, select a joist designation from the Standard Load Table or specify a joist using a load/load designation. The loads can then be summed into a single Add-Load (or a combination of an Add-Load and Option 1 or 2). The following can be specified on the structural plan:

a) "**Design for a (__) lb. concentrated load located at any one panel point along the joist**" or "**Design for a (__) lb. Add-Load.**"

b) "**Design for additional bending stresses resulting from a (__) lb. concentrated load located at any location along (___) chord,**" or **"design (__) chord for a (__) lb. Bend-Check Load."**

   The *Bend-Check* and can be specified on the top chord, bottom chord or both top and bottom chords.

c) Both (a) and (b) above can be specified. The load in (b) can be the same or different depending on how the joist is loaded.

Option 4 Select a KCS joist whose moment and shear capacity are greater than the actual moment and end reaction. Only "net" wind uplift loading can be specified to be added to the KCS design. All other loads need to be considered when the KCS designation is chosen by the specifying professional.

Option 5 For complex loading conditions, a joist can be called out as a special joist (e.g. 28LH SP) and then a load diagram or load schedule can be provided on the structural drawings to communicate the requirements to the joist manufacturer. The diagram or schedule should include the magnitude of the load and the type of load (dead, live, wind, et cetera). In addition, it should be noted if loads are ASD or LRFD design loads. The SP designation can be specified on the plans with a note similar to the following "Joist manufacturer to design joist to support loads as shown in diagram." An example of two joist load diagrams is given in Figure 6.3.1.

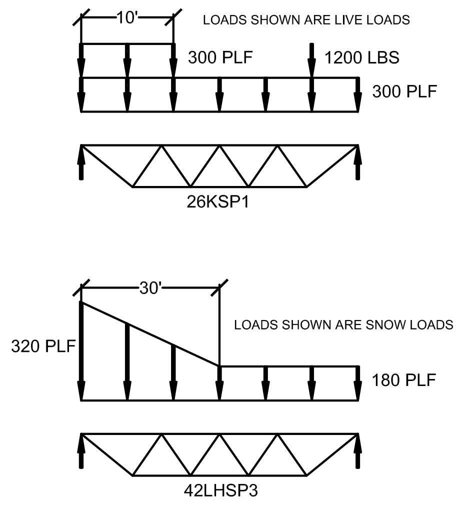



**Fig. 6.3.1 Joist Nominal Load Diagrams**

*Comments on the Specifying Options:*

Option 1: This option is the most labor intensive for the specifying professional. Specifying a standard joist to resist concentrated loads results in a joist that is generally less economical to manufacture than specifying a special joist. To select the appropriate joist the specifying professional must choose a joist that has enough shear and moment capacity to resist the loads. The specifying professional must calculate the moment and shears due to the combined loads on the joist. The equivalent uniform load can then be calculated from the maximum moment. The equivalent uniform load based on shear can be calculated from a shear diagram that completely covers the actual shear diagram. The larger of these two equivalent uniform loads can then be used to select a standard joist from the SJI tables. The point of zero shear on the joist should be determined. If this point is not relatively close (one foot) to the center of the joist, there may be diagonal members that are subject to a stress reversal. If a stress reversal occurs, a different specifying option should be used. If concentrated loads occur between the panel points, the uniform load may not cover localized stress and a Bend-Check load may be required as well. This option is illustrated in Example 6.3.1 later in this section.

Option 2: This option works well for a few added loads per joist with known magnitude and location. The dimension to the load must be indicated on the structural plans by the specifier or provided by others. For small roof top units, mechanical sub-contractors sometimes shift the unit along the span of the joist to avoid obstructions below the roof. For small roof top units, it may be better to use an Add-Load in Option 3. If a joist has multiple loads, some with known locations and some with unknown locations a combination of Option 2 and Option 3 can be used.

Option 3: This is a very versatile option. Using an Add-Load to cover concentrated loads simplifies the process. The location of the load does not have to be shown. If the load needs to shift due to field conditions, the moment and shear in the joist is covered and it might just require field installed webs to be added. The need for a Bend-Check in addition to the Add-Load will depend on how the joist is loaded. One good option is to specify the joist be designed for a small Bend-Check (i.e. 250 lb.) and then require field installed webs be added for larger loads. That way small loads are covered without any field modifications and the joist is not penalized for large point loads. This is a good option when a joist must be designed for axial loads in addition to vertical loads, since the load combinations can be determined using an Add-Load.

This option is an excellent way to handle loads from small roof top units. A note, like the following, can be added to the typical joist notes; "Each joist supporting mechanical units shown on the structural roof framing plans shall be designed for 2/3 of the **total** mechanical unit weight. Load from the mechanical unit shall be treated as an Add-Load and is in addition to the other Add-Loads in the joist schedule." This option can also be used in conjunction with Option 2.

Option 3 is illustrated in Example 6.3.2 later in this section.

Option 4: The use of KCS joists is another possibility. This option is used by some specifying professionals when multiple concentrated loads exist, or the exact locations of the loads is not known. This option does require additional calculations by the specifying professional. The maximum possible moment and the maximum possible shear for all the loading and all the applicable load cases (including downward wind loads when applicable) must be determined in order to specify the KCS joist. The only additional load that can be specified on the plans for a KCS joist is "net" wind uplift load.

It is important to note that KCS joists cannot be designed for axial loads. If the joist will have axial loads another option should be chosen.

Option 5: The use of load diagrams and load schedules is an excellent way of specifying loading in more complex situations. This is the preferred option when a joist supports partial length uniform loads or tapered loads. It is important that the specifying professional indicate the load types and specify ALL applied loads. That way the joist can be properly designed for all the applicable load combinations. Bend-Check loads can be used in conjunction with load diagrams/schedules. A common use of a joist load diagram is to convey the snow drift requirements.

Depending on the complexity of point loads, it may simplify things to use a schedule. The schedule can simplify the call outs on the framing plan and allow the loading types to be clearly shown. Roof screen loads and platform loads are two possibilities where a schedule would be useful. Table 6.3.1 below is an example.

| POINT LOAD SCHEDULE (1) | | | | |
|---|---|---|---|---|
| | Vertical Loads (kips) | | | |
| Load Number | Dead Load | Roof Live Load $L_r$ | Wind Load 1.0W | Seismic Load 1.0E |
| P1 | 0.25 | 0.40 | ± 5.2 | - |
| P2 | 0.1 | ± 0.35 | ± 1.2 | ± 0.5 |
| P3 | 0.15 | -0.45 | - | ± 0.3 |

(1) See Roof Framing Plan for point load locations.

**Table 6.3.1 – Point Load Schedule**

**Summary:**

1. Concentrated loads must be applied to joist chord concentrically
2. Specify whether the load is applied to the top chord or bottom chord
3. For a few added loads per joist with known magnitude and locations use Option 2 or Option 3
4. When exact locations of concentrated loads are variable, or not known, make use of the **Add-Load** and the **Bend-Check** procedures for optimum designs
5. Account for chord bending when concentrated loads cannot be located at panel points by either specifying field installed webs to be installed at the concentrated load, or by specifying a **Bend-Check** Load
6. Specify a joist seat depth that can support the end reaction and detail the surrounding joists to have compatible seat depths
7. Specified concentrated loads must be indicated as nominal or as D, $L_r$, 1.0W, 1.0E, etcetera

## Examples:

### Example 6.3.1 Joist with a Concentrated Load – Option 1

Select a joist using ASD to support the nominal loads provided below.

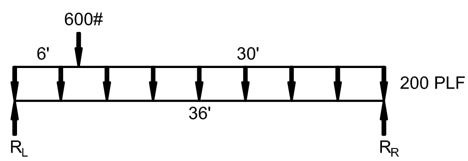



**Fig. 6.3.2 Example 6.3.1**

**Given:**

Joist span = 36 feet

Uniform dead load = 50 lbs/ft

Uniform live load = 150 lbs/ft

Concentrated dead load = 600 lbs located 6 feet from one end of the span

Maximum live load deflection = Span/240 = 1.80 in.

**Solution:**

1. Solve for reactions:

   $R_L$ = (600 lbs)(30 ft/36 ft)+ 200 lbs/ft)(36 ft/2) = 4,100 lbs

   $R_R$ = (600 lbs)(6 ft/36 ft) + (200 lbs/ft)(36 ft/2) = 3,700 lbs

2. Solve for the maximum moment:

   Zero shear is located at 3,700 lbs/200 lbs/ft = 18.5 ft from the right end.

   (Note location of point of zero shear. Possible shear reversal is insignificant.)

   $M_a$ = (18.5 ft)(3,700 lbs) - (200lbs/ft)(18.5 ft)$^2$/2

$M_a$ = 34,225 ft.-lbs.

3. Solve for the end shear that will completely cover the actual shear diagram (see Figure 6.3.3).

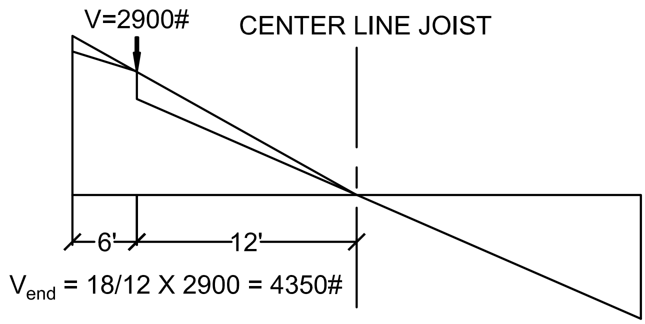



**Fig 6.3.3 Shear Diagram**

4. Solve for the equivalent uniform loads based on the maximum moment and joist end shear:

   For Shear:

   $w_{eq} = V_{end}$ /18 ft

   $w_{eq}$ = 4,100 lbs/18 ft = 228 lbs/ft

   For Moment:

   $w_{eq} = 8M_a/L$

   $w_{eq}$ = 8(34,225 ft-lbs)/(36 ft)$^2$ = 211 lbs/ft

   *Equivalent for shear controls.*

5. Select joist and check capacities:

   Centerline to Centerline Bearing = 36 ft – 0.33 ft = 35.67 feet

   Round span down to 35 ft for initial joist selection. Choose a 24K4 from the "Vulcraft Economical Joist Guide" Vulcraft (2017c).

   Round span up to 36 ft for allowable load check:

   Check total load requirement for 24K4:

   From the Standard Load Tables (Vulcraft 2017c) for span = 36 feet

   $w_a$ = 229 lbs/ft ≥ 228 lbs/ft - Joist is **o.k.** for total load. Check the live load deflection for 24K4:

   From the Standard Load Tables for span = 36 ft, the allowable live load for span/360 deflection (Red Figure) is 150 lbs/ft. The deflection requirement for this joist is span/240. As a result, the Red Figure can be ratioed to determine the allowable live load.

$w_{LL\ allow}$ = (360/240)(150 lbs/ft) = 225 lbs/ft $\geq$ 150 lbs/ft actual live load - Live Load deflection is **o.k.**

**Final Result: Specify a 24K4 for this member.**

Concentrated load reinforcement may require a field installed web at the concentrated load location. An example of such reinforcement is shown in Figure 6.3.4. This reinforcement must be applied to both sides of the joist.

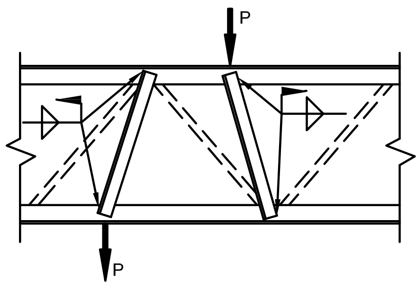



**Fig. 6.3.4 Concentrated Load Reinforcement**

### Example 6.3.2 Joist with a Concentrated Load – Option 3

Specify joist with Load/Load designation and Add-Load

Use same loading and criteria from Example 6.3.1 above.

1. Solve for total uniform load

   Total load = Dead + Live = 50 lbs/ft. + 150 lbs/ft. = 200 lbs/ft.

2. Determine joist depth

   Using the economical joist tables (as described in example 6.3.1) the 24K4 was the economical standard designation. For the Load/Load designation use same depth and joist series. Use 24K joist.

3. Assemble Load/Load Designation

   24K joist, total load = 200 lbs/ft., live load = 150 lbs/ft. so the designation is

   24K 200/150

4. Treat the concentrated load as an Add-Load

   Since there is only one load: Add-Load = 600 lbs. Because it is an Add-Load, location does not need to be specified and can vary.

5. Since this is a Load/Load designated joist, the live load deflection criteria of span/240 must be specified on the plans.

**Final Result: Specify 24K 200/150, Joist to be designed for 600 lbs. Add-Load. Maximum Live Load Deflection = span/240.**

As noted at the end of example 6.3.1, this Load/Load designated joist may require field installed webs at the concentrated load.

### Joist Seats

The end seat depth of the special joist must be compatible with the end seat depth of the surrounding joists. The seat depth is dependent on the chord size and the construction of the end diagonal. To verify that the seat depth of the special joist is compatible with the desired joist seat depth the actual moment and the maximum end shear imposed on the joist must be calculated. Then uniform loads that resulting from this moment and this end shear are calculated. If a standard K-Series joist of the desired depth can be selected to resist the greater of these uniform loads, Vulcraft will be able to supply the special joist with a 2.5 inch standard depth seat. If an LH or DLH joist is required to support the load, then the special joist can be supplied with a 5 inch or deeper seat. Examination of the KCS load table indicates that the shear capacity for a joist with a 2.5 inch end seat is limited to about 9.2 kips. If required because of the chord size, the special joists can be supplied with a deeper seat. However, this change in seat depth will affect the seat depths of the other joists and will have to be accounted for in the height of the support steel and possibly the determination of the eave height.

### Beams

In cases where large concentrated loads exist and their locations are not known, it may be best for the specifying professional to require a wide flange beam be used. If the load is due to crane or conveyor loading, the use of a beam will mitigate the problems associated with fatigue. Also, in some cases the load cannot be conveniently connected to the joists. In addition, the use of a wide flange beam can also solve difficult detailing problems. The use of beam framing around large openings can also facilitate the attachment of headers or stair framing.

If the specifying professional decides to use a beam in place of a joist, the beam should have an end seat designed with the same depth as the adjacent joist seats. A beam end can be reinforced for the 2.5 inch or 5 inch end seat as long as the beam web can transfer the shear through the shallow seat section. An example of the design of 2.5 inch end seat for a beam is presented in Section 5.2. Another option is to have an angle (with or without a gusset plate) connected to the end of the beam. This angle then bears on top of the Joist Girder. If a connection bearing on top of the Joist Girder is not possible, the beam can be detailed to attach to a vertical web member on the girder. The vertical member must be designed and installed by Vulcraft. If the vertical web connection is used, the specifying professional should be aware the connection needs to be at least 1.25 times the Joist Girder depth away from the end of the Joist Girder. This is required in order to properly configure the Joist Girder. If the beam occurs at a column, it is often easier to have the beam be supported directly by the column.

## 6.4 END MOMENTS AND AXIAL CHORD FORCES IN JOISTS

### Joist End Moments

When joists are used as part of a rigid frame the specifying professional must provide the joist end moments. This can be accomplished using a joist load diagram or a schedule of joist moments. Diagrams should be used for only simple cases as shown in Fig.6.9.1. More data can be shown in schedules including the magnitude and direction of the moments for the various load cases considered.

The bottom chords of joists are often connected to the column before some or all the dead load is present and before the live load occurs. Thus, the end moments from both dead and live load should be provided in a diagram or schedule. In addition, the specifying professional should require that the bottom chord bridging be designed and furnished.

Unless specifically instructed otherwise, Vulcraft's policy is to design the joist as a simple span member and then to check the chords and web members for the effects of the end moments. The use of a joist load diagram to specify end moments on joists is illustrated in Figure 6.4.1. See Section 6.5 for typical joist schedules.

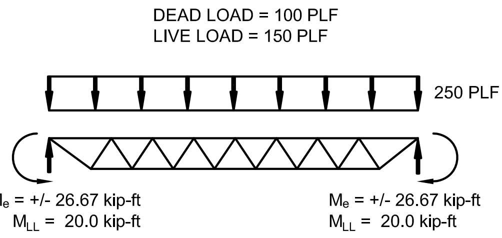



**Fig. 6.4.1 Joist Load Diagram**

## Axial Forces

There are multiple ways joists can be used where axial loads exist in the chords of a joist (top chord, bottom chord, or both). Examples include joists in a braced frame, joists in a moment frame, joists used as drag struts for lateral loads and joists used in the seismic wall anchorage system. These forces should be specified in a simple and clear way, either on the framing plan, on a load diagram or in a schedule. As mentioned above, the specifying professional should also require that bottom chord braces be designed and furnished by Vulcraft. Vulcraft will check the effect of the axial forces and adjust the chord design accordingly. An alternate procedure for determining the capacity of a joist chord to resist applied chord forces is illustrated in Example 4.2.1. If this method is used, the specifying professional should check all required load cases and load combinations when selecting the joist. To obtain a more economical joist it is often easier for the specifying professional to specify the joist for the vertical loads and to call out the additional axial load to be included in the design.

A special case of transferring axial forces through a joist exists when an axial force is transferred into the top chord of the joist and transferred out of the bottom chord. One example of this occurs when the diagonal of a braced frame is attached to the bottom chord of a joist and the roof bracing or diaphragm is in the horizontal plane of the top chord. The design and specification of joists for this condition was discussed in Chapter 4. An example load diagram is also provided in Figure 4.2.15. See Chapter 4 for additional discussions regarding axial loads and connections.

As mentioned in Section 6.3, the specifying professional must also verify if the joist requires a special depth end seat, due to top chord angle size. To do this determine the effective moment from the axial load. This can be done by multiplying the axial force times the effective joist depth to obtain the effective moment. Then add the effective moment to the moment from the vertical load for the total moment. This moment can then be compared to the limiting moments shown in the Tables 6.4.2 and 6.4.3 to determine if the standard seat will work or if a deeper seat is required.

The maximum vertical design load end reaction for the joist depths shown below is 9.2 kips (ASD) and 13.8 kips (LRFD) for 2.5-inch seats. For shallower joists, refer to the KCS tables for the maximum end reaction. Larger end reactions require deeper seats.

**K-SERIES JOISTS**

| Joist Depth (inches) | Limiting maximum joist moment with 2.5 inch deep seats (kip-ft.) |
|---|---|
| 18 | 81 |
| 20 | 91 |
| 22 | 100 |
| 24 | 110 |
| 26 | 120 |
| 28 | 130 |
| 30 | 140 |

**Table 6.4.2 - Limiting Maximum K-Series Joist Moments with 2.5 -inch Deep Seats**

**LH-SERIES JOISTS**

| Joist Depth (inches) | Limiting maximum joist moment with 5 inch deep seats (kip-ft.) |
|---|---|
| 24 | 273 |
| 28 | 323 |
| 32 | 373 |
| 36 | 423 |
| 40 | 473 |
| 44 | 523 |
| 48 | 573 |
| 52 | 623 |
| 56 | 674 |
| 60 | 724 |
| 64 | 773 |
| 68 | 824 |
| 72 | 874 |
| 78 | 949 |
| 84 | 1024 |
| 90 | 1099 |
| 96 | 1174 |

**Table 6.4.3 - Limiting Maximum LH Joist Moments with 5 -inch Deep Seats**

## 6.5 JOIST SCHEDULES

The use of a joist schedule is an efficient way to convey loading requirements. A schedule allows the loading to be consolidated in one place preventing the drawings from becoming cluttered. It is common to have the same joist designation used in more than one bay. With a schedule, the specifying professional can call out the mark number (e.g. J1) in appropriate bays in lieu of calling out the designation in each bay. By having the designation in a schedule, it decreases the chances of errors because the designation and any other required loading is noted once and not multiple times. The use of a schedule is recommended for joists with end moments as shown in Table 6.5.4.

The complexity of a schedule will vary by project. Schedules are adjustable to have as many or as few headings as required. Tables 6.5.1 thru 6.5.4 provide example schedules for joists. Tables 6.5.5 thru 6.5.10 provide additional information that could be used for a more complex schedule. With the load combinations in building codes becoming more complex, a schedule is a clear way to convey all the different types of loads. When the specifying professional breaks out all the different load types in a schedule, Vulcraft can design the joist for the appropriate load combinations. Table 6.5.5 illustrates a schedule with load types clearly designated.

It is important for the specifying professional to make sure the table clearly conveys the load information, the load type and whether the design will use ASD or LRFD load combinations. The specifying professional is encouraged to coordinate with their local Vulcraft office when developing a schedule.

Table 6.5.1 is an example of a simple schedule. The schedule has standard SJI designations and directly calls out the wind uplift loading. This type of schedule is appropriate for small rectangular buildings. For a small building it is often best to just use the worst-case uplift load for all the joist, in lieu of an uplift diagram.

| JOIST SCHEDULE (1)(3) | | | |
|---|---|---|---|
| Joist Mark Number | Designation | Net Wind Uplift load (plf) (2) | Comments |
| J1 | 24K6 | 100 | |
| J2 | 28LH05 | 150 | |

(1) See framing plan for additional loads to be included in joist design, including mechanical loads.
(2) Net Wind Uplift is the result of the 0.6D+0.6W load combination.
(3) Downward Wind Loads included in designations above.

**Table 6.5.1 - Joist Schedule**

Table 6.5.2 is a more complex schedule. This schedule takes advantage of using the Load/Load designations for the joists due to axial load requirements. The schedule calls for an Add-Load to be included in the design to allow for future loading. Since the joists use the Load/Load designation, the deflection criteria must be provided and is shown in the footnotes. The footnotes refer Vulcraft to a Net Wind Uplift Diagram for uplift loading which is common on large warehouse projects so that the large number of joists in the middle of the building do not have to be designed for the higher perimeter uplift loading.

| JOIST SCHEDULE (2)(3)(4) | | | | | |
|---|---|---|---|---|---|
| | | Axial Load (5) | | | |
| Joist Mark Number | Designation (1) (Total Load/ Live Load) | Wind Load 1.0W (kips) | Seismic Load 1.0E (kips) | Add-Load (kips) | Comments |
| J1 | 20K 288/160 | 10.5 | 23.0 | 1.0 | Office |
| J2 | 32LH 224/128 | 10.5 | 23.0 | 0.5 | |

(1) 26LH 245/135 (values noted at left are to define callout)
Joist Live Load (plf)
Joist Total Load (plf) (Dead + Live)
SJI joist type
Joist Depth

(2) Deflection Criteria: Live Load Deflection ≤ L/240 .
(3) See Net Wind Uplift Diagram for uplift loads on joists.
(4) See framing plan for additional loads to be included in joist design, including mechanical loads.
(5) Top chord axial load, Tension or Compression Load.

**Table 6.5.2 - Joist Schedule**

Table 6.5.3 is an example of a schedule used for projects where some of the joists are used as the chord members of the roof diaphragm. This type of schedule will likely have all the typical joists called out with standard SJI designations. The schedule notes the minimum standard SJI designation for the joist with axial load. It also provides the necessary uniform loads that must be used in the load combinations with the ASD wind axial load shown. This allows Vulcraft to design the joists for all the required loading.

| JOIST SCHEDULE | | | | | | |
|---|---|---|---|---|---|---|
| | | Loads for Combined Bending and Axial Check (2) | | | | |
| Joist Mark Number | Designation (1) | Wind Top Chord Axial Load 0.6W | Dead Load | Roof Live Load $L_r$ | Downward Wind load 0.6W | Net Wind Uplift load (3) |
| J1 | 30K7 | 20.0 kips | 44 plf | 63 plf | 32 plf | 150 plf |

(1) Standard designation is minimum requirement. Joist Manufacturer to modify joist design as required for combined loading requirements.
(2) Joist manufacturer to use these load in the applicable code load combinations to design the joist for combined bending and axial.
(3) Net Wind Uplift is the result of the 0.6D+0.6W load combination.

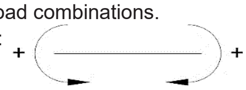
**Table 6.5.3 - Joist Schedule**

Table 6.5.4 is an example of a schedule used for a joist that is part of an Ordinary Truss Moment Frame. The schedule notes the joist depth and a LH-Series joists. Indicated is the uniform loading on the joist and the type of load. Also provided are the required axial loads and the end moments for which the joist is to be designed. The minimum moment of inertia is also listed to ensure that the joist is designed stiff enough to meet the requirements that were used for the frame analysis by the designer.

| JOIST SCHEDULE (1)(2) - PART 1 | | | | | | | |
|---|---|---|---|---|---|---|---|
| | | Uniform Loads | | | | Axial Load (4) | |
| Joist Mark Number | Joist Depth & Series | Dead Load (plf) | Roof Live Load $L_r$ (plf) | Downward Wind load 0.6W (plf) | Net Wind Uplift load (plf) (3) | Wind Axial Load 0.6W (kips) | Seismic Axial Load 0.7E (kips) |
| J1 | 24LH | 63 | 50 | 32 | 150 | 14.4 | 17.5 |

| JOIST SCHEDULE (1)(2) - PART 2 | | | | | | | |
|---|---|---|---|---|---|---|---|
| End Moments (6) | | | | | | Min. Moment of Inertia $I_J$ ($in^4$) | Addload (kips) (5) |
| Live Load Continuity Moment $L_r$ (kip-ft) | | Wind Moment 0.6W (kip-ft) | | Seismic Moment 0.7E (kip-ft) | | | |
| Left | Right | Left | Right | Left | Right | | |
| 6.7 | 6.7 | ± 22.0 | ∓ 22.0 | ± 27.5 | ∓ 27.5 | 216 | 0.5 |

(1) Joist manufacturer to use these loads in the applicable code load combinations.
(2) Deflection Criteria: Live Load Deflection ≤ L/240 .
(3) Net Wind Uplift is the result of the 0.6D+0.6W load combination.
(4) Top chord axial load, Tension or Compression Load.
(5) Addload is to be treated as a Dead Load "D" for load combinations.
(6) End Moment Sign Convention, Positive moments:




(Note: schedule was split into 2 parts just to fit onto page. Schedule on plans should be one table)

**Table 6.5.4 - Joist Schedule**

Some projects may have complex loading and design requirements. Tables 6.5.5a thru 6.5.10 provide additional columns that could be included in a schedule. The design professional could mix and match columns from the tables below to create a complex schedule that would meet the needs of the project. The tables are organized by load type, e.g. uniform loads, axial loads, end moments, etc. The design professional would only need to use the columns that apply to the specific project. The tables provided below are not an exhaustive list, but are intended to give the design professional a starting point for projects with complex loading and design requirements.

A uniform load schedule is provided in Tables 6.5.5a and 6.5.5b that could be included in schedules.

| JOIST UNIFORM LOADS (ROOF) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Joist Mark Number | Joist Depth & Series | Dead Load (plf) | Dead Load Collateral (plf) | Roof Live Load $L_r$ (plf) | Snow $S_{min}$ (plf) (2) | Snow $S_1$ (plf) (3) | Snow $S_2$ (plf) (3) | Rain R (plf) | Downward Wind load 1.0W (plf) | Net Wind Uplift load (plf) (1) |
| J1 | 24LH | 80 | 16 | 136 | 120 | 85 | 385 | 48 | 100 | 150 |

(1) Net Wind Uplift is the result of the 0.6D+0.6W load combination.
(2) $S_{min}$ is minimum uniform snow load for low sloped roof. This load is not to be combined with drift, sliding, unbalanced, or partial loads.
(3) See Diagram below for $S_1$ and $S_2$ loads, drift condition. These loads are not combined with $S_{min}$.

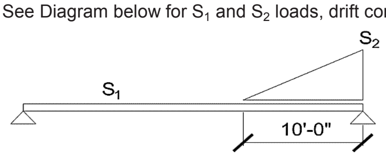



**Table 6.5.5a - Roof Joist Uniform Loads**

| JOIST UNIFORM LOADS (FLOOR) | | | | |
|---|---|---|---|---|
| Joist Mark Number | Joist Depth & Series | Dead Load (plf) | Dead Load Collateral (plf) | Floor Live Load L (plf) |
| FJ1 | 30LH | 210 | 60 | 600 |

**Table 6.5.5b - Floor Joist Uniform Loads**

Provided in Table 6.5.6a and 6.5.6b are axial loads that could be included in a schedule. The most common requirements will be Wind and Seismic Loads. Called out are the loads to the top chord of the joist. The specifying professional can also include columns for axial loads applied to the bottom chord of the joist, e.g. joists in braced frame. There may be projects that require the joist seat to be designed for a smaller axial load than the top chord of the joist. Additional columns could be added to the table for the seat (see Examples 4.8.1 and 4.8.2).

| JOIST AXIAL LOADS(1) (ROOF) | | | | | | |
|---|---|---|---|---|---|---|
| Joist Mark Number | Wind Load 1.0W (kips) | Seismic Load 1.0E (kips) | Dead Load (kips) | Roof Live Load $L_r$ (kips) | Snow S (kips) | Rain R (kips) |
| J1 | 8.5 | 26.0 | - | - | - | - |

(1) Top chord axial load, Tension or Compression Load.

**Table 6.5.6a - Roof Joist Axial Loads**

| JOIST AXIAL LOADS (1) (FLOOR) | | | | |
|---|---|---|---|---|
| Joist Mark Number | Wind Load 1.0W (kips) | Seismic Load 1.0E (kips) | Dead Load (kips) | Floor Live Load L (kips) |
| FJ1 | 4.3 | 13.0 | - | - |

(1) Top chord axial load, Tension or Compression Load.

**Table 6.5.6b - Floor Joist Axial Loads**

Illustrated in Table 6.5.7a and 6.5.7b are end moment categories that could be included in a schedule. End moments will be needed when a joist is used as part of an Ordinary Moment Frame.

| JOIST END MOMENTS (kip-ft) (1) - PART 1 (ROOF) | | | | | | |
|---|---|---|---|---|---|---|
| Joist Mark Number | Dead Load Moment D | | Roof Live Load Moment $L_r$ | | Snow Load Moment S | |
| | Left | Right | Left | Right | Left | Right |
| J1 | 10.0 | 10.0 | 6.7 | 6.7 | 8.3 | 8.3 |

| JOIST END MOMENTS (kip-ft) (1) - PART 2 (ROOF) | | | | | | |
|---|---|---|---|---|---|---|
| Joist Mark Number | Rain Load Moment R | | Wind Moment 1.0W | | Seismic Moment 1.0E | |
| | Left | Right | Left | Right | Left | Right |
| J1 | - | - | ± 22.0 | ± 22.0 | ± 27.5 | ± 27.5 |

(1) End Moment Sign Convention, Positive moments:

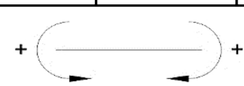



**Table 6.5.7a - Roof Joist End Moments**

| JOIST END MOMENTS (kip-ft) (1) (FLOOR) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Joist Mark Number | Dead Load Moment D | | Floor Live Load Moment L | | Wind Moment 1.0W | | Seismic Moment 1.0E | |
| | Left | Right | Left | Right | Left | Right | Left | Right |
| FJ1 | 10.0 | 10.0 | 20.0 | 20.0 | ± 25.0 | ± 25.0 | ± 32.5 | ± 32.5 |

(1) End Moment Sign Convention, Positive moments:


**Table 6.5.7b - Floor Joist End Moments**

Shown in Table 6.5.8a and 6.5.8b are Add-Load categories that can be included for a joist design. As previously discussed, Add-Loads are a way to provide extra capacity into a joist and to account for loads that do not have an exact location. Wall brace loads is an example of a possible use for wind and/or seismic Add-Loads. Seismic Add-Loads can also be used for the load from sprinkler main seismic sway braces.

| JOIST ADD-LOADS (ROOF) | | | | | | | |
|---|---|---|---|---|---|---|---|
| Joist Mark Number | Dead Load (kips) | Dead Load Collateral (kips) | Roof Live Load $L_r$ (kips) | Snow Load S (kips) | Rain Load R (kips) | Seismic Load 1.0E (kips) | Wind Load 1.0W (kips) |
| J1 | 0.5 | 0.75 | - | - | - | ±1.5 | - |

**Table 6.5.8a - Roof Joist Add-Loads**

| JOIST ADD-LOADS (FLOOR) | | | | | |
|---|---|---|---|---|---|
| Joist Mark Number | Dead Load (kips) | Dead Load Collateral (kips) | Floor Live Load L (kips) | Seismic Load 1.0E (kips) | Wind Load 1.0W (kips) |
| FJ1 | 0.4 | 0.3 | 0.75 | ± 2.0 | - |

**Table 6.5.8b – Floor Joist Add-Loads**

Shown in Table 6.5.9a and 6.5.9b are Bend-Check load categories. Loads from a solar system post are an example where including a wind Bend-Check load might be beneficial. This allows the post to be located anywhere along the joist top chord. As previously discussed, Bend-Check loads are only for localized stress in the top or bottom chord and do not contribute to global moment or shear on the joist.

| JOIST BEND-CHECK LOADS (1) - PART 1 (ROOF) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Joist Mark Number | Dead Load (kips) | | Dead Load Collateral (kips) | | Roof Live Load $L_r$ (kips) | | Snow Load S (kips) | |
| | TC | BC | TC | BC | TC | BC | TC | BC |
| J1 | - | - | 0.6 | 0.4 | 0.4 | - | - | - |

| JOIST BEND-CHECK LOADS (1) - PART 2 (ROOF) | | | | | | |
|---|---|---|---|---|---|---|
| Joist Mark Number | Rain Load R (kips) | | Seismic Load 1.0E (kips) | | Wind Load 1.0W (kips) | |
| | TC | BC | TC | BC | TC | BC |
| J1 | - | - | 0.6 | - | 0.9 | - |

(1) For Bend-Check Load: TC = Top Chord, BC = Bottom Chord

**Table 6.5.9a - Roof Joist Bend-Check loads**

| JOIST BEND-CHECK LOADS (1) - (FLOOR) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Joist Mark Number | Dead Load (kips) | | Dead Load Collateral (kips) | | Floor Live Load L (kips) | | Seismic Load 1.0E (kips) | | Wind Load 1.0W (kips) | |
| | TC | BC | TC | BC | TC | BC | TC | BC | TC | BC |
| FJ1 | - | - | 0.6 | 0.4 | - | - | 0.6 | - | - | - |

(1) For Bend-Check Load: TC = Top Chord, BC = Bottom Chord

**Table 6.5.9b - Floor Joist Bend-Check loads**

Additional columns are shown in Table 6.5.10 that could be added to a joist schedule. These columns can be used in a schedule for roof joists and for floor joists. Moment of Inertia requirements are often needed for joist that are part of a moment frame. They can also be required for joists on projects that have vibration design criteria or ponding requirements. Joists may need a minimum size, thickness or just width of the horizontal leg for a connection.

| JOIST ADDITIONAL REQUIREMENTS | | | | | | | |
|---|---|---|---|---|---|---|---|
| Joist Mark Number | Min. Moment of Inertia $I_{chord}$ (in$^4$) | Min. Top Chord Thickness (in) | Min. Top Chord Horiz. Leg (in) | Min. Bottom Chord Thickness (in) | Min. Bottom Chord Horiz. Leg (in) | Min. Seat Angle Thickness (in) | Additional Requirements* |
| J2 | 150 | 0.23 | 2 | 0.137 | 2 | 0.25 | Design Joist Webs to transfer Axial loads from Top Chord to Bottom Chord |

*Additional Requirements Column is for additional information the Engineer of Record wishes to convey to Vulcraft. The note shown is just one example of the information that can be provided in this column.

**Table 6.5.10 - Joist Additional Requirements**

## 6.6 SPECIAL PROFILE LH JOISTS

Special Profile LH joists are Non-Standard Configurations and require special design beyond the SJI Specifications. The common Non-Standard Configurations are shown in Figure 6.6.1.

- Bowstring
- Arch Chord
- Scissor
- Multi-Pitch
- Double Pitch
- Single Pitch

See Figure 6.6.1 for example profiles.

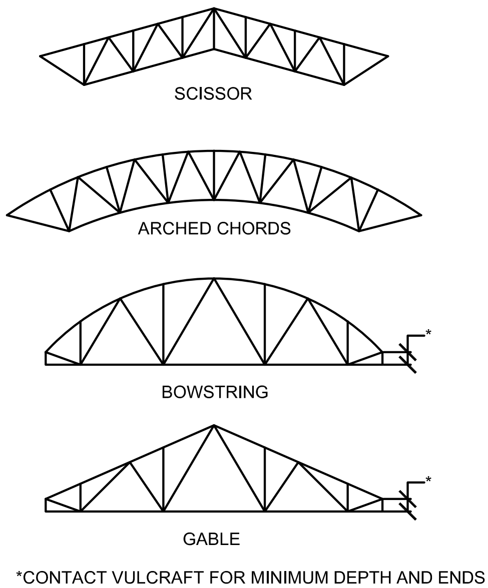



**Fig. 6.6.1 Special Profile Joists**

Each of these are depicted in the Vulcraft's Steel Joist & Joist Girder Systems Manual (Vulcraft, 2017c).

Note that special profile joists are only available for LH joists. K-Series joists do not have special configurations. The depth of the Special Profile LH joists can be deeper than 48 inches. If the specifying professional wishes to use deep special profile joists, e.g. 8 ft. deep, they can contact Vulcraft about shipping impact and about depth limitations. The shipping impacts and depth restrictions will vary depending on the jobsite location.

Special Profile LH joists generally require a deeper end bearing seat. The minimum seat depth will depend on the type of joist, the slope of the top chord, the need for top chord extensions and the type of supporting member. The Vulcraft Steel Joist & Joist Girder System Manual provides a table for required LH-Series seat depths for sloped conditions. These joists can also be designed as bottom chord bearing joists. For example, a double pitched joist can be designed as bottom chord bearing to create a gabled joist. The specifying professional should be aware that bottom chord bearing joists, both standard and special profile, require X bridging adjacent to the support to stabilize the joist.

Often a minimum end depth of 18 inches or more is needed. This minimum depth requirement is to allow Vulcraft to configure the webs of the joist at the end support. A rule of thumb for the minimum end depth of special profile joists in inches is the (span in feet)/4 with 18 inch minimum. For example, a 76-foot joist should have a minimum end depth equal to 76 ft./4 = 19 inches. The specifying professional should contact Vulcraft if end depths less than 18 inches are desired for a specific project to determine if a shallower depth is possible.

### Scissor and Arched Chord

Care must be taken in the specification and in the supporting structure design for scissors and arched chord joists. When these members deflect vertically under gravity loads, the end supports translate outward (horizontal deflection), as seen in Figure 6.6.2. Depending on the loading and the configuration of the joist, the horizontal deflection can be significant.

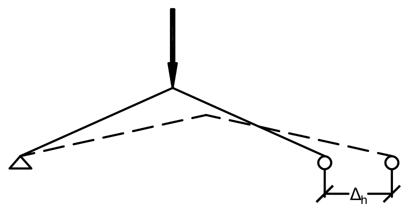


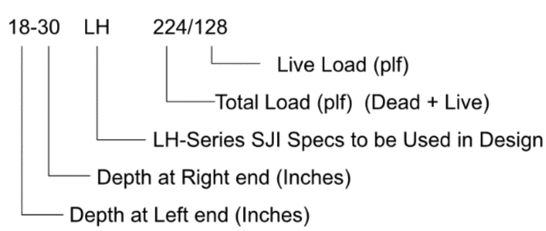

**Fig. 6.6.2 Loaded Scissor Joist**

Scissor joists are typically designed with one of two end connections, pinned/roller or pinned/pinned. The end reaction will not be truly pinned or truly a roller. Pinned/roller connections allow the end of the joist to slide, as illustrated in Figure 6.6.2. The supporting structure must accommodate the horizontal deflection with this type of connection. Pinned/pinned connections at the ends of the joist will restrain the horizontal deflection of the joist. This restraint imparts a horizontal thrust to the supporting structure. The supporting structure must be able to withstand the thrust loads with this type of connection. For projects with scissor or arched chord joists it is beneficial for the specifying professional to contact Vulcraft during the design phase for assistance in determining the horizontal reactions or the horizontal deflections for the proposed design. This allows the specifying professional to make an informed decision about what type of support connection to use and the implications to the support structure. If the specifying professional must limit the horizontal deflection, due to the supporting structure's needs, it may impact the joist design. The more stringent the horizontal deflection requirement the heavier the joist will have to be, in order to have a large enough moment of inertia to satisfy the criteria. If the horizontal deflection criteria are too stringent, it may not be possible to design a stiff enough joist to meet the criteria. Coordinating with Vulcraft early in the design phase can help avoid this issue.

The construction documents should clearly indicate the type of support used for scissor or arched chord joists whether pinned/roller or pinned/pinned. For the pinned/roller case, the specifying professional must indicate the horizontal deflection criteria on the plans. This is typically done by specifying the maximum horizontal deflection in inches.

### Pitched Joists

For single pitched joists, a simple way to specify the joist is to use a modification of the Load/Load designation. The modified designation indicates the depth at each end of the joist. This modified designation conveys the depth requirements. The following is an example of the single pitched Load/Load designation.




Similarly, for a double pitched joist, the depth at the left end, ridge, and right end can be included in the Load/Load designations. This designation is as follows

24-36-24 LH 288/160

- 160 — Live Load (plf)
- 288 — Total Load (plf) (Dead + Live)
- LH — LH-Series SJI Specs to be Used in Design
- 24 — Depth at Right end (Inches)
- 36 — Depth at Ridge (Inches)
- 24 — Depth at Left end (Inches)

The Special Products section of the Vulcraft website provides additional information on special configuration LH joists and examples of the information that should be provided on the construction documents for the different types of joists.

## 6.7 JOIST GIRDERS SUBJECTED TO GRAVITY LOADS

### Joist Girder Designation

For simple span Joist Girders subjected to equal uniformly spaced point loads, noting the Joist Girder designation on the plan provides an adequate specification for the member. The spacing of the joists should be called out on the plans, even if the joists are "equally spaced" between two given grid lines. An example of a standard Joist Girder designation is:

48 G 6N 10.6K

- 10.6K — Total Load at each Panel Point
- 6N — # of Joist Spaces Between Girder Supports
- G — SJI Joist Girder
- 48 — Depth (Inches)

There is a large range of depths that can be used for Joist Girders. Joist Girders can be designed and fabricated with depths in between those listed in the standard load tables. Depths should be specified in full inch increments, for example 49G, 50G, or 51G. For the number of joist spaces, partial spaces need to be counted as a space. This is because the number of panel point loads on the Joist Girder will be the number of spaces minus one. An example would be a girder with (6) 10 foot typical joist spaces and an end space of 5 feet; the 5 foot space would be treated as a space, so the designation would be 7N. The 10.6K indicates the magnitude of each panel point load in kips for an ASD design. The specifier should include the self-weight of the Joist Girder in the panel point load. Due to the load combinations, even those for ASD design, it is often necessary for the live load at each panel point to be provided by the specifying professional. This allows Vulcraft to design the Joist Girder for all the required load combinations especially when axial loads are present. It also allows the Joist Girder to be properly checked for live load deflection criteria. Adding the panel point live load to the designation will take the following format:

48 G 6N 10.6K/4.6K

- 10.6K/4.6K — Total Load/Live Load at each Panel Point
- 6N — # of Joist Spaces Between Girder Supports
- G — SJI Joist Girder
- 48 — Depth (Inches)

If the specifying professional wishes to have the Joist Girder designed using LRFD load combinations it is best to specify the Joist Girder in a schedule. The schedule should have columns for girder depth and the number of spaces. The panel point dead load and the panel point live load should be specified separately. Provide a column for Panel Point Dead Load and another column for Panel Point Live Load. Showing the dead load and live load is especially important if the Joist Girder must be designed for any wind or seismic loads. Note in the schedule that Vulcraft is to design Joist Girders using LRFD based on the nominal design loads in the schedule. Table 4.8.3 in Example 4.8.2 provides an illustration of such a Joist Girder schedule.

Vulcraft will use the most economical web configuration based on the depth of the Joist Girder and the spacing of the joists. Indicated in Figure 6.7.1 are the usual configurations of the Joist Girder web diagonals, as produced by Vulcraft. The D/S ratios shown indicate geometrical configurations for the girders. D and S are in inch units.

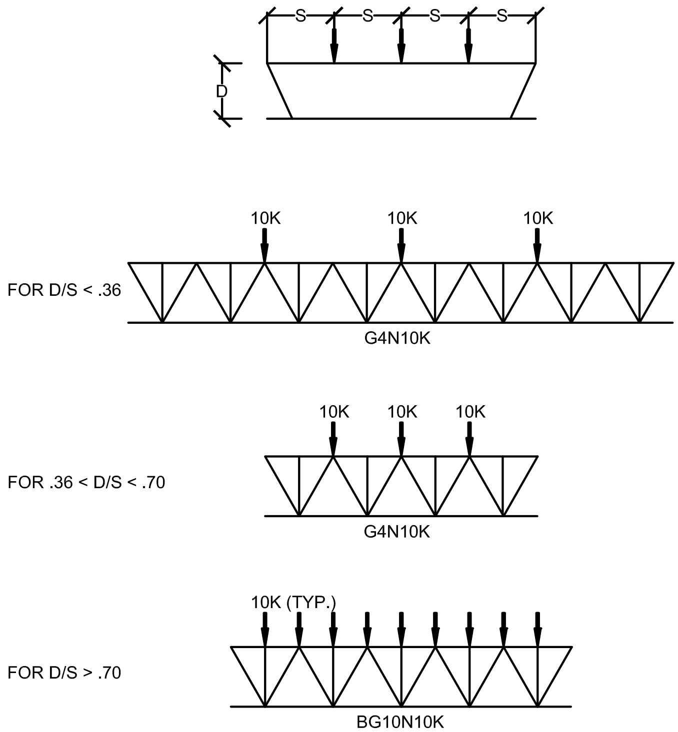



**Fig. 6.7.1 Joist Girder Web Configurations**

Vulcraft also offers a VG Joist Girder. The VG type has the largest number of unobstructed openings in the girder web, because the joists align with the web verticals and do not block the open panels formed by the bottom chord and the adjacent webs. If this feature is desirable, girders should be specified with the VG designation. An example of this designation is 32VG8N10K. This is illustrated in Figure 6.7.2. The VG type is slightly more expensive than a G type. If the specifying professional wants to have items pass through the Joist Girder, for instance sprinkler mains or ducts, they can specify on the structural plans what needs to go through the Joist Girders, i.e. the size and the location. Vulcraft will determine the best web configuration based on the requirements.

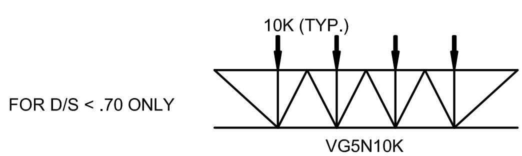



**Fig. 6.7.2 VG5N10K Joist Girder**

If the spacing and magnitude of loading varies, the design requirements for the Joist Girder must be clearly communicated. This can be done in a variety of ways. For simple cases, the additional loads can be specified directly on the structural plan. A note like the following should be provided on the plans, "Loads shown on the plans are in addition to typical loads in the Joist

Girder designation." An example of this would be a girder with the typical joists spaced 8 foot on center and one 10 foot joist space at the end of the Joist Girder. There would be an additional point load on the girder at the first panel point. This additional load could be called out on the framing plan at that panel point. For complex cases, the specifying professional should use a load diagram to illustrate the loading applied to the Joist Girder. Loading applied to the bottom chord of the member could also be indicated on the load diagram. An illustration of a Joist Girder subjected to an uneven load distribution is illustrated in Figure 6.7.3. Vulcraft will determine the optimum web configuration for the Joist Girder.

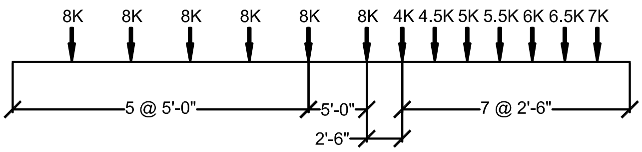



**Fig. 6.7.3 Joist Girder Load Diagram**

Bottom chord braces should be specified to be designed and furnished by Vulcraft. This is true for all Joist Girders, whether they have gravity loads, uplift loads, end moments, axial loads, etc.

### Add-Load and Bend-Check Loads

Add-Loads and Bend-Check loads can also be specified for Joist Girders. Add-Loads are a common way to have the Joist Girder designed for additional load to allow for future mechanical equipment or other loads. They can also be used to cover traveling loads. See the Add-Load and Bend-Check load discussions for joists in Section 6.3 for additional information. The information applies to both joists and Joist Girders.

## 6.8 JOIST GIRDERS SUBJECTED TO UPLIFT LOADING

Like joist, Joist Girders in roof systems will be subjected to net uplift loads if the code-imposed wind uplift exceeds the allowable permanent dead load. See Section 6.2 for additional discussion on "net" uplift. Net uplift loading will affect the design of the Joist Girders and the amount of uplift bracing. Under gravity loads, the top chord of the Joist Girder is in compression and the bottom chord is in tension. If a net uplift loading occurs, the bottom chord of the joist will be in compression.

Due to this load reversal in the chords, the bracing design must always be adjusted to account for the uplift condition. Net uplift also causes a stress reversal in the diagonals. This condition must also be checked by Vulcraft.

Since the Joist Girders often support large roof areas it may be allowed by the code to design the Joist Girders for the Main Force Wind Resisting System (MFWRS) loads in lieu of the Component and Cladding loads. The MFWRS uplift loads will be less than those for components and cladding. The use of the MFWRS uplift loads for the Joist Girders will reduce the impacts of uplift loading. See ASCE 7 for Tributary Area requirements and any other restrictions for using the MFWRS uplift loads.

It is the responsibility of the specifying professional to communicate the required "net" wind uplift loading for the Joist Girders. This may be accomplished the same way the joist net uplift loads are communicated, e.g. a note or diagram. An example of a Joist Girder note on the drawings is "Design and furnish Joist Girders for a net wind uplift of 11 psf - ASD." Illustrated in Figure 6.2.1, in the joist uplift discussions in Section 6.2, is a net wind uplift diagram. If the Joist Girders are designed for the MFWRS uplift loads, it may be necessary to provide separate diagrams for the joists and the Joist Girders. This is because the zones and distances are different for Component and Cladding loads than they are for MFWRS loads.

## 6.9 END MOMENTS AND AXIAL CHORD FORCES IN JOIST GIRDERS

### Girder End Moments

Vulcraft's design procedure for Joist Girders subjected to end moments is analogous to the design procedure for joists subjected to end moments. Unless specifically instructed otherwise, Vulcraft's policy is to design Joist Girders as simple span members and then to check the chords and web members for the effects of the end moments, unless specifically instructed otherwise.

When Joists Girders are used as part of a rigid frame the specifying professional must provide the Joist Girder end moments. This can be accomplished using a load diagram or a schedule. Diagrams should be used for only simple cases as shown in Fig.6.9.1 More data can be shown in schedules including the magnitude and direction of the moments for the various load cases considered.

The bottom chords of these Joist Girders are often connected to the column before some or all the dead load is present and before the live load occurs; thus, the end moments from both dead, collateral, live, seismic, wind and snow loads can be provided. See Section 6.10 for examples of Joist Girder schedules.

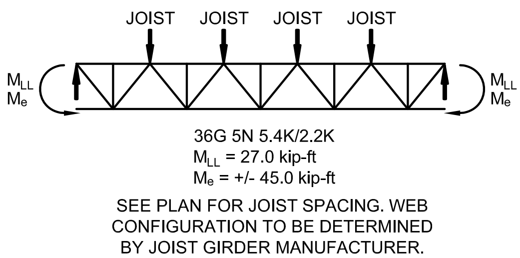



**Fig 6.9.1 Girder Load Diagram**

### Axial Forces

There are multiple ways that Joist Girders can be used in a building that would impart axial load in the chord of a Joist Girder (top chord, bottom chord or both). Some examples include: Joist Girders used in a braced frame, Joist Girders used as drag struts for lateral loads, Joist Girders used as diaphragm chord members and Joist Girders used in the seismic wall anchorage system. Just like with joists, these forces should be specified either on a load diagram or in a schedule. The type of axial load will need to be noted as well so that Vulcraft can combine the axial load in the appropriate load combinations. If the axial load is applied to the top chord of the Joist Girder and needs to be transferred down to the bottom chord, like for a braced frame, this must be noted on the structural plans. It should be noted in the Joist Girder schedule or in a diagram as shown in Figure 4.2.15. See Chapter 4 for additional discussions regarding axial loads and connections. Section 6.10 contains examples of Joist Girder schedules, including axial loads.

## 6.10 GIRDER SCHEDULES

The use of a schedule is an efficient way to convey loading requirements for Joist Girders. A Joist Girder schedule allows most or all the loading to be consolidated in one place. It is recommended that the specifying professional use a separate schedule for joists and for Joist Girders. This is because the designations are different between the two and the units are often different for the loads applied to a joist versus a Joist Girder. By having separate schedules, it is easier to have the schedules present the loading in a concise manner. The other advantages of a schedule discussed in Section 6.5 for joists also apply to Joist Girders. The complexity of a schedule will vary by project. The schedule can be adjusted to have as many or as few headings as required. Tables 6.10.1 thru 6.10.3 provide examples of schedules for Joist Girders. With the load combinations in codes becoming more complex a schedule is a clear way to convey all the different types of loads. When the specifying professional clarifies all the different load types in a schedule Vulcraft can refine the design and design the Joist Girder for all the appropriate load combinations. Illustrated in Table 6.10.3 is a schedule with load types clarified.

It is important for the specifying professional to provide a schedule that clearly conveys the load information. The following statement is contained in Section 2.4 of the SJI Code of Standard Practice (COSP) (SJI, 2017a), "The specifying professional shall provide the nominal loads and load combinations as stipulated by the applicable code under which the structure is designed and shall provide the design bases (ASD or LRFD)." The specifying professional is encouraged to coordinate with Vulcraft when developing schedules different than those presented below.

Table 6.10.1 is an example of a schedule for Joist Girders. The designation includes the live load portion of the panel point loads so the load combinations with wind and seismic loads can be designed and so the Joist Girder can be checked for the required deflection criteria. An Add-Load has also been specified to provide additional capacity for items like mechanical units. The footnotes refer to a diagram for the uplift loads. This is common, especially if the girders have enough tributary area to be able to use the Main Force Wind Resisting System uplift loads.

**JOIST GIRDER SCHEDULE (1)(2)(3)(4)(5)**

| Girder Mark Number | Designation (Total Load/ Live Load) | Axial Load (6) | | Add-Load (kips) | Comments |
|---|---|---|---|---|---|
| | | Seismic Load 1.0E (kips) | Wind Load 1.0W (kips) | | |
| G1 | 56G 7N 12.5K/5.8K | 160 | 85 | 2.0 | |
| G2 | 56G 7N 14.4K/5.8K | 160 | 85 | 4.0 | Office |

(1) Manufacturer to design Joist Girders using ASD. Nominal design loads shown are to be used in the applicable ASD code load combinations.
(2) Deflection Criteria: Live Load Deflection ≤ L/240 .
(3) See Net Wind Uplift Diagram for uplift loads on girders.
(4) See framing plan for additional loads to be included in Joist Girder design, including mechanical loads.
(5) See framing plan for joist spacing along girder.
(6) Top chord axial load, Tension or Compression Load.

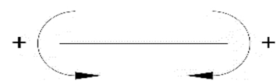
**Table 6.10.1 - Joist Girder Schedule**

Table 6.10.2 is an example of a schedule used on a project where some of the Joist Girders are used in Ordinary Truss Moment Frames. Noted in the schedule are the Joist Girder depth, number of spaces and panel point loads, including live load. Also noted are the required axial loads and the end moments. The minimum Moment of Inertia is also listed so that the Joist Girder will be designed with enough stiffness to meet the requirement used in the frame analysis and frame deflection as specified.

| JOIST GIRDER SCHEDULE (1)(2)(3)(4)(5) - PART 1 | | | | | |
|---|---|---|---|---|---|
| | | Axial Load (6) | | | |
| Girder Mark Number | Designation (Total Load/ Live Load) | Wind Axial Load 1.0W (kips) | Seismic Axial Load 1.0E (kips) | Add-Load (kips) (7) | Min. Moment of Inertia $I_G$ ($in^4$) |
| G1 | 40G 5N 13.0K/7.0K | - | - | 4.0 | - |
| G2 | 36G 5N 6.5K/3.5K | 18.0 | 20.0 | 2.0 | 1,128 |

| JOIST GIRDER SCHEDULE (1)(2)(3)(4)(5) - PART 2 | | | | | | | |
|---|---|---|---|---|---|---|---|
| | End Moments (8) | | | | | | |
| Girder Mark Number | Live Load Continuity Moment (kip-ft) | | Wind Moment 1.0W (kip-ft) | | Seismic Moment 1.0E (kip-ft) | | |
| | Left | Right | Left | Right | Left | Right | Comments |
| G1 | - | - | - | - | - | - | |
| G2 | 75.0 | 95.0 | ± 55.0 | ± 60.0 | ± 62.0 | ± 67.0 | Frame Girder |

(1) Manufacturer to design Joist Girders using ASD. Nominal design loads shown are to be used in the applicable ASD code load combinations.
(2) Deflection Criteria: Live Load Deflection ≤ L/240 .
(3) See Net Wind Uplift Diagram for uplift loads on girders.
(4) See framing plan for additional loads to be included in Joist Girder design, including mechanical loads.
(5) See framing plan for joist spacing along girder.
(6) Top chord axial load, Tension or Compression Load.
(7) Add-Load is to be treated as a Dead Load "D" for load combinations.
(8) End Moment Sign Convention, Positive moments:




(Note: schedule was split into 2 parts just to fit onto page. Schedule on plans should be one table)

### Table 6.10.2 - Joist Girder Schedule

Some projects have complex loading and design requirements for Joist Girders. In Tables 6.10.3a thru 6.10.9 columns are provided that could be included in schedules. The design professional can mix and match columns from the tables to create a complex schedule that would meet the needs of the project. The tables are organized by load type, e.g. panel point loads, axial loads, end moments, etc. The design professional only needs to use the columns that apply to the specific project. The tables provided are not an exhaustive list but are intended to give the design professional a starting point for projects with complex loading and design requirements.

For complex projects, having the total load and live load at each panel point in the Joist Girder designation may not be adequate. Shown in Table 6.10.3a and 6.10.3b are columns that could be used to show various loads at the panel points. It will be rare that all the columns are needed on a project. For instance, most projects will only have Roof Live Load or Floor Live Load, not both.

| Girder Mark Number | JOIST GIRDER PANEL POINT LOADS (1) (ROOF) | | | | | |
|---|---|---|---|---|---|---|
| | Girder Depth & Number Spaces (2) | Dead Load (kips) | Dead Load Colateral (kips) | Roof Live Load $L_r$ (kips) | Snow Load $S_{min}$ (kips) (3) | Rain Load R (kips) |
| G1 | 40G 5N | 6 | 1.0 | 7.0 | 7.6 | - |

(1) Joist Girder manufacturer to use these load in the applicable code load combinations.
(2) See framing plan for spacing of joists along girders.
(3) $S_{min}$ is minimum uniform snow load for low sloped roof. This load is not to be combined with drift, sliding, unbalanced, or partial loads.

**Table 6.10.3a - Roof Joist Girder Panel Point Loads**

| Girder Mark Number | JOIST GIRDER PANEL POINT LOADS(1) (FLOOR) | | | |
|---|---|---|---|---|
| | Girder Depth & Number Spaces (2) | Dead Load (kips) | Dead Load Colateral (kips) | Floor Live Load L (kips) |
| FG1 | 38G 5N | 8.4 | 2.4 | 24.0 |

(1) Joist Girder manufacturer to use these load in the applicable code load combinations.
(2) See framing plan for spacing of joists along girders.

**Table 6.10.3b - Floor Joist Girder Panel Point Loads**

For snow and wind loads, the design professional may wish to simplify the loading for the Joist Girder by specifying the loads as uniform loads. In Table 6.10.4 columns are provided that could be included in a schedule for this situation. If this method of calling out the snow load is utilized, the design professional would not want to have a Snow Panel Point Load column in the schedule.

| JOIST GIRDER UNIFORM LOADS (ROOF) | | | | | |
|---|---|---|---|---|---|
| Girder Mark Number | Snow $S_{min}$ (plf) (2) | Snow $S_1$ (plf) (3) | Snow $S_2$ (plf) (3) | Downward Wind load 1.0W (plf) | Net Wind Uplift load (plf) (1) |
| G1 | 750 | 532 | 2410 | 375 | 940 |

(1) Net Wind Uplift is the result of the 0.6D+0.6W load combination.
(2) $S_{min}$ is minimum uniform snow load for low sloped roof. This load is not to be combined with drift, sliding, unbalanced, or partial loads.
(3) See Diagram below for $S_1$ and $S_2$ loads, drift condition. These loads are not combined with $S_{min}$.

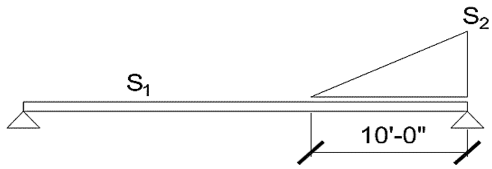



**Table 6.10.4 - Joist Girder Uniform Loads**

Illustrated in Table 6.10.5a and 6.10.5b are axial loads that could be included. The most common requirements will be wind and seismic loads. This table calls out the loads to the top chord of the Joist Girder, which is the most common case. The specifying professional can also include columns for axial loads to the bottom chord of the Joist Girder, e.g. a Joist Girder in a braced frame. There may be occasional cases where a specialty axial load is required. Another column could be added to the schedule and a note for how the specialty load is to be applied to the Joist Girder. An example is the axial load due to the tie force from a Factory Mutual Tied Maximum Foreseeable Loss (MFL) wall or a High Challenge Fire wall. A footnote is needed to specify that only the tie force must be combined with Dead load and Live Load and is not to be combined with seismic loads. There also may be projects that require the Joist Girder Seat to be designed for a smaller axial load than the top chord of the Joist Girder. Additional columns could be added to the table for the seat, see Examples 4.8.1 and 4.8.2.

| JOIST GIRDER AXIAL LOADS (1) (ROOF) | | | | | | |
|---|---|---|---|---|---|---|
| Girder Mark Number | Wind Load 1.0W (kips) | Seismic Load 1.0E (kips) | Dead Load (kips) | Roof Live Load $L_r$ (kips) | Snow Load S (kips) | Rain Load R (kips) |
| G1 | 40 | 250 | - | - | - | - |

(1) Top chord axial load, Tension or Compression Load.

**Table 6.10.5a - Roof Joist Girder Axial Loads**

| JOIST GIRDER AXIAL LOADS (1) (FLOOR) | | | | |
|---|---|---|---|---|
| Girder Mark Number | Wind Load 1.0W (kips) | Seismic Load 1.0E (kips) | Dead Load (kips) | Floor Live Load L (kips) |
| FG1 | 20 | 150 | - | - |

(1) Top chord axial load, Tension or Compression Load.

**Table 6.10.5b - Floor Joist Girder Axial Loads**

Provided in Table 6.10.6a and 6.10.6b are end moment categories that could be included in a schedule. End moments will be needed when a Joist Girder is used as part of an Ordinary Moment Frame.

| JOIST GIRDER END MOMENTS (kip-ft) (1) - PART 1 (ROOF) | | | | | | |
|---|---|---|---|---|---|---|
| Girder Mark Number | Dead Load Moment D | | Roof Live Load Moment $L_r$ | | Snow Load Moment S | |
| | Left | Right | Left | Right | Left | Right |
| G1 | 34.0 | 34.0 | 30.7 | 30.7 | - | - |

| JOIST GIRDER END MOMENTS (kip-ft) (1) - PART 2 (ROOF) | | | | | | |
|---|---|---|---|---|---|---|
| Girder Mark Number | Rain Load Moment R | | Wind Moment 1.0W | | Seismic Moment 1.0E | |
| | Left | Right | Left | Right | Left | Right |
| G1 | - | - | ± 105 | ± 105 | ± 120 | ± 120 |

(1) End Moment Sign Convention, Positive moments:

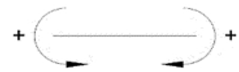



**Table 6.10.6a - Roof Joist Girder End Moments**

| JOIST GIRDER END MOMENTS (kip-ft) (1) (FLOOR) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Girder Mark Number | Dead Load Moment D | | Floor Live Load Moment L | | Wind Moment 1.0W | | Seismic Moment 1.0E | |
| | Left | Right | Left | Right | Left | Right | Left | Right |
| FG1 | 34.0 | 34.0 | 75.6 | 75.6 | ± 110 | ± 110 | ± 150 | ± 150 |

(1) End Moment Sign Convention, Positive moments:

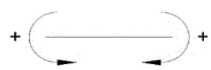



**Table 6.10.6b - Floor Joist Girder End Moments**

Add-Load categories are provided in Table 6.10.7a and 6.10.7b that could be included in a Joist Girder design. As previously discussed, Add-Loads are a good way to provide extra capacity into a Joist Girder and to account for loads where the exact location is not known. One example for use of Add-Loads is to account for wall bracing that is required for wind or seismic loads. Add-Loads may also be a way to build capacity into the Joist Girders where the size of mechanical units will not be determined until the Tenant Improvement phase.

| JOIST GIRDER ADD-LOADS (ROOF) | | | | | | | |
|---|---|---|---|---|---|---|---|
| Girder Mark Number | Dead Load (kips) | Dead Load Collateral (kips) | Roof Live Load $L_r$ (kips) | Snow Load S (kips) | Rain Load R (kips) | Seismic Load 1.0E (kips) | Wind Load 1.0W (kips) |
| G1 | - | 2.0 | 2.0 | 1.5 | - | - | - |

**Table 6.10.7a - Roof Joist Girder Add-Loads**

| JOIST GIRDER ADD-LOADS (FLOOR) | | | | | |
|---|---|---|---|---|---|
| Girder Mark Number | Dead Load (kips) | Dead Load Collateral (kips) | Floor Live Load L (kips) | Seismic Load 1.0E (kips) | Wind Load 1.0W (kips) |
| FG1 | - | 2.5 | 4.0 | - | - |

**Table 6.10.7b - Floor Joist Girder Add-Loads**

Bend-Check Load categories are shown in Table 6.10.8a and 6.10.8b that could be included in a Joist Girder design. As previously discussed, Bend-Check Loads are just for localized stress in the top or bottom chord and do not contribute to global moment or shear on the joist.

| JOIST GIRDER BEND-CHECK LOADS (1) - PART 1 (ROOF) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Girder Mark Number | Dead Load (kips) | | Dead Load Collateral (kips) | | Roof Live Load $L_r$ (kips) | | Snow Load S (kips) | |
| | TC | BC | TC | BC | TC | BC | TC | BC |
| G1 | - | - | - | 1.0 | 0.4 | - | - | - |

| JOIST GIRDER BEND-CHECK LOADS (1) - PART 2 (ROOF) | | | | | | | |
|---|---|---|---|---|---|---|---|
| Girder Mark Number | Rain Load R (kips) | | Seismic Load 1.0E (kips) | | Wind Load 1.0W (kips) | | |
| | TC | BC | TC | BC | TC | BC | |
| G1 | - | - | - | - | - | - | |

(1) For Bend-Check Load: TC = Top Chord, BC = Bottom Chord

**Table 6.10.8a - Roof Joist Girder Bend-Check Loads**

| JOIST GIRDER BEND-CHECK LOADS (1) (FLOOR) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Girder Mark Number | Dead Load (kips) | | Dead Load Collateral (kips) | | Floor Live Load L (kips) | | Seismic Load 1.0E (kips) | | Wind Load 1.0W (kips) | |
| | TC | BC | TC | BC | TC | BC | TC | BC | TC | BC |
| FG1 | 0.75 | - | - | 1.0 | 0.5 | - | - | - | - | - |

(1) For Bend-Check Load: TC = Top Chord, BC = Bottom Chord

**Table 6.10.8b - Floor Joist Girder Bend-Check Loads**

Shown in Table 6.10.9 are additional columns that could be added to a Joist Girder Schedule. These columns can be used in a schedule for roof Joist Girders and for floor Joist Girders. Moment of Inertia requirements are often needed for Joist Girders that are part of a moment frame. They can also be required for Joist Girders on projects that have vibration design criteria or ponding requirements. Joist Girders may need to have a minimum size, thickness or just width of the horizontal leg for a connection.

| JOIST GIRDER ADDITIONAL REQUIREMENTS | | | | | | | |
|---|---|---|---|---|---|---|---|
| Girder Mark Number | Min. Moment of Inertia $I_{chord}$ ($in^4$) | Min. Top Chord Thickness (in) | Min. Top Chord Horiz. Leg (in) | Min. Bottom Chord Thickness (in) | Min. Bottom Chord Horiz. Leg (in) | Min. Seat Angle Thickness (in) | Additional Requirements* |
| G2 | 1300 | 0.5 | 5 | 0.375 | 4 | 0.31 | Design Joist Girder Webs to transfer Axial loads from Top Chord to Bottom Chord |

*Additional Requirements Column is for additional information the Engineer of Record wishes to convey to Vulcraft. The note shown is just one example of the information that can be provided in this column.

**Table 6.10.9 - Joist Girder Additional Requirements**

## 6.11 BRIDGING CONSIDERATIONS

Joist bridging is required for the following reasons:

1. To align the joists during erection
2. To provide stability for the joist during erection
3. To provide gravity load stability for joists with standing seam roofs
4. To provide bracing for the bottom chord for wind uplift and axial loads
5. To control the slenderness ratio of the bottom chord
6. To assist in stabilizing the web system

In typical situations, the size, type and number of rows of bridging required depend on the length, spacing and designation of the joists in the area under consideration. The bridging requirements are also affected by wind uplift loading and the type of deck supported by the joists.

The two types of bridging are horizontal or diagonal. Horizontal bridging consists of continuous angles connected to the top and bottom chords. Diagonal bridging consists of pairs of angles that cross diagonally from the top chord to the bottom chord in the space between each joist. For typical situations, the required number of rows of bridging is given in tabular form in the SJI Standard Specifications for K, LH and DLH-Series joists. The SJI Standard Specifications also indicate when diagonal bridging is required. Bridging requirements for KCS joists are contained in the Vulcraft Manual. It is best to note on the structural drawings that only standard SJI bridging be sized and furnished. Any other bracing required would not be standard bridging and should be designed by the specifier and provided by others.

Bridging for all joists requires positive anchorage at the end of the bridging line. When a beam or a wall exists at the end of the bridging line, the bridging is normally anchored to the beam or the wall. When a joist exists at the end of the bridging line, X-bridging should be used in addition to horizontal bridging between the last two joists. This condition often exists at expansion joints and when joists are used in lieu of beams at end walls.

Standard bridging is required to laterally stabilize the top chord of the joists until the permanent deck is attached. Construction loads must not be applied to the joists until the bridging is attached to the joists and anchored at its ends.

Floor and roof decks usually have adequate stiffness to provide lateral stability to the top chord of joists subjected to design loads. The most common exception is standing seam roof systems. The specifying professional should assume that the standing seam roof has no diaphragm capability and specify that enough bridging be provided to laterally brace the joists under design loads. The standing seam roof may be able to stabilize the top chord of the joist, but this should be substantiated with test data. If the roof does not have sufficient diaphragm stiffness to brace the top chord, the joist and bridging design (size and spacing) must be adjusted to provide sufficient lateral bracing for the design loads.

Wind loading on joists will affect the design of the bridging. Under net uplift, the bridging is required to provide lateral stability to the bottom (compression) chord. The SJI Specifications require that the bridging design account for the uplift forces. The Specifications require that joists subjected to uplift have a line of bridging near each of the first bottom chord panel points. Depending on the actual amount of uplift, additional bridging may be required. For roof systems subjected to uplift, the authors recommend against designating the number of rows of bridging. Rather, the uplift should be specified on the design documents. See Section 6.2 and 6.8 regarding the specification of uplift forces for joists and Joist Girders.

The use of standard bridging, as a brace, to resist lateral loads due to monorails or other equipment should be evaluated carefully. The decision should often be made against using standard bridging for these situations. It is best for the specifier to design and detail additional bracing to resist the lateral loads. This bracing would be in addition to the typical joist bridging.

The bracing of Joist Girders is typically done using angles from the bottom chord of the girder to the joists at the panel points. The size of the angles will depend on the girder depth loading, the joist depth and the joist end panel length. The number of braces will depend on the girder loading. Additional braces may be required for Joist Girders with net wind uplift loading, depending on the magnitude of the net uplift loads.

## 6.12 SAMPLE SPECIFICATIONS

Sample specifications for joists, Joist Girders and deck can be downloaded from the Vulcraft website, www.vulcraft.com/design-tools.

# Chapter 7

# Connection Design

## 7.1 THE BASIC CONNECTION

The Basic Connection for interior columns framed with joists and Joist Girders is shown in Figure 7.1.1. As mentioned in Chapter 2 this connection is the least expensive and most common detail that can be used to transmit gravity loads to the column. The use of the Basic Connection to support wind or seismic moments to the column requires detailed connection design. In this section calculations are presented to assist the designer in the proper use of the Basic Connection. In addition, guidelines are presented to inform the designer when modifications must be made to the Basic Connection and when modifications must be made to joist and Joist Girder ends in order to safely carry wind, seismic and continuity moments. Section 4.5 provides additional discussion, details and summary of modifications to the basic connection for resisting moments. Joist and Joist Girder end moments are always in the form of a force couple. The couple is formed by the force in the chords times the distance between the centroids of the top and bottom chords. Throughout this Chapter methods of designing for these chord forces will be discussed. The specifying professional should be aware that modifications to the Basic Connection can increase the costs of joists and Joist Girders.

Before proceeding with wind and seismic moment effects on the Basic Connection, a point needs to be made relative to creating continuity by welding the Joist Girder and joist bottom chords to the column. If the top and bottom chords of the joists and Joist Girders are welded, the joists and Joist Girders will behave as continuous members, that is, continuity moments will be developed at the member ends. Welding the bottom chords in place after dead loads have been applied will eliminate the continuity moments from dead loads; however, continuity moments from live loads will still occur. Continuity moments will cause additional chord forces and consequently stresses in the joists, Joist Girders, and their connections. These forces must not be ignored in the design. A variety of problems can occur. These include:

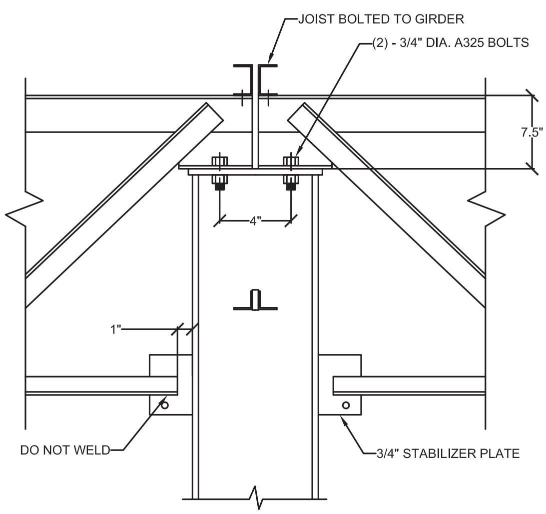



**Fig. 7.1.1 The Basic Connection**

1. Bending moments in the top chords of the joists and Joist Girders which will cause overstresses and possible chord bending failures

2. Compressive forces in the bottom chords of the joists and Joist Girders which are not accounted for in the normal design of these members and which may cause buckling failures of the joists and Joist Girders
3. Weld and bolt failures within the connection
4. Bottom chord forces may also cause column web buckling, web crippling and web yielding

As mentioned previously, these forces must not be ignored in the design. The specifying professional must calculate their magnitude and determine if the Basic Connection can safely support the loads.

The specifying professional is responsible for the design of the Basic Connection if it is subjected to any loads other than simple span gravity loadings. Vulcraft must check the adequacy of the joists and Joist Girders for any specified end moments created by wind, seismic or continuity loading. Vulcraft must also know whether the forces caused by the end moments are concentrically applied to the chords of the joists and Joist Girders, or if eccentricities exist which will cause bending stresses in the member chords. This can only be determined if given design documents clearly illustrating the connections. Chapter 6 deals with the proper specification of these forces. If the Basic Connection is used to resist continuity, wind or seismic moments these eccentricities will exist. Because the chord capacity is greatly reduced by eccentric loading, it is the responsibility of the specifying professional to limit the use of the Basic Connection to conditions where the joists and Joist Girders can be physically designed by Vulcraft to accommodate the intended loads.

The calculations presented below provide the specifying professional guidance as to whether or not the use of the Basic Connection would be appropriate for a given design. Based on the details and calculations presented, the specifying professional will also know if special measures are required to accommodate the connection eccentricities.

The calculations relative to the Basic Connection are divided into five sections:

(1) Maximum eccentric top chord force for Joist Girders

(2) Maximum eccentric top chord force for joists

(3) Rollover capacity of Joist Girder seats

(4) Modifications which can be made to the Joist Girders to resist forces greater than those indicated in (1) above

(5) Modifications which can be made to the Joist Girder seats and the joists to resist forces greater than those indicated in (2) above

(6) Special considerations relative to the connection of the joist and Joist Girder bottom chords

## Maximum Eccentric Top Chord Force for Joist Girders

As previously mentioned, chord moments caused by eccentric axial loads can cause premature failure of Joist Girder top chords. This condition is illustrated in Figure 7.1.2 below:

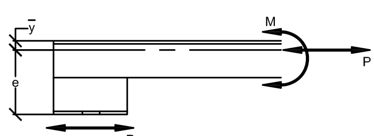



**Fig. 7.1.2 Joist Girder Chord Moments**

Considering the Joist Girder seat connection at the column top to be a “pinned connection,” a secondary moment, $M = \pm P(\mathrm{e})$ is developed in the top chord. The double angles which comprise the top chord possess a limited moment capacity; the Joist Girder chord will fail at a relatively low load due to the secondary moment and the axial load. If the seat can be rigidly attached to the column cap, then the chord moment can be eliminated. With the rigid attachment, the seat can be thought of as an extension of the column. The seat and its attachment to the column cap must be able to resist the moment, $\pm P(\mathrm{e})$. Using the standard single bolt line connection and welding along the seat angle edges, will generally not be sufficient to develop full rigidity. Calculations and details to accomplish a fully restrained seat connection are contained later in this chapter.

Vulcraft has done extensive testing of the maximum eccentric top chord force capacity for Joist Girders. Based on their test program, the maximum horizontal load for 7.5-inch-deep seats are presented in Table 7.1.1.

**JOIST GIRDER HORIZONTAL LOADS**

| Joist Girder (7.5″ Seat) Top Chord Leg Size | ASD $P_n/\Omega$ * kips | LRFD $\phi P_n$ * kips |
|---|---|---|
| 2.5″ | 4 | 6 |
| 3.0″ | 8 | 12 |
| 3.5″ and larger | 10 | 15 |

*These values are based on using 3/4-inch A325 bolts and a minimum of two 1/4-inch fillet welds 5 inches long along the sides of the seat. Vulcraft must be notified of seat forces for final seat design.

**Table 7.1.1 Maximum Horizontal Loads for Joist Girders**

## Maximum Eccentric Top Chord Force for Joists

Examining the Basic Connection shown in Figure 7.1.3, for the axial chord force in the joist to be transferred to the column, the force must pass thru the joist seat, thru the Joist Girder seat and into the column cap plate.

For the connection to function, either the connection of the joist seat to the top of the Joist Girder must be "fixed," or the connection between the Joist Girder seat and the column must be fixed. If neither is "fixed," a two-hinged mechanism would exist, and no lateral force could be resisted. In either case, a moment will exist in the top chord of the joist. (It should also be obvious that the Basic Connection is not well suited as a moment connection for the joists).

The moment in the top chord of the joist can be eliminated if both connections are "fixed." However, from a practical point of view it is very difficult to design the connection between the joist seat and the Joist Girder seat to be "fixed."

Even if stiffeners are placed in the seat of the Joist Girder to prevent Joist Girder seat from rotating, the seat angles of the joist are very flexible and would require stiffeners to "fix" the joist seat. Performing these measures results in a very uneconomical connection design. However, for small joist end moments the connection can be made to function. The approach taken herein is to design the connection between the Joist Girder seat and the column to be a fully restrained connection. Based on an ultimate approach the moment in the top chord of the joist equals the axial force in the joist times the distance from the top of the girder seat to the plastic centroid of the joist top chord. The Joist Girder seat to column connection must resist the moment caused by the joist chord axial force times the Joist Girder seat depth.

To determine the maximum permissible joist end moment that can be transferred through the Basic Connection, several potential failure mechanisms must be examined. These include:

(a) The failure of the joist top chord due to axial load and the chord moment
(b) The failure of the Joist Girder seat, i.e. rollover capacity of the Joist Girder seat
(c) Failure of the welds between the joist seat and the Joist Girder seat

The eccentric axial load strength for a K-Series joist chord can be determined by finding the axial load and bending moment combination for the top chord angles in the joist.

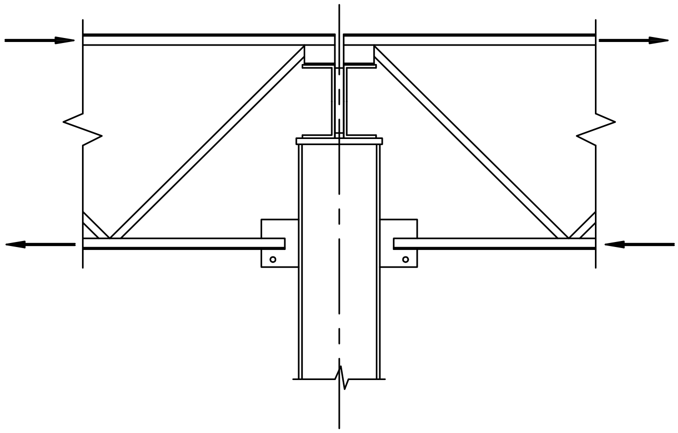


**Fig. 7.1.3 Force Transfer**

### Example 7.1.1 Joist Top Chord Reinforcement

**Given:**

The seat detail is shown in Figure 7.1.4.

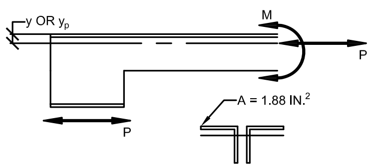



**Fig. 7.1.4 Joist Chords**

The seat is "pinned" at the support. Consider the case of a K-Series joist with top L2x2x1/4 top chord angles. The deck laterally supports the top chord thus, the limit state of lateral-torsional buckling does not occur. The maximum effective length of the top chord about its x-x axis is 24 inches. For the L2x2x1/4 angles the limit states of local buckling also do not occur.

Since the specifying professional does not know the exact geometrical configuration of the joist, and since chord forces due to the horizontal component in the end diagonal also exist, the maximum chord force can be used as an upper bound estimate. The solution also neglects any uniform distributed load on the joist top chord.

The specifying professional must indicate the joist chord force requirements on the contract documents.

### Solution:

The seat depth is 2.5 inches, therefore the moment in the chord equals top chord axial force generated by the end moment times the eccentricity from the bottom of the joist seat to the plastic centroid ($y_p$ = 0.236 in.).

$$M_a = P_a(2.5\text{ in.}-0.236\text{ in.}) = P_r(2.26\text{ in.})$$

For this condition, $P\Delta$ forces do not exist; however, small $P\delta$ forces do occur. The beam-column must satisfy AISC Specification Equations H1-1a or H1-1b.

When $\dfrac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0 \qquad \text{AISC Eq. (H1-1a)}$$

When $\dfrac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0 \qquad \text{AISC Eq. (H1-1b)}$$

Rather than performing a second order analysis which includes $P\delta$, use a solution using the $B_1$ multiplier. To determine the required moment, use Equations (A-8-1) and (A-8-2) from AISC Appendix 8.

$M_a = B_1 M_{nt} + B_2 M_{lt}$ AISC Eq. (A-8-1)

$M_a = B_1(2.26P_r)$

$P_a = P_{nt}$ (No second order effects on $P_a$) AISC Eq. (A-8-2)

where

$B_1 = C_m/(1\text{-}\alpha P_a/P_{el}) \geq 1$

$\alpha$ = 1.0 for LRFD and 1.6 for ASD

$C_m$ = 1.0 (Conservative for beam-columns subject to transverse loading between supports).

$P_{e1} = \pi^2 EI_x/(L_{c1})^2$

$I_x$ = 2(0.346 in.$^4$) = 0.692 in.$^4$ AISC Manual (Table 1-7)

$L_{c1}$ = 24 in.

$P_{e1} = \pi^2$(29,000 ksi)(0.692 in.$^4$)/(24 in.)$^2$ = 344 kips
$B_1$ = 1.0/[1-(1.0)($P_a$)/(344 kips)]
Take $B_1$ = 1.0 and verify later.

Determine $P_c$ for use in the interaction equations:

$P_n = F_{cr}A_g$
$L_c/r_x = KL/r_x$ = 24 in./0.605 in. = 39.7
$F_e = \pi^2 E/(L_c/r_x)^2 = \pi^2$(29,000 ksi)/(39.7)$^2$ = 182 ksi
$F_y/F_e$ = 50 ksi/182 ksi = 0.27
$F_{cr} = (0.658^{Fy/Fe})F_y = (0.658^{0.27})$50 ksi = 44.7 ksi
$A_g$ = 1.88 in.$^2$
$P_n$ = (44.7 ksi)(1.88 in.$^2$) = 84.0 kips
$P_c = \phi P_n$ = 0.9(84 kips) = 75.6 kips (LRFD)
$P_c = P_n/\Omega$ = 84 kips/1.67 = 50.4 kips (ASD)

Determine $M_{cx}$ for use in the interaction equations:

Depending on the moment direction, the top chord may be in tension or compression, thus the lower bound nominal moment strength of the double angle top chord must be determined from the vertical leg in compression or the vertical leg in tension.

For web legs in tension: $M_p = F_y Z_x \leq 1.6M_y$. AISC Eq. (F9-2)

For web legs in compression: $M_p = 1.5M_y$. AISC Eq. (F9-5)

The critical case is when $M_{cx}$ is a minimum, therefore AISC Equation F9-5 should be used.

For the two angles:

$Z_x = 0.88$ in.$^3$

$I_x = 0.70$ in.$^4$

$S_{xb} = 0.70$ in.$^4$/(2.0 in.-0.585 in.) = 0.495 in.$^3$

$M_y$ = (0.495 in.$^3$)(50 ksi) = 24.8 kip-in.

$S_{xt} = 0.70$ in.$^4$/0.585 in. = 1.20 in.$^3$

$M_y$ = (1.20 in.$^3$)(50 ksi) = 60.0 kip-in. (Causes maximum $M_p$)

Leg in tension:

$M_x = M_p$ = (50 ksi)(0.88 in.$^3$) = 44 kip-in.

Leg in compression:

$M_x = M_p$ = 1.5(24.8 in.$^3$) = 37.2 kip-in. (Critical case)

$M_{cx}$ = 0.9(37.2 kip-in.) = 33.5 kip-in. (LRFD)

$M_{cx}$ = (37.2 kip-in.)/1.67 = 22.3 kip-in. (ASD)

Substituting and solving for $P_r$ in the interaction equations:

For LRFD

When $\dfrac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0 \qquad \text{AISC Eq. (H1-1a)}$$

$$\frac{P_r}{75.6} + \frac{8}{9}\left(\frac{2.26P_r}{33.5}\right) = 1.0$$

$P_r = 13.7$ kips

When $\dfrac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) \leq 1.0 \qquad \text{AISC Eq. (H1-1b)}$$

$$\frac{P_r}{2(75.6)} + \left(\frac{2.26P_r}{33.5}\right) = 1.0$$

$P_r = 13.5$ kips

Check $B_1$ =1.0 assumption:

$B_1 = C_m/(1\text{-}\alpha P_r/P_{el}) \geq 1$
$= 1.0/[1\text{-}1.0(13.5 \text{ kips}/344 \text{ kips})] = 1.04$

Revising the calculations for $B_1$ = 1.04 results in $P_r$ = 13.0 kips

Solving the equations using the ASD values for $P_c$ and $M_{xc}$:

$P_r = 8.7$ kips

## Rollover Capacity of Joist Girder Seats

The Joist Girder seat is typically bolted to its supporting element. Welding may or may not be used to connect the seat to the supporting element. The rollover strength can be determined using the prying action procedure from the AISC Manual (AISC, 2017). The added strength of welding the seat angles when they are also bolted is negligible.

The standard Joist Girder has a 7.5 in. seat depth. The seat is usually fabricated using 7⁄16 in. seat angles as shown in Figure 7.1.5.

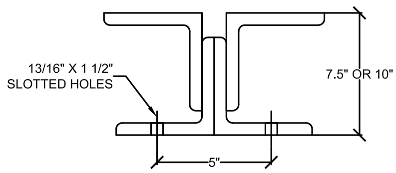



**Fig. 7.1.5 Standard Joist Girder Seat**

The rollover strength for the girder seat can be determined by calculating the maximum resisting couple that can be developed at the base of the seat angles. The couple is formed by the tensile bolt force and the corresponding compression on the opposite side of the seat. The force system is shown in Figure 7.1.6.

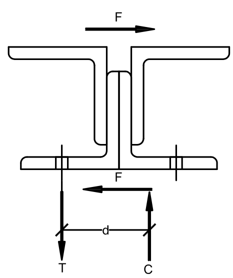



**Fig. 7.1.6 Joist Girder Rollover Resistance**

The available force is determined from the AISC hanger equations. If 3/4-in. A325 bolts are used to connect the seat to the supporting element, the strength will normally be controlled by the bending resistance of the seat angle.

## Example 7.1.2-7⁄16 inch Seat Angles (Standard seat)

Determine the rollover resistance for a Joist Girder with 7/16 in. thick seat angles.

**Given:**

Seat angles $F_y = 50$ ksi

¾ in. A325 bolts with a 5 in. gage

Seat length = 7.0 in.

The bolts are placed at mid-length of the seat length.

**Solution:**

$a$ = 1.0 in.

$a' = a + d_b/2$ = 1.0 in. + (0.75 in.)/2 = 1.375 in.

$b = g/2 - t$ = 2.5 in. - 0.4375 in. = 2.06 in.

where, $t$ = 7/16 in. = 0.4375 in.

$b' = b - d_b/2$ = 2.06 in.- (0.75 in.)/2 = 1.69 in. AISC Manual Eq. (9-19)

$d'$ = slot length = 2.0 in.

$B_c$ = 29.8 kips (LRFD)

= 19.9 kips (ASD) AISC Manual (Table 7-2)

$F_u$ = 65 ksi

Seat length: $L_s$ = 7.0 in.

$p = 3.5b \le L_s$ = (3.5)(2.06 in.) = 7.21 in. > 7.0 in.

$p$ = 7.0 in.

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{2.0\text{ in.}}{7.0\text{ in.}} = 0.71$$ AISC Manual Eq.(9-20)

$$\rho = \frac{b'}{a'} = \frac{1.69\text{ in.}}{1.375\text{ in.}} = 1.23$$

$$t_c = \sqrt{\frac{4B_c b'}{\phi p F_u}} = \sqrt{\frac{(4)(29.8\text{ kips})(1.69\text{ in.})}{(0.9)(7.0\text{ in.})(65\text{ ksi})}} = 0.70\text{ in.}$$ AISC Manual Eq.(9-19a)

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] = \frac{1}{(0.71\text{ in.})(1+1.23)}\left[\left(\frac{0.70\text{ in.}}{0.4375\text{ in.}}\right)^2 - 1\right] = 0.99$$ AISC Manual Eq.(9-28)

$$Q = \left(\frac{t}{t_c}\right)^2(1+\delta\alpha') = \left(\frac{0.4375\text{ in.}}{0.70\text{ in.}}\right)^2\left[1+(0.71)(0.99)\right] = 0.67$$

$$T_c = QB_c = (0.67)(29.8\text{ kips}) = 20.0\text{ kips (LRFD)}$$ AISC Manual Eq.(9-27)

$$T_c = QB_c = (0.67)(19.9\text{ kips}) = 13.3\text{ kips (ASD)}$$

The rollover strength, $V_c$, equals the force $T_c$ times the lever arm, $d$ divided by the seat depth, 7.5 in.

$d$ is taken as $g/2$ + k.

$k$ = for 7/16 in. angles = 13/16 in.= 0.8125 in.

$d$ = $g/2 + k$ = 5.0 in./2 + 0.8125 = 3.32 in.

$V_u$ = $T_c\, d$/7.5 in. = (20.0 kips)(3.32 in.)/7.5 in.

For LRFD: $V_c$ = 8.85 kips (LRFD)

For ASD: $V_c$ = 5.89 kips (ASD)

Using this design procedure also provides the uplift values for the seat, i.e. for this case the uplift on the seat equals $2T_c$ = 2(20.0 kips) = 40 kips (LRFD), 26.6 (ASD).

### Example 7.1.3-¼ inch Seat Angles (Non-standard seat)

Determine the rollover resistance for a Joist Girder with ¼ in. thick seat angles.

The solution is based on calculating the available bolt tension force strength using the prying procedure from the AISC Manual.

**Given:**

Seat angles: $F_y$ = 50 ksi

¾ in. A325 bolts with a 5 in. gage

Seat length = 7.0 in.

The bolts are placed at mid-length of the seat length.

**Solution:**

$a$ = 1.0 in.

$a' = a + d_b/2$ = 1.0 in. + (0.75 in.)/2 = 1.375 in.

$b = g/2 - t$ = 5 in./2 - 0.25 in. =2.25 in.

where, $t$ = ¼ in. = 0.25 in.

$b' = b - d_b/2$ = 2.25 in. – (0.75 in.)/2 = 1.875 in. AISC Manual Eq. (9-19)

$d'$ = slot length = 2.0 in.

$B_c$ = 29.8 ksi (LRFD)

$B_c$ = 19.9 ksi (ASD)

$F_u$ = 65 ksi

Seat length, $L_s$ = 7.0 in.

$p = 3.5b \le L_s$ = (3.5)(2.25 in.) = 7.88 in. > 7.0 in.

$p$ = 7.0 in.

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{2.0\text{ in.}}{7.0\text{ in.}} = 0.71$$ AISC Manual Eq.(9-20)

$$\rho = \frac{b'}{a'} = \frac{1.875\text{ in.}}{1.375\text{ in.}} = 1.364$$

$$t_c = \sqrt{\frac{4B_c b'}{\phi p F_u}} = \sqrt{\frac{(4)(29.8\text{ ksi})(1.875\text{ in.})}{(0.9)(7.0\text{ in.})(65\text{ ksi})}} = 0.74\text{ in.}$$ AISC Manual Eq.(9-19a)

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] = \frac{1}{(0.71\text{ in.})(1+1.364)}\left[\left(\frac{0.74\text{ in.}}{0.25\text{ in.}}\right)^2 - 1\right] = 4.62$$ AISC Manual Eq.(9-28)

$$Q = \left(\frac{t}{t_c}\right)^2 (1+\delta\alpha') = \left(\frac{0.25\text{ in.}}{0.74\text{ in.}}\right)^2 \left[1+(0.71)(4.62)\right] = 0.49$$

$T_c = QB_c = (0.49)(29.8\text{ kips}) = 14.6$ kips (LRFD) AISC Manual Eq. (9-27)

$T_c = QB_c = (0.49)(19.9\text{ kips}) = 9.75$ kips (ASD)

The rollover strength, $V_c$, equals the force $T_c$ times the lever arm, $d$ divided by the seat depth, 7.5 in.

$d$ is taken as $g/2 + k$.

$k$ = for 1/4 in. angles = 0.50 in.

$d = g/2 + k$ = 5.0 in./2 + 0.50 = 3.00 in.

$V_u = T_c\, d/7.5$ in. = (14.6 kips)(3.00 in.)/7.5 in.

For LRFD: $V_c$ =5.8 kips

For ASD: $V_c$ = 3.9 kips

Using this design procedure also provides the uplift values for the seat, i.e. for this case the uplift on the seat equals $2T_c$ = 2(14.6 kips) = 29.2 kips (LRFD) and 2(9.75 kips) = 19.5 kips (ASD).

### Weld Requirements

The third failure mode that must be considered is the capacity of the welds connecting the joist seat to the girder top chord. The specifying professional is responsible for the design of these welds. Because of the relatively low axial load resistance of the joist, the welding requirements are minimal and present no special problem. The weld must be designed to resist the maximum shear force occurring at the joist seat. Some bending may also exist on the weld group. A minimum fillet weld of 3⁄16" by 2-½" long on each side of the joist seat is recommended to resist the 8.7 kip joist axial force. These requirements must be considered in detail when modifications are made to the Basic Connection.

### Summary

For ASD:

1. Per Table 7.1.1 Joist Girder force **couples** are limited to 4 to 10 kips times the distance from the centroid of the bottom chord to the top of the column
2. The allowable eccentric chord force permitted on a K-Series joist (2x2x1/4 in. chords) is 8.7 kips
3. The maximum lateral shear force (rollover force), $V_n/\Omega$ that can be applied perpendicular to a **standard Joist Girder seat** (7.5 in. seat) is 5.89 kips

For LRFD:

1. Per Table 7.1.1 Joist Girder force couples are limited to 6 to 15 kips times the distance from the centroid of the bottom chord to the top of the column
2. The maximum design strength eccentric chord force permitted on a K-Series joist (2x2x1/4 in. chords) is 13.0 kips
3. The maximum lateral shear force (rollover force), $\phi V_n$ that can be applied perpendicular to a **standard Joist Girder seat** is 8.85 kips

## 7.2 MODIFICATIONS TO THE BASIC CONNECTION

Based on the preceding calculations that only small moments can be transferred to the column using the Basic Connection. Modifications can be made to resist larger moments. See Chapter 4 for additional details of joist and Joist Girder moment connections.

### Joist Girder Modifications

Several modification options exist for Joist Girders.

For interior girders, the top chords of adjacent girders can be connected to one another to virtually eliminate the continuity moments (chord forces) from passing through the Joist Girder seats. The Joist Girder seat is then required to only transfer wind and seismic moments into the column.

To obtain greater capacities than those shown in Table 7.1.1 the Joist Girder seat detail can be modified to achieve "full" rigidity to the column top, or the Joist Girder can be modified to carry additional eccentric chord moment.

The design of the seat attachment to the column top is the responsibility of the specifying professional, whereas the modification of the Joist Girder to accommodate the secondary moment is the responsibility of Vulcraft.

The most common methods of continuity transfer are shown in Fig 7.2.1 and 7.2.2. The connection angles or plates are sized to resist the full continuity moments. The welding of the angles or plates to the top chords is accomplished using standard procedures. The designer is cautioned to limit the size of the fillet welds to 3/16 inch, if possible, so that the tie angle thickness does not govern the allowable weld capacity, and so that the weld size does not exceed the top chord thickness.

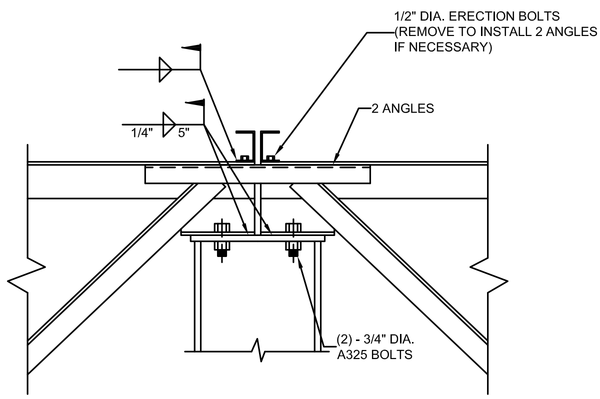



**Fig. 7.2.1 Continuity Angles**

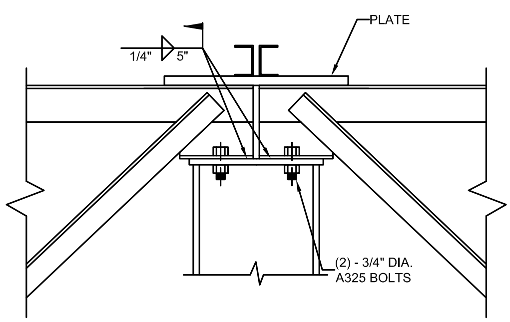



**Fig. 7.2.2 Continuity Plates**

### Design of "Fixed" Joist Girder Seats

The attachment of the seat for full rigidity is accomplished effectively by using additional bolts between the seat angle and the column cap. Conventional connection design approaches can be used to design the connection. The "fixity" to the supporting element, or column, is accomplished by the couple between the bolt tension force and the bearing of the seat against the supporting element. Shown in Figure 7.2.3 is the resisting force couple.

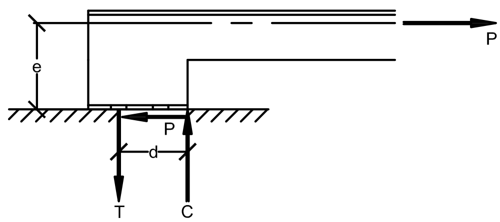



**Fig. 7.2.3 Fixed Joist Girder Seat**

For force ($P$) reversal, the bearing point will occur at the heel of the seat and the interior bolts will be in tension. The capacity of the force resisting couple $Td$ must be greater than the overturning moment created by the axial force $P$ times the eccentricity $e$. The axial force $P$ is located at the centroid of the top chord angles. The maximum force $T$ is determined in the same manner as in the Joist Girder seat rollover calculations in Section 7.1. The location of the compressive force $C$ is found based on the contact area required to resist the compression force (see Figure 7.2.4). $C$ must equal $T$. If the width of bearing is taken as $2.5k$, where $k$ is the distance from the back of the seat angles to the edge of the angle fillet, the compressive force equals $2.5kL'(\phi F_y)$. $L'$ is determined by setting the compressive force equal to the tension force $2T$, where $T$ is the force in each bolt.

Thus,

$L' = 2T_c/[2.5k(\phi F_y)]$ (LRFD)

$L' = 2T_c/[2.5k(F_y/\Omega)]$ (ASD)

From Figure 7.2.4 $d = L - L'/2$.

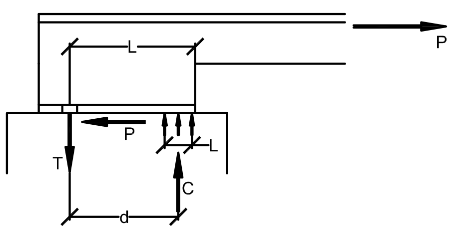



**Fig. 7.2.4**

To determine (e) the size of the top chord must be known. The specifying professional can estimate the top chord angle size from Table 7.1.2. The procedure is to first calculate the value of the parameter "A" using the equation applicable to the Joist Girder in question. Based on the "A" value, enter Table 7.1.2 to determine the minimum top chord width. Since a one-inch space exists between the top chord angles, an "A" value of 2.65 or larger will typically indicate that 4-inch chord angles would be used for the girder.

For an odd number of joist spaces:

$$A = \frac{0.028P}{D}(N^2S - 0.67N + 0.67 - S)$$

For an even number of joist spaces

$$A = \frac{0.028P}{D}(N^2S - 0.67N + 0.67)$$

where

P = Panel point load (kips)

N = No. of joist spaces

S = Joist spacing (feet)

D = Joist Girder depth (in.)

**JOIST GIRDER CHORD WIDTHS**

| A, in$^2$ | Minimum Top Chord Width, in. |
|---|---|
| 0.00 - 0.94 | 5 |
| 0.95 - 1.19 | 6 |
| 1.20 - 1.78 | 7 |
| 1.79 - 2.64 | 8 |
| 2.65 - 3.75 | 9 |
| 3.76 - 4.75 | 11 |
| 4.76 - 8.44 | 13 |
| Greater than 8.44 | Consult with Vulcraft |

Note: Vulcraft Joist Girder chords are always equal legged angles.

**Table 7.1.2 Joist Girder Minimum Top Chord Width (ASD)**

### Example 7.2.1 Joist Girder Fixed Seat

Design a Joist Girder to column connection which can transmit a chord force, $P_a$ = 25 kips (ASD).

**Given:**

The geometry is shown in Figure 7.2.5.

A 40G8N12K girder is used.

The joist spacing equals 5 feet.

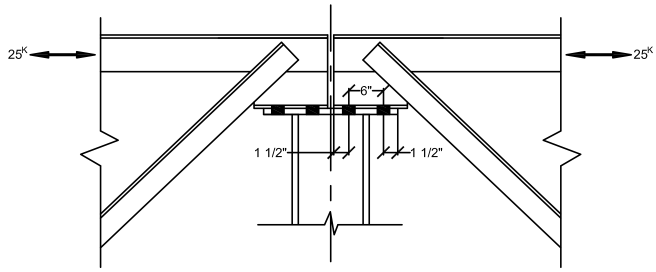



**Fig. 7.2.5 Example 7.2.1**

## Solution:

*Preliminary Design:*

Estimate the top chord size for the 40G8N12K girder.

From Table 7.1.2:

$A = 0.028P[N^2S\text{-}0.67N+0.67]/D$

$A = (0.028)(12.0\text{ kips})[(8)^2(5\text{ ft.})\text{-}(0.67)(8)+0.67]/40\text{ in.} \approx 2.65\text{ in.}^2$

From Table 7.1.2 the minimum chord width is 8 inches, thus the chords will likely be 3-1/2-in. angles.

Assume angle thickness is ½ in.

*Determine the moment resistance:*

The moment at the base of the seat equals the force *P* times its height above the column:

$M_a = P[(7.5\text{ in.})\text{-}(y_{bar})]$

where

$y_{bar}$ = the centroid distance for a 3-1/2x3-1/2x1/2 angle = 0.75 in.

$M_a$ = (25 kips)[(7.5 in.) - (0.75 in.)] = 169 kip-in.

Based on the calculations made in Example 7.1.2 the allowable bolt tension for a 7/16-inch seat angle is 13.1 kips.

$L' = 2T_c/[2.5k(F_y/\Omega)]$ = (2)(13.3 kips)/[(2.5)(0.875 in.)(50 ksi/1.67)] = 0.41 in.

where:

$k$ = 0.875 in.

From Figure 7.2.5,

$d$ = 6.0 in. + 1.5 in. - 0.41 in./2 = 7.3 inches

The moment resistance is:

$M/\Omega = 2T_cd$ (2bolts)

= (2)(13.3 kips)(7.3 in.) = 194 kip-in. > 169 kip-in. **o.k.**

Since the holes in the standard Joist Girder seat angles are long slots, the seat angle should be welded to the column to resist the wind shear of 25 kips. Using 1/4 fillet welds on both sides of the seat angle requires a length of weld of approximately 4 inches.

The final solution is shown in Figure 7.2.6. The designer must also check the cap plate thickness.

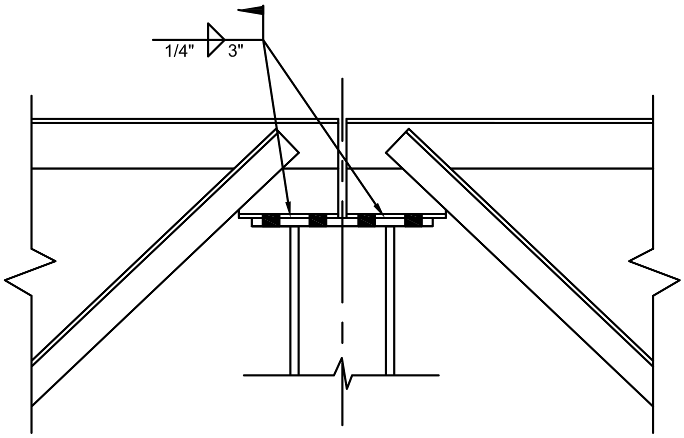



**Fig. 7.2.6 Solution Example 7.2.1**

### Increasing Joist Girder Chord Capacity with Seat Extensions

The Joist Girder eccentric top chord force can be increased by using a seat extension on the Joist Girder. Vulcraft refers to the seat extension as an E member. In most cases the Joist Girder chord can be reinforced by inserting a one-inch thick plate between the top chord angles. In some instances, the seat angles are extended back into the girder. This is an expensive detail and should be avoided if possible. In either case, the moment created by the eccentric force is resisted by the reinforced chord extension. Both types of extensions are illustrated in Figure 4.5.4. As mentioned in Section 4.5 a practical eccentric chord force limit is 50 kips for Joist Girder seat extensions. The specifying professional should indicate locations and force requirements for Joist Girder E member extensions on the structural drawings.

### Joist Modifications

Connection plates can be used to connect the top chords of the joists together to eliminate the forces from the continuity moments in the joist seat. Continuity moments can also be reduced by specifying round rod bottom chord extensions. A discussion about the use of rod extensions is contained in Section 7.3.

The joist eccentric top chord force capacity can be increased to some extent by reinforcing the joist using E member reinforcing.

### Increasing Joist Moment Resistance with Joist Girder Seat Stiffeners

By adding vertical stiffeners in the Joist Girder seat, the resistance to rollover can be improved. If the stiffener is placed near the bolts in the seat, the bolt force can be increased. In addition, the lever arm between the resisting force couple is increased. The result is a significant increase in seat rollover capacity. In some cases, it may not be possible for Vulcraft to place stiffeners adjacent to the bolt holes. For these cases stiffeners in combination with welding the seat to the supporting member may be used to increase the rollover resistance. The rollover strength of a Joist Girder seat with stiffeners is illustrated in Example 7.2.2.

### Example 7.2.2 Joist Girder Rollover Strength with Seat Stiffeners

Determine the allowable strength (ASD) and the design strength (LRFD) transverse shear (rollover) on the Joist Girder seat shown in Figure 7.2.7.

**Given:**

$F_y$ of the stiffeners = 36 ksi

The stiffener width = 4 in.

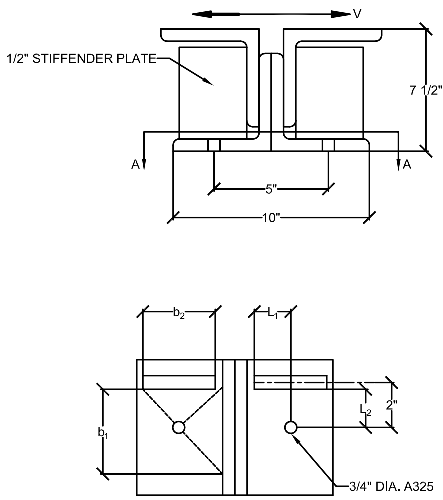



**Fig. 7.2.7 Example 7.2.2**

**ASD Solution:**

*Determine the effective bending length along the 7⁄16 in. seat angle and along the stiffener:*

For the seat angle:
The effective length = $b_1$ = 2.5 $L_1$, where 2.5 is from (Young, 2011).

$L_1$ = 2.5 in.-$k$ = 2.5 in.-0.875 in. = 1.625 inches

$b_1$ = (2.5 in.)(1.625 in.) = 4.06 inches

One-half of $b_1$ must be less than the distance from the hole to the fillet weld on the stiffener ($L_2$) so that the effective length for the angle does not overlap with the effective length for the stiffener.

$L_2$ = 2.0 in.-($t_s$ in.)/2- the fillet weld size

Use a 1/4-inch fillet weld.

$L_2$ = 2.0 in.-0.25 in.- 0.25 in. = 1.50 inches

$b_1$ /2 = 4.06 in./2 = 2.03 inches > 1.5 inches

Use $b_1$ = 1.5 + 2.03 = 3.53 inches.

For the effective bending length adjacent to stiffener:

$b_2$ = 2.5$L_2$ = (2.5)(1.5 in.) = 3.75 in.

Maximum width = (10.0 in.)/2-$k_{seat\ angle}$

$= 5.0$ in.-0.8125 in.

$= 4.19$ in.

where

$k_{seat\ angle} = 13/16$ in.

Use $b_2 = 3.75$ in.

*Determine the plastic moment capacity along the 7/16-inch seat angle and along the stiffener:*

$M_p = ZF_y$, where $Z = bt^2/4$

Along the seat angle:

$Z = (3.53 \text{ in.})(0.4375 \text{ in.})^2/4 = 0.17$ in.

$M_p = F_y Z = (50 \text{ ksi})(0.17 \text{ in.}) = 8.50$ kip-in..

Along the stiffener:

$Z = (3.75 \text{ in.})(0.4375 \text{ in.})^2/4 = 0.18 \text{ in.}^3$.

$M_p = F_y Z = (50 \text{ ksi})(0.18 \text{ in.}^3) = 9.0$ kip-in.

*Determine the bolt force:*

The nominal bolt force equals the plastic moment divided by the distance to the inflection point between the bolt and the plastic moment location. Assume the inflection points to be 1/2 of $L_1$ and $L_2$.

The nominal bolt force:

$T_n = (8.45 \text{ kip-in.})/(L_1 \text{ in.}/2) + (9.0 \text{ kip-in.})/(L_2/2)$.

$T_n = (8.45 \text{ kip-in.})/(1.625 \text{ in.}/2) + (9.0 \text{ kip-in.})/(1.5 \text{ in.}/2)$

$= 10.4 \text{ kips} + 12.0 \text{ kips} = 22.4$ kips.

Using a safety factor, $\Omega = 1.67$

$T_a = (22.4 \text{ kips})/1.67$ kips

$= 13.4$ kips

$T_a$ must be less than the AISC available tensile strength.

The available tensile strength = 19.9 kips AISC Manual (Table 7-2)

13.4 kips < 19.9 kips **o.k.**

Note: Prying forces are generally neglected when bolts are in reentrant corners as shown in Figure 7.2.7.

*Determine the allowable transverse shear:*

The compression force can be assumed to act at the edge of the stiffener.

Lever arm = 2.5 in. + $k_{seatangle}$ + stiffener width

Lever arm = 2.5 in. + 0.8125 in. + 4 in. = 7.31 in.

The available moment

$M_c = (7.31 \text{ in.})T_a$

$= (7.31\text{ in.})(13.4\text{ kips})$

$= 98.0\text{ kip-in.}$

The allowable shear $V_c$, equals $M_c$ divided by the seat depth.

$V_c = (98.0\text{ kip-in.})/(7.5\text{ in.}) = 13.1\text{ kips}.$

If the seat is not welded to the column cap, the bolts must be checked for combined tension and shear. Using the AISC Specification for threads excluded from the shear plane for bearing type connections the allowable bolt tension stress equals:

$F'_{nt}$ = nominal tensile stress modified to include the effects of shear stress, ksi

$$=1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt} \text{ (LRFD)} \qquad \text{AISC Eq. (J3-3a)}$$

$$=1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_{rv} \leq F_{nt} \text{ (ASD)} \qquad \text{AISC Eq. (J3-3b)}$$

$F_{nt}$ = nominal tensile stress from AISC Table J3.2, ksi

$F_{nv}$ = nominal shear stress from AISC Table J3.2, ksi

$f_{rv}$ = required shear stress using LRFD or ASD load combinations, ksi

Since two bolts resist the shear,

$f_{rv} = V/(\text{bolt area})$

$f_{rv} = (13.1\text{ kips})/[(2)(0.44\text{ in.}^2)] = 14.9\text{ ksi}$

$F_{nt} = 90\text{ ksi}$

$F_{nv} = 68\text{ ksi}$

Solving for $F'_{nt}$ from AISC Eq. J3-3b

$F'_{nt} = 77.6\text{ ksi}.$

$13.4\text{ ksi} < F'_{nt}/\Omega$

where $\Omega = 2.0$

$13.4\text{ ksi} < 77.6\text{ ksi}/2.0 < 38.8\text{ ksi}$ **o.k.**

The shear in the bolts could be eliminated by welding the seat angle to the column cap.

*Determine the stiffener weld requirements:*

Force in stiffener equals the percent of the bolt force that goes into the stiffener. This can be determined based on the proportion of moment that goes into the $b_2$ length.

Stiffener force:

$F_r = T_r[9.0/(9.0+8.45)] = 0.52\ T_r$

$= (0.52)(13.4\text{ kips})$

$= 6.97\text{ kips}$

Welding on one side only:

Fillet size:

$D(0.707)(0.6)(F_{EXX})(3.75\text{ in.})/\Omega = 6.97\text{ kips}$

$\Omega = 2.0$

$F_{EXX} = 70$ ksi

Solving for $D$:

$D = 0.13$ in.; Use 1/4 in. fillet weld.

*Check stiffener size:*

Available force = $F_y A/\Omega$

$P_a = (36 \text{ ksi})(3.81 \text{ in})(0.5 \text{ in.})/1.67 = 41.1$ kips

6.90 kips < 41.1 kips **o.k.**

Therefore, The allowable transverse shear, $V_c = 13.4$ kips (ASD).

Repeating the calculations for LRFD, $V_c = 20.1$ kips (LRFD)

## Increasing Joist Chord Capacity with Seat Extensions

A seat extension detail (E member) for a standard joist is shown in Figure 7.2.8. The purpose of the E member is to provide reinforcing to the joist top chord. The reinforcing is designed to resist the secondary moment. The E member acts compositely with the top chord to form a shape which is much more effective in resisting moment than the top chord angles.

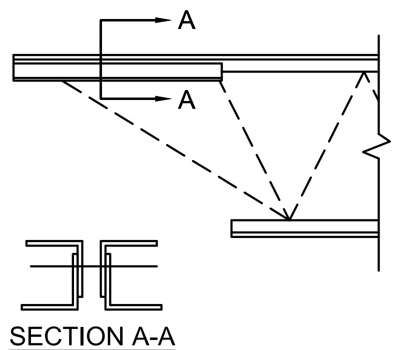



**Fig. 7.2.8 Joist with Seat Extension**

The E member is designed by Vulcraft. However, Vulcraft must be provided the information regarding the connection and the imposed forces so that a proper design can be made. It is also important that the specifying professional know whether the E member solution is feasible.

By examining the maximum chord size for a K joist (2x2x1/4) with an (S TYPE) member extension, an upper bound can be obtained. The solution is provided below.

The properties for the extended seat for a K12 joist shown in Figure 7.2.9 are:

$I_x = 3.016$ in.$^4$, $A = 3.76$ in.$^2$, $S = 2.413$ in.$^3$, $y_{bar} = 1.25$ in., $r_x = 0.90$ in.

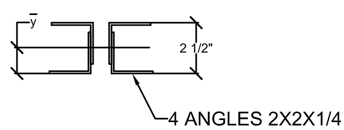



**Fig. 7.2.9 Extended Seat for a K12 Joist**

## Example 7.2.3 Extended Seat

**LRFD Solution:**

The AISC combined bending and axial load equations can be solved in order to determine the maximum allowable force $P_c$.

The moment in the extended seat equals the force, $P_u$, at the bottom of the seat, times $y_{bar}$

$M_{ux} = 1.25P_u$

Second order effects are considered neglectable for the calculation of $M_{ux}$

If the deck is assumed to laterally brace the chord about its y-y axis, then $P_c$ is based on the x-x properties. For a K-Series joist, the maximum unbraced length of the chord about the x-x axis is approximately 48 inches, and assuming $K$=1.0, $L_c = KL$ =48 in.

$L_c/r_x$ = (48 in.)/(0.9 in.) = 53.3 in.

$\phi F_{cr}$ = 36.7 ksi AISC Manual Table (4-14)

$P_c = \phi F_{cr}A$ = (36.7 ksi)(3.76 in.$^2$) = 138 kips

$M_{cx} = \phi M_y$ = (0.9)(50 ksi)(2.413 in.$^3$) = 109 kip-in.

$M_y$ is used here for $M_{cx}$ since the attachment of the angles to one another may consist of intermittent welds and the section cannot be considered compact.

Solving the interaction equations for $P_u$ = 57.3 kips (LRFD).

Solving the interaction equations for $P_a$ = 38.2 kips (ASD).

Since the end panel geometry may be slightly different than that assumed above, the specifying professional should not arbitrarily use an E member extension without notifying Vulcraft of the force requirements.

### Summary

1. Continuity ties can be used to connect adjacent Joist Girder chords together, virtually eliminating the continuity chord forces from passing through the Joist Girder seats. Thus, the entire force capacity per Table 7.1.1 can be utilized for wind or seismic forces
2. Joist Girder fixed seats can be designed to increase the Joist Girder seat resistance beyond the Table 7.1.1 values. The magnitude of increased capacity is dependent upon the distance between the seat bolts, and the strength of the seat bolts
3. Seat stiffeners can be used to increase the rollover strength of seats
4. E member extensions can be used on joists to increase the eccentric force capacity

## 7.3 BOTTOM CHORD EXTENSIONS

As mentioned in Section 7.1, when the joist or Joist Girder bottom chords are extended and welded to the column, continuity moments will be developed. Several situations must be examined when the bottom chords are extended and welded to the column. These include:

1. Determining the magnitude of the continuity, wind and seismic forces
2. Design of the bottom chord for the continuity, wind, and seismic forces
3. Design of the bottom chord connection to the column

The specifying professional must perform a structural analysis order to determine the forces in the bottom chord. Comments regarding rigid frame analysis are made in Chapter 4. The specification of these forces is discussed in Chapter 6. It is the responsibility of Vulcraft to design the bottom chords of the joists and girders for the specified forces. However, it is the responsibility of the specifying professional to design the connection to the column. The specifying professional must consider:

1. The magnitude of the bottom chord force
2. The geometrical and tolerance requirements imposed by the joist or girder
3. The type of column

The magnitude of the bottom chord force is dependent upon the loads on the structure and the way the structure is framed. The magnitude of the chord force will dictate the type of connection used. As mentioned earlier, continuity forces can be reduced by welding the bottom chords to their supports after dead loads are applied. However, it is usually better to account for the dead load moments initially rather than requiring the erector to return to weld the connections. In addition, joist bottom chord continuity forces can be substantially reduced by using bottom chord extensions which are not capable of generating large forces. The round rod extension has been used precisely for this reason. The rod extension can be designed to elastically buckle at low axial loads. The maximum force generated is then the elastic buckling load of the rod. The joist and its rod extension can be used to provide moment resistance, provided the rod is acting in tension. Based on the joist depth, the specifying professional can calculate the approximate rod extension force. In general, the rod length must be at least 2.5 times the joist depth to ensure that it will lap sufficiently with the joist bottom chord. In order to obtain the most conservative compressive force value, the ends of the rod should be considered as fixed.

### Geometrical and Tolerance Requirements

Since the exact size and thickness of the bottom chord angles are generally not known by the specifying professional, the connection must be designed and accommodate these variations. Of specific concern is the size of welds. If possible, fillet weld sizes should be restricted to 3⁄16 inches for joist chord extensions. This will accommodate the typical thicknesses of the bottom chords. Thegap between joist bottom chords will vary between ½ of an inch to 1 inch. The space between the bottom chords for Joist Girders is typically one inch. The use of ¾ inch stabilizer plates are common. Typically, the Joist Girder chords can be squeezed together and clamped to the stabilizer prior to welding. Joist Girders with heavy bottom chords, i.e. those with 5 and 6-inch angles, may not be able to be squeezed together. A ⅞-inch plate may be preferable in these instances. A one-inch thick plate may not allow enough tolerance to pass between the chords during erection.

The designer should also be aware that bottom chords are not cut to exact lengths. Their length can vary by ± ½ inch; thus, the bottom chord detail must allow for length tolerance.

### Column Considerations

Three conditions exist relative to chord extensions into columns. These are:

1. Attachment to the flange of W shapes
2. Attachment to the web of W shapes
3. Attachment to HSS column walls

Attachment to the flange of a wide flange column is best accommodated by using a stabilizer plate. This situation is shown in Figure 7.3.1. The stabilizer is welded to the face of the flange. If the stabilizer is "cut to fit" against the flange, the welds need not be designed to transfer the compression force into the column. However, they must be designed to transfer any tension force. The stabilizer plate must be designed to resist the chord loads.

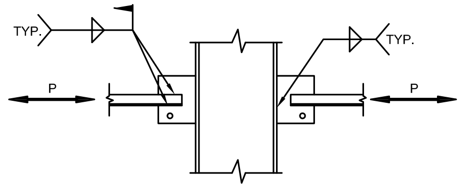



**Fig. 7.3.1 Bottom Chord Attachment to Flanges**

The column web must be checked for its ability to resist the applied forces. For extensions causing compression in the column web, the web must be checked for web local yielding, web local crippling and web compression buckling when the stabilizer plates are on both sides of the column. The AISC Specification does not address this specific geometrical situation. However, basic principles may be used to determine the need for stiffeners or doubler plates. The web is assumed to act as a column spanning between the flanges to resist the compressive force from the chords. The following criteria can be used for this check:

1. The effective web width can be assumed to equal the height of the stabilizer plates plus 5 k
2. The effective length factor for the web should be taken as 1.0

If the column web requires stiffening, doubler plates or stiffeners may be used. If only a slight overstress exists, the stabilizer plates can be heightened to increase the effective web column height until the web does not buckle or a column with a thicker web can be selected. Doubler plates can be sized in the same manner that the column web is checked. Unless measures are taken to connect the doublers to the column web, they must be designed as individual columns. They should be placed on both sides of the column web to avoid eccentricities in loading from the stabilizer plate. In lieu of doubler plates, fitted stiffeners could be used to prevent buckling of the column web. The increase in strength may be based on the larger allowable unit stress in the web due to the prevention of buckling. However, the effectiveness of the stiffener area is questionable since load from the stabilizer plate cannot get directly into the stiffener.

The condition when the stabilizer plates are connected to the web of a wide flange column is illustrated in Figure 7.3.2. The stabilizer plates should extend beyond the column flanges to facilitate erection of the girder. As depicted, only small wind and seismic forces could be transferred to the column, since the stabilizer plates frame into the web of the column.

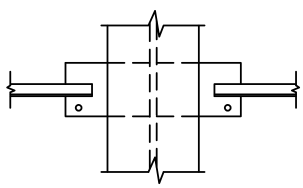


**Fig. 7.3.2 Framing to Column Web**

To transfer the wind and seismic bottom chord forces into the weak axis of the column, stiffeners can be added to each side of the stabilizer plate as shown in Figure 7.3.3.

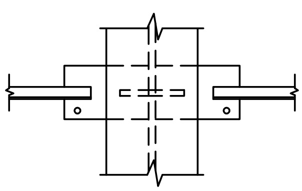


**Fig. 7.3.3 Stiffener Reinforcement to Stabilizer Plates**

The welds need to be designed only for the wind and seismic loading, or for any unbalanced continuity loading.

In addition to transferring the forces to the flanges of the column, the stiffeners perform another important function. Without the stiffeners, the stability of the bottom chord of the joist or Joist Girder becomes a concern. Vulcraft checks the bottom chord for the compression forces specified by the designer. The long stabilizer plate introduces a weak link at the end of the bottom chord which could allow a hinge to occur at the web of the column, thus significantly reducing the buckling capacity of the bottom chord. The stiffeners on the stabilizer plate provide a significantly better situation for bracing the bottom chord.

As an alternate the authors have conducted stability studies relative to the design of the stabilizer plate for the situation shown in Figure 7.3.4. Based on these studies, it is recommended that the stabilizer plate design be based on an allowable stress of 15 ksi. This allowable stress is based on a length of plate of 8 inches. That is, the distance from the web of the column to the attachment of the bottom chords.

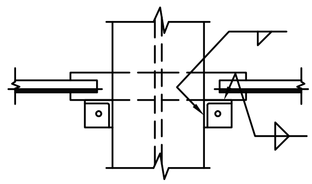


**Fig. 7.3.4 Angle Reinforcement to Stabilizer Plates**

The design of a bottom chord system is provided in Example 7.3.1.

### Example 7.3.1 Bottom Chord Force Transfer

Design the bottom chord connection shown in Figure 7.3.5: To minimize welds the stabilizers are slotted.

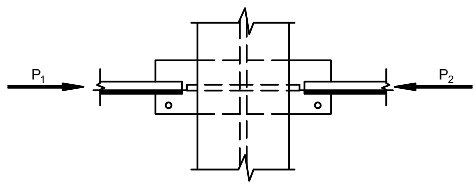



**Fig. 7.3.5 Bottom Chord Connection**

### Given:

Load Cases:
Case 1: $P_1$ = 100 kips $P_2$ = 100 kips
Case 2: $P_1$ = 125 kips $P_2$ = -75 kips
Case 3: $P_1$ = 75 kips $P_2$ = 125 kips

The column is a W10x33:

Flange width = 7.96 in.

$F_y = 50$ ksi

A36 steel is used for all plate material.
Joist Girder bottom chord 2L3x3x3/8
Stabilizer plate:
Thickness = 7/8 in.
Height = 6 in.

**ASD Solution:**

*Weld Requirements and stiffener strength:*

Chord to Stabilizer Plate:
$P_a$ = 125 kips

Try 4 welds 5/16 in.:
Length req'd = $P_a$/weld strength
Length req'd = 125 kips/[(4)(0.707)(21 ksi)(0.3125 in.)] = 6.73 in.

Use 4-5⁄16 in. Fillet welds 7 in. long

Also use 5⁄16 in. fillet welds to connect the stabilizer to the column flanges.

Design of the stiffeners from the stabilizer plates to the column flanges:

The maximum force to be transferred by the stiffeners into the column flanges is 50 kips (Cases 2 and 3). If the force is assumed to be distributed equally, each stiffener must transfer 12.5 kips.

See Figure 7.3.5. Neglect any strengthening effect of the stabilizer plate attachment to the column web.

The stiffeners can be designed as shear elements. Using ½-inch plate material (the stiffener thickness should be approximately the same thickness as the bottom chord of the girder). Use 4 inches of 5⁄16 in. fillet weld to connect the stiffeners to the stabilizer and to the column flanges.

Shear strength of stiffeners:

$V_n = 0.6F_yA_wC_{v1}$ AISC Eq. (G2-1)

where

$A_w$ = area of stiffener

$C_{v1} = 1.0$

$V_n = 0.6(36 \text{ ksi})(4 \text{ in.})(0.5 \text{ in.})(1.0) = 43.2 \text{ kips}$

$V_n/\Omega = V_n/1.67 = 25.9 \text{ kips}$

25.9 kips > 12.5 kips **o.k.**

Weld shear strength:

$r_n/\Omega = (0.928)D = (0.928 \text{ kips/in.})(5) = 4.64 \text{ kips/in.}$ AISC Manual Eq. (8-20b)

$R_n/\Omega = (4 \text{ in.})(4.64 \text{ kips/in.}) = 18.6 \text{ kips}$

18.6 kips > 12.5 kips **o.k.**

*Check the stabilizer plates:*

The stabilizer plate and stabilizer to chord welds must be designed for a 125-kip force (Cases 2 and 3).

Whitmore width of stabilizer:

Width above the bottom chord = 1in. (by observation)

Width below the bottom chord ≈ (weld length + $b_f/2$)tan 30°

= 7 in. + 7.96 in./2 = 11.0 in.

Whitmore width = 8 in.

$P_n = F_{cr}A_g$ AISC Eq. (E3-1)

$P_c = (22 \text{ ksi})(7/8 \text{ in.})(8 \text{ in.}) = 154 \text{ kips}$

154 kips > 125 kips **o.k.**

Attachment to HSS columns is similar in nature to attachment to the web of W shapes in that the HSS wall has a very limited capacity to resist the concentrated load delivered by the stabilizer plate. The details shown in Figures 7.3.6 and 7.3.7 can be used to reinforce thin HSS walls.

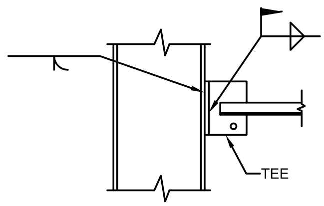



**Fig. 7.3.6 T-Reinforcement**

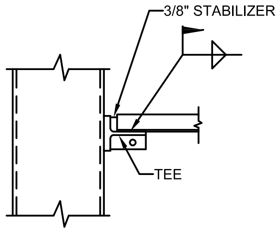



**Fig. 7.3.7 T-Reinforcement with Stabilizer**

Stabilizer plates can be passed thru the HSS walls when large continuity forces exist. To transfer moments into the HSS when the stabilizer must pass thru the HSS, the angle below the stabilizer can be added. If additional reinforcement is required to transfer moments into the HSS, the designer should question the feasibility of using HSS columns.

The designer should carefully consider the cost implications of modifying the Basic Connection. The use of continuity ties, stiffeners, E members and special seats can add significant costs to any project. As mentioned in Chapter 4, it is generally cost efficient to avoid moment frames if possible. The designer should evaluate the cost of providing column brackets and field welded moment plates in lieu of modifying the Basic Connection. Section 7.5 contains information on the design of moment plate connections.

## 7.4 TYPICAL CONNECTIONS

The details provided in Figures 7.4.1 to 7.4.4 are presented as typical connections to provide moment resistance using the Basic Connection.

### Example 7.4.1 Determining a Joist Girder Maximum Chord Force

Determine the maximum permissible Joist Girder bottom chord force for the detail shown in Figure 7.4.1 using a W8x24 column. The bottom chord consists of 2L3x3 in.

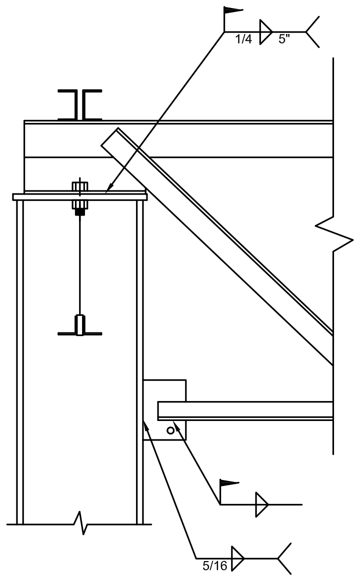



**Figure 7.4.1 Typical Connection 7.4.1 (Joist Girder with Special Seat)**

The bottom chord of the Joist Girder must be attached to the stabilizer plate to resist the 30 kip force. In addition, the stabilizer plate must transfer this same force to the column. Stabilizer plates are normally sized based on a ¾ in. thickness of plate. Using a ¾ in. plate allows the plate to fit between the bottom chord angles allowing fillet welds to be made to the heels and toes of the chord angles. For economy the stabilizer plates can usually be connected to the column using only fillet welds. If moment reversal exists, the stabilizer plate must be welded to the column web to also resist a tensile force. The specifying professional must specify a minimum thickness Joist Girder bottom chord to accommodate the required weld size. As is required for the top chord, Vulcraft has the responsibility to check the bottom chord angles for shear lag. Case 4 from AISC Specification Table D3.1 is applicable for this check. For reference, the shear lag factor is calculated for the bottom chord based on the angle size. Shear lag factors greater than 0.92 do not affect the Joist Girders.

Longer length fillet welds will reduce shear lag effects.

**Given:**

Top chord and bottom Joist Girder force = 30 kips

Design the bottom chord connection to the W8x24 column

Use ASD

**ASD Solution:**

*Stabilizer Checks:*

Determine the weld between the bottom chord and the stabilizer ($\Omega = 2.0$)

There are four welds:

Use 3⁄16 in. fillet welds

Required length = $P_a/r_n/\Omega$

$r_n/\Omega = (4)(0.928)(3) = 11.1$ kips/ in. AISC Manual Eq. (8-20b)

Required length per weld = $P_a/r_n/\Omega = 30$ kips/11.1 kips/in. = 2.7 in.

Use a weld length = 6 in.

The weld length should be two times the bottom chord leg height to avoid a shear lag reduction for the stabilizer.

*Stabilizer Plate: Limit State of Yielding*

$P_a \leq R_n/\Omega$ AISC Eq. (D2-1)

If the bottom chord weld starts at the end of the stabilizer the Whitmore width equals (2)(tan30°)(Weld Length) + the bottom chord leg height.

Conservatively use 3 in. as the Whitmore width.

$R_n/\Omega = t_s h_s F_y/\Omega$

$t_s$ = stabilizer thickness

$h_s$ = stabilizer effective width based on the Whitmore width (AISC Manual Section 9-3)

$R_n/\Omega = (0.75 \text{ in.})(3.0 \text{ in.})(36 \text{ ksi})/1.67 = 48.5$ kips

48.5 kips > 30 kips **o.k.**

*Stabilizer Plate: Limit State of Compression Buckling:*

Consider the same Whitmore width used for the yielding calculation. The unbraced length of the plate from the end of the Joist Girder chord to the face of the columns is 3 inches. Use $K$ =1.2.

$Kl/r = 1.2(3 \text{ in.})/(0.22 \text{ in.}) = 16.4$

where

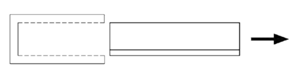
$$r = \frac{t_s}{\sqrt{12}} = \frac{0.75 \text{ in.}}{\sqrt{12}} = 0.22 \text{ in.}$$

Refer to AISC Section J4.4: Connecting elements with $Kl/r$ less than or equal to 25 have the same capacity as the yielding limit state.

$R_n/\Omega = 48.5$ kips

*Stabilizer Plate - Block Shear Rupture Strength*

a. Block shear plane 1:



$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$

$A_{nv}$ = *net area* subject to shear, in.$^2$

$A_{nt}$ = *net area* subject to tension, in.$^2$

$F_y$ =36 ksi

$F_u$ = 58 ksi

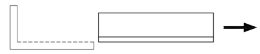
$U_{bs}$ = 1.0

Considering only one shear plane:

$R_n = (0.60)(58 \text{ ksi})(6.0 \text{ in.})(0.75 \text{ in.}) = 157$ kips

Further investigation is not required. Block shear will not control.

b. Block shear plane 2:



Checked as in (a)

By inspection block shear does not control.

Weld strength between the stabilizer and the column.

There are two welds:

Use 5/16 in. welds

$R_n/\Omega = (2)(0.928)(5) = 9.28$ kips/in.

Allowable strength = (2)(6 in.)(9.28 kips/in.) = 111 kips > 30 kips **o.k.**

The directional strength increase for the weld of 1.5 was not used.

Some designers prefer to provide enough weld to develop the full strength of the stabilizer.

*Column Checks:*

*Web Local Yielding:*

$$R_n = F_{yw} t_w (5k + l_b) \qquad \text{AISC Eq. (J10-2)}$$

where:

$F_{yw}$ = specified minimum yield stress of the web material = 50 ksi

$k$ = distance from outer face of the flange to the web toe of the fillet = 0.794 in.

$l_b$ = length of bearing (not less than k for end beam reactions) = 6.0 in.

$t_w$ = thickness of web = 0.245 in.

$$R_n = F_{yw} t_w (5k + l_b)$$

$$= (50 \text{ ksi})(0.245 \text{ in.})[5(0.794 \text{ in.}) + 6.0 \text{ in.}] = 122 \text{ kips}$$

$R_n/\Omega$ = 122 kips/1.50 = 81.3 kips > 30 kips **o.k.**

*Web Local Crippling:*

$$R_n = 0.80 t_w^2 \left[ 1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f \qquad \text{AISC Eq. (J10-4)}$$

$$R_n = (0.80)(0.245 \text{ in.})^2 \left[ 1 + 3\left(\frac{6.0 \text{ in.}}{8.06 \text{ in.}}\right)\left(\frac{0.245 \text{ in.}}{0.400 \text{ in.}}\right)^{1.5} \right] \sqrt{\frac{(29{,}000 \text{ ksi})(50 \text{ ksi})(0.400 \text{ in.})}{0.245 \text{ in.}}} (1.0)$$

$$R_n = 153 \text{ kips}$$

$R_n/\Omega$ = 153 kips/2.0 = 76.5 kips > 30 kips **o.k.**

**LRFD Solution:**

**Given:**

Top chord and bottom Joist Girder force = 45 kips (Factored load).

Design the bottom chord connection to the W8x24 column.

*Stabilizer Checks:*

Determine the weld between the bottom chord and the stabilizer ($\phi = 0.75$)

There are four welds:

Use 3⁄16 in. fillet welds. D = 3

Required length per weld = $P_r/\phi R_n$

$\phi r_n = (4)(1.392)(3) = 16.7$ kips/ in. AISC Manual Eq. (8-20b)

Required length per weld = $P_u/R_n/\Omega$ = 45 kips/16.7 kips/in. = 2.7 in.

Use a weld length = 6 in.

The weld length should be two times the bottom chord leg height to minimize shear lag reduction for the stabilizer.

*Stabilizer tension yielding* ($\phi$ =0.90).

$P_u \leq \phi R_n$ AISC Eq. (D2-1)

$\phi R_n = \phi t_s h_s F_y$

If the bottom chord weld starts at the end of the stabilizer the Whitmore width equals (2)(tan30°)(Weld Length) + the bottom chord leg height.

Conservatively use 3 in. as the Whitmore width.

$\phi R_n$ = (0.90)(0.75 in.)(3.0 in.)(36 ksi) = 72.9 kips

$t_s$ = stabilizer thickness

$h_s$ = stabilizer effective width based on the Whitmore width (AISC Manual Section 9-3)

72.9 kips > 45 kips **o.k.**

*Stabilizer Plate: Limit State of Compression Buckling:*

Consider the same Whitmore width used for the yielding calculation. The unbraced length of the plate from the end of the Joist Girder chord to the face of the columns is 3 inches (use $K$ =1.2).

$Kl/r$=1.2(3 in.)/(0.22 in.) =16.4

where

$$r = \frac{t_s}{\sqrt{12}} = \frac{0.75 \text{ in.}}{\sqrt{12}} = 0.22 \text{ in.}$$

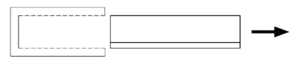
Refer to AISC Section J4.4: Connecting elements with $Kl/r$ less than or equal to 25 have the same capacity as the yielding limit state.

72.9 kips > 45 kips **o.k.**

*Stabilizer block shear rupture strength* ($\phi = 0.75$) AISC Eq. (J4.3)

Stabilizer Plate - Block Shear Rupture Strength

a. Block shear plane 1:



$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$

$A_{nv}$ = *net area* subject to shear, in.$^2$

$A_{nt}$ = *net area* subject to tension, in.$^2$

$F_y$ =36 ksi

$F_u$ = 58 ksi

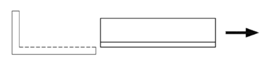
$U_{bs}$ = 1.0

$R_n$ = 2(0.60)(58 ksi)(6.0 in.)(0.75 in.)+ 1.0(58 ksi)(3.0 in.)(0.75 in.) = 444 kips

$\le$ 2(0.60)(36 ksi)(6.0 in.)(0.75 in.) + 1.0(58 ksi)(3.0 in.)(0.75 in.) = 325 kips

$\phi R_n$ = 0.75(325 kips) = 244 kips

b. Block shear plane 2:



Refer to (a)

By inspection block shear does not control.

Weld strength between the stabilizer and the column.

There are two welds:

Use 5/16 in. welds

$\phi r_n$ = (2)(1.392)D

$\phi R_n$ = (2)(1.392)(5) = 13.9 kips/in.

Design strength = (2)(6 in.)(13.9 kips/in.) = 167 kips > 45 kips **o.k.**

The directional strength increase for the weld of 1.5 was not used.

Some designers prefer to provide enough weld to develop the full strength of the stabilizer.

*Column Checks:*

*Web Local Yielding:*

$R_n = F_{yw}t_w(5k + l_b)$ AISC Eq. (J10-2)

where:

$F_{yw}$ = specified minimum yield stress of the web material = 50 ksi

$k$ = distance from outer face of the flange to the web toe of the fillet = 0.794 in.

$l_b$ = length of bearing (not less than k for end beam reactions) = 6.0 in.

$t_w$ = thickness of web = 0.245 in.

$R_n = F_{yw}t_w(5k + l_b)$

= (50 ksi)(0.245 in.)[5(0.794 in.) + 6.0 in.] = 122 kips

$\phi R_n$ = (1.0)(122 kips) = 122 kips > 45 kips **o.k.**

*Web Local Crippling:*

$$R_n = 0.80t_w^2\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f \qquad \text{AISC Eq. (J10-4)}$$

$$R_n = (0.80)(0.245\text{ in.})^2\left[1+3\left(\frac{6.0\text{ in.}}{8.06\text{ in.}}\right)\left(\frac{0.245\text{ in.}}{0.400\text{ in.}}\right)^{1.5}\right]\sqrt{\frac{(29{,}000\text{ ksi})(50\text{ ksi})(0.400\text{ in.})}{0.245\text{ in.}}}(1.0)$$

$$R_n = 153\text{ kips}$$

$\phi R_n$ = (0.75)(153 kips) = 115 kips > 45 kips **o.k.**

### Example 7.4.2- Typical Connection (Joists Bearing on Joist Girder Seat)

Determine the maximum joist chord force for the connection shown in Figure 7.4.2.

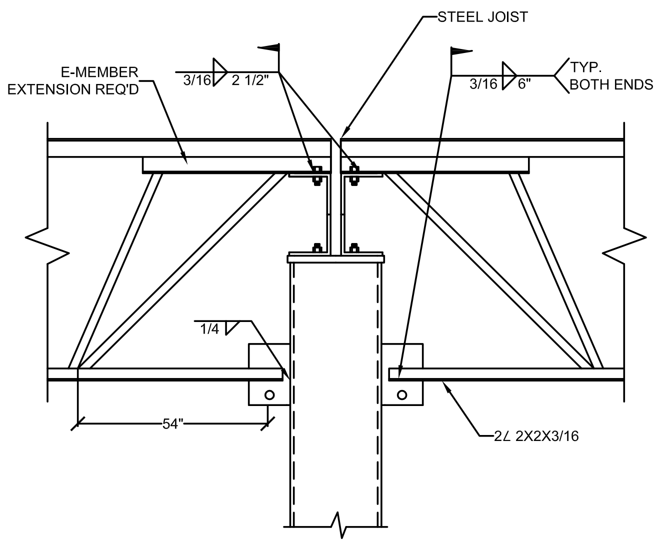



**Fig. 7.4.2 Typical Connection (Joists Bearing on Joist Girder Seat)**

**Given:**

Two Joist Girders frame into the column top.

The column is an HSS 8x8x3/8, ASTM 1085, $F_y$ =50 ksi.

HSS wall thickness, $t$ = 0.375 in.

(Note: For ASTM 1085 HSS, the nominal wall thickness can be used in calculations.)

The unbraced length of the bottom chord equals 54 inches.

Joist depth = 24 in.

The column supports an axial load of 50 kips.

Each joist has an end moment of 125 kip-in.

**ASD Solution:**

The ASD rollover strength for a Joist Girder seat with 7⁄16 in. seat angles is 5.81 kips. (See Example 7.1.1). Since there are two Joist Girders the maximum top chord force for each joist equals 5.81 kips.

*Weld strength:*

The available weld strength connecting each joist to each Joist Girder seat equals

$R_n/\Omega$ = (3/16 in.)(0.707)(21 ksi)(2)(2.5 in.) = 14.0 kips

14.0 kips > 5.81 kips, therefore the welds are adequate to transfer the 5.81 kips

*Bottom chord connection:*

*Determine the available tension strength of* 2L2x2x3/16:

Angle Properties:

$A$ = 1.44 in.$^2$

$T_n = AF_y$ = (1.44 in.$^2$)(50 ksi) = 72.0 kips

$T_n/\Omega$ = 72.0/1.67 = 43.1 kips

*Determine the available compression strength of* 2L2x2x3/16:

The available compression strength of the bottom chord angles is determined from the AISC column equation E3-1.

$r_x$ = 0.612 in.

$r_y$ = 1.12 in.

S = 24.9 in.

$$P_n = F_{cr}A_g \qquad \text{AISC Eq. (E3-1)}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} = \frac{\pi^2(29{,}000 \text{ ksi})}{\left(\frac{35.1 \text{ in.}}{0.612 \text{ in.}}\right)^2} = 87.0 \text{ ksi} \qquad \text{AISC Eq. (E3-4)}$$

where $K$ is taken as 0.65

$$L_c = KL = (0.65)(54 \text{ in.}) = 35.1 \text{ in.}$$

$$\frac{F_y}{F_e} = \frac{50 \text{ ksi}}{87.0 \text{ ksi}} = 0.57$$

$$\text{when } \frac{F_y}{F_e} \leq 2.25$$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y = \left(0.658^{0.57}\right)(50 \text{ ksi}) = 39.4 \text{ ksi} \qquad \text{AISC Eq. (E3-2)}$$

$$P_n = F_{cr}A_g = (39.4 \text{ ksi})\left(1.44 \text{ in.}^2\right) = 56.7 \text{ kips}$$

$$\frac{P_n}{\Omega} = \frac{56.7 \text{ kips}}{1.67} = 34.0 \text{ kips}$$

Strength of the (2) 6 in. 3/16 in. bottom chord fillet welds from the chord angles to the connection angle:

$R_n/\Omega$ = (2)(21 ksi)(6 in.)(3/16 in.)(0.707) = 32.4 kips.

*Determine the available force that can be delivered to the face of the HSS:*

From the 2010 AISC Specifications:

Note: The 2010 AISC Specifications provide a more direct solution than do the AISC 2016 Specifications, both provide the same results.

Limit state: HSS Shear Yielding (Punching) AISC 2010 Eq. (K1-12)

$$R_n \sin\theta = \frac{F_y t^2}{1-\frac{t_p}{B}}\left[\frac{2l_b}{B} + 4\sqrt{1-\frac{t_p}{B}}Q_f\right] = \frac{(50\text{ ksi})(0.375\text{ in.})^2}{1-\frac{0.75\text{ in.}}{8\text{ in.}}}\left[\frac{2(8\text{ in.})}{8\text{ in.}} + 4\sqrt{1-\frac{0.75\text{ in.}}{8\text{ in.}}}(0.94)\right] = 45.0\text{ kips}$$

$R_n / \Omega = 45\text{ kips} / 1.5 = 30\text{ kips}$

where

$B$ = overall width of the HSS, in. = 8 in.

$t$ = HSS wall thickness, in. = 0.375 in.

$t_p =$ stabilizer plate thickness, in. = 0.75 in.

$l_p = 8$ in.

$\sin\theta = 1.0$

$Q_f = \sqrt{1-U^2} = \sqrt{1-0.33^2} = 0.94$ AISC 2010 Eq. (K1-17)

$$U = \left|\frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S}\right| = \left|\frac{50\text{ kips}}{(30\text{ ksi})(10.4\text{ in.}^2)} + \frac{125\text{ kip-in.}}{(30\text{ ksi})(24.9\text{ in.}^3)}\right| = 0.33$$ AISC 2010 Eq. (K1-6)

where

$F_c = 0.6F_y = 30$ ksi (ASD)

$P_{ro} = 50$ kips (Given)

$M_{ro} = 125$ kip-in. (Given)

Limit state: Local Yielding of HSS sidewalls, when $B = 1.0$

$R_n = 2F_y t(5k + l_b)$ $\Omega$ = 1.50 ASD AISC 2010 Eq. (K1-9)

$R_n = 2(50\text{ ksi})\left[5(0.375\text{ in.}) + 0.25\text{ in.}\right] = 213$ kips

$R_n / \Omega = 213$ kips/1.50 = 142 kips

where

$k =$ outside corner radius of HSS $\geq 1.5t =$ 0.375 in.

$l_b =$ plate thickness, in. = 0.25 in.

Limit state: Local Crippling of HSS walls

$$R_n = 1.6t^2\left(1 + \frac{3l_b}{H-3t}\right)\sqrt{EF_y}Q_f \quad \Omega = 2.0\text{ ASD}$$ AISC 2010 Eq. (K1-10)

$$R_n = 1.6(0.25\text{ in.})^2\left(1 + \frac{3(0.25\text{ in.})}{8\text{ in.} - 3(0.25\text{ in.})}\right)\sqrt{(29{,}000\text{ ksi})(50\text{ ksi})}(0.94) = 125\text{ kips}$$

$R_n = 125$ kips/2.0 = 62.5 kips

where

$H$ = HSS width = 8 in.

By observation block shear will not control on the stabilizer plate.

*Determine the maximum chord forces:*

Top chord: = 5.81 kips (Controlled by rollover of the Joist Girder seat).

Bottom chord: 30.0 kips (Controlled by HSS Shear Yielding).

*Maximum end moment:*

The maximum joist end moment based on the Joist Girder seat rollover equals 5.81 kips times (joist depth - joist seat height) = (5.81 kips)(24 in.-2.5 in.) = 125 kip-in.

### Example 7.4.3- Allowable Joist Moment (Joist with TCX)

Determine the allowable joist moment (ASD) for the connection shown in Figure 7.4.3. Two Joist Girders frame into the column. The column is a HSS 8x8x3/8. The difference between the connection detail in Figure 7.4.3 from the detail in Figure 7.4.2 is that only one joist is framing into the two Joist Girder seats, thus the ASD rollover strength is 2(5.81 kips) = 11.6 kips.

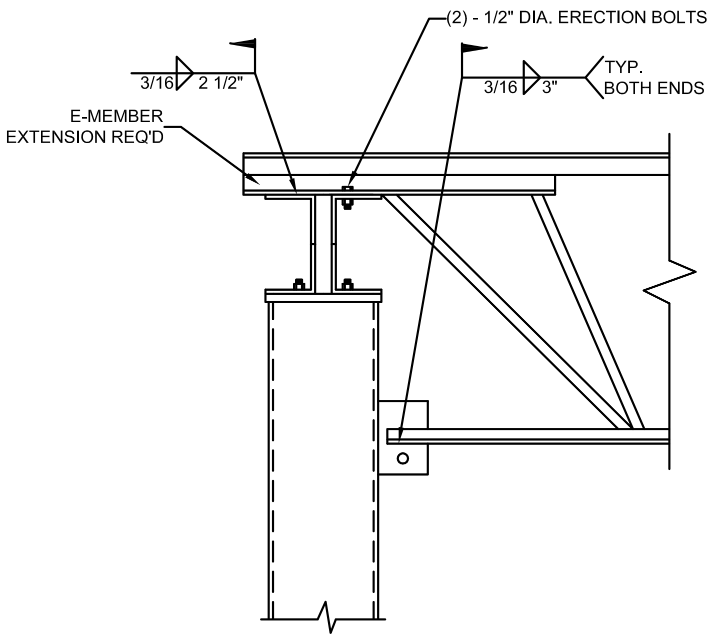



**Fig. 7.4.3 Typical Connection 7.4.3 (Joist with TCX)**

**Given:**

The stabilizer plate is A36 steel.

HSS:

ASTM 1085

$F_y$ = 50 ksi

HSS wall thickness, $t$ = 0.25 in.

Examining the ASD results from the Typical Connection 7.4.2:

1. The available weld strength connecting each joist to each Joist Girder seat equals 14.0 kips
2. The available tension strength of 2L2x2x3/16 of the bottom chord angles equals 43.1 kips
3. The available compression strength of the bottom chord angles equals 34 kips
4. Strength of the (2) 6 in. 3⁄16 in. bottom chord fillet welds attached to the angle equals 32.4 kips
5. The available force that can be delivered to the face of the HSS equals 30.0 kips

The maximum joist end moment based on the Joist Girder seat rollover equals 11.8 kips (ASD) times the centroid to centroid distance of the joist chords for a single joist framing to a column. The joist top chord must have adequate strength to transfer the 11.8 kips from the top of the Joist Girder seat to the centroid of the top chord. When two joists frame to the column only 5.9 kips must be transferred.

### Example 7.4.4- Typical Connection (Joist with TCX on Girder with Stiffened Seat)

Determine the maximum available joist chord force for (ASD and LRFD) for the connection shown in Figure 7.4.4.

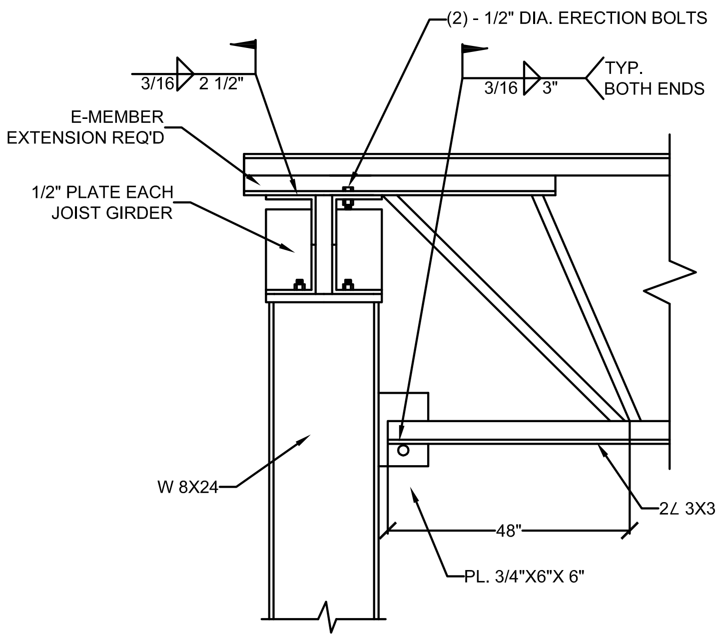



**Fig. 7.4.4 Example 7.4.4 (Joist with TCX on Girder with Stiffened Seat)**

**Given:**

The column is a W8X24.

Joist bottom chord consists of 3x3 in. angles.

Stabilizer plate height equals 6 in.

Stabilizer plate thickness equal ¾ in.

Joist Girder seats are stiffened thus:

The ASD rollover strength is 2(14 kips) = 28 kips.

The LRFD rollover strength equals 2(21 kips) = 42 kips.

**ASD Solution:**

*Joist top chord weld strength:*

$R_n/\Omega$ = (3/16 in.)(0.6)(70 ksi)(0.707)(5 in.)/2.0 = 13.9 kips.

*Bottom chord connection to the column.*

Stabilizer Checks:

Weld - Joist Bottom Chord to Stabilizer Plate ($\Omega$ = 2.0)

There are four 3/16 in. fillet welds.

Total length of welds = 12 in.

$R_n/\Omega$ = (12 in.)(0.928)D = (12 in.)(0.928)(3) =33.4 kips AISC Manual Eq. (8-20b)

Stabilizer yielding ($\Omega$ = 1.67).

*Stabilizer Plate: Limit State of Tension Yielding*

$P_a \le R_n/\Omega$ AISC Eq. (D2-1)

where

$t_s$ = stabilizer thickness

$h_s$ = stabilizer effective width based on the Whitmore width (AISC Manual Section 9-3).

If the bottom chord weld starts at the end of the stabilizer the Whitmore width equals (2)(tan30$^{o}$)(Weld Length) + the bottom chord leg height.

Conservatively use 3 in. as the Whitmore width.

$R_n/\Omega = t_s h_s F_y/\Omega$

$R_n/\Omega$ = (0.75 in.)(3.0)(36)/1.67 = 48.5 kips

*Stabilizer Plate: Limit State of Compression Buckling:*

Consider the same Whitmore width used for the yielding calculation. The unbraced length of the plate from the end of the Joist Girder chord to the face of the columns is 3 inches. Use $K$ =1.2.

$Kl/r$ =1.2(3 in.)/(0.22 in.) =16.4

where

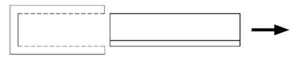
$$r = \frac{t_s}{\sqrt{12}} = \frac{0.75 \text{ in.}}{\sqrt{12}} = 0.22 \text{ in.}$$

Refer to AISC Section J4.4: Connecting elements with $Kl/r$ less than or equal to 25 have the same capacity as the yielding limit state.

Stabilizer block shear rupture strength ($\Omega$ = 2.0) AISC Eq. (J4.3)

Stabilizer Plate - Block Shear Rupture Strength

a. Block shear plane 1:



$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$

$A_{nv}$ = *net area* subject to shear, in.$^2$

$A_{nt}$ = *net area* subject to tension, in.$^2$

$F_y$ =36 ksi

$F_u$ = 58 ksi

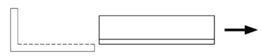
$U_{bs}$ = 1.0

$R_n$ = 2(0.60)(58 ksi)(6.0 in.)(0.75 in.)+ 1.0(58 ksi)(3.0 in.)(0.75 in.) = 444 kips

$\leq$ 2(0.60)(36 ksi)(6.0 in.)(0.75 in.) + 1.0(58 ksi)(3.0 in.)(0.75 in.) = 325 kips

$R_n/\Omega$ = (325 kips)/2.0 = 163 kips

b. Block shear plane 2:



Refer to (a)

By inspection block shear does not control.

Weld strength between the stabilizer and the column.

There are two 5/16 in. welds:

$R_n/\Omega$ = (2)(0.928)D = kips/in.

$R_n/\Omega$ = (2)(0.928)(5) = 9.28 kips/in.

Allowable strength = (2)(6 in.)(9.28 kips/in.) = 111 kips

The directional strength increase for the weld of 1.5 was not used.

Some designers prefer to provide enough weld to develop the full strength of the stabilizer.

*Column Checks:*

*Web Local Yielding:*

$R_n = F_{yw}t_w(5k + l_b)$ AISC Eq. (J10-2)

where

$F_{yw}$ = specified minimum yield stress of the web material = 50 ksi

$k$ = distance from outer face of the flange to the web toe of the fillet = 0.794 in.

$l_b$ = length of bearing (not less than $k$ for end beam reactions) = 6.0 in.

$t_w$ = thickness of web = 0.245 in.

$R_n = F_{yw}t_w(5k + l_b)$

= (50 ksi)(0.245 in.)[5(0.794 in.) + 6.0 in.] = 122 kips

$R_n/\Omega$ = 122 kips/1.50 = 81.3 kips

*Web Local Crippling:*

$$R_n = 0.80t_w^2\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f \qquad \text{AISC Eq. (J10-4)}$$

$$R_n = (0.80)(0.245\text{ in.})^2\left[1+3\left(\frac{6.0\text{ in.}}{8.06\text{ in.}}\right)\left(\frac{0.245\text{ in.}}{0.400\text{ in.}}\right)^{1.5}\right]\sqrt{\frac{(29{,}000\text{ ksi})(50\text{ ksi})(0.400\text{ in.})}{0.245\text{ in.}}}(1.0)$$

$$R_n = 153\text{ kips}$$

$R_n/\Omega$ = 153 kips/2.0 = 76.5 kips.

The strength of the connection is limited by the weld from the joist seat to the top of the Joist Girders. To achieve full strength (28 kips) the weld size or length must be increased.

**LRFD Solution:**

*Joist top chord weld strength:*

$\phi R_n$ = (4)(1.392)D = (5 in.)(1.392)(3) = 20.9 kips AISC Manual Eq. (8-20b)

*Bottom chord connection to the column.*

Stabilizer Checks:

Weld - Joist Bottom Chord to Stabilizer Plate ($\phi$ = 0.75)

There are four 3/16 in. fillet welds.

Total length of welds = 12 in.

$\phi r_n$ = (4)(1.392)D = 16.7 kips/in.

f$R_n$ = (0.75)(16.7 kips/in.)(12 in.) = 150 kips AISC Manual Eq. (8-20b)

Stabilizer yielding ($\phi$ =0.90).

$\phi R_n = \phi t_s h_s F_y$

where

$t_s$ = stabilizer thickness

$h_s$ = stabilizer effective width based on the Whitmore width (AISC Manual Section 9-3)

If the bottom chord weld starts at the end of the stabilizer the Whitmore width equals (2)(tan30°)(Weld Length) + the bottom chord leg height.

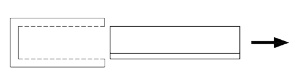
Conservatively use 3 in. as the Whitmore width.

$\phi R_n$= (0.90)(0.75 in.)(3.0)(36) = 72.9 kips

Stabilizer block shear rupture strength ($\phi$ = 0.75) AISC Eq. (J4.3)

Stabilizer Plate - Block Shear Rupture Strength

a. Block shear plane 1:



$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.60F_yA_{gv} + U_{bs}F_uA_{nt}$

$A_{nv}$ = *net area* subject to shear, in.$^2$

$A_{nt}$ = *net area* subject to tension, in.$^2$

$F_y$ =36 ksi

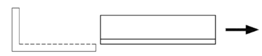
$F_u$ = 58 ksi

$U_{bs}$ = 1.0

$R_n$ = (0.60)(58 ksi)(6.0 in.)(0.75 in.) = 157 kips

$\phi R_n$ = 0.75(157 kips) = 118 kips

b. Block shear plane 2:



Refer to (a)

By inspection block shear does not control.

Weld strength between the stabilizer and the column.

There are two ⁵⁄₁₆ in. welds:

$\phi R_n$ = (2)(1.392)D

$\phi R_n$ = (2)(1.392)(5) = 13.9 kips/in.

Design strength = (2)(6 in.)(13.9 kips/in.) = 167 kips

The directional weld strength increase of 1.5 was not used.

*Column Checks:*

*Web Local Yielding:*

$R_n = F_{yw} t_w (5k + l_b)$
AISC Eq. (J10-2)

where

$F_{yw}$ = specified minimum yield stress of the web material = 50 ksi

$k$ = distance from outer face of the flange to the web toe of the fillet = 0.794 in.

$l_b$ = length of bearing (not less than k for end beam reactions) = 6.0 in.

$t_w$ = thickness of web = 0.245 in.

$R_n = F_{yw} t_w (5k + l_b)$

= (50 ksi)(0.245 in.)[5(0.794 in.) + 6.0 in.] = 122 kips

$\phi R_n$ = (1.0)(122 kips) = 122 kips

*Web Local Crippling:*

$$R_n = 0.80 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}Q_f \qquad \text{AISC Eq. (J10-4)}$$

$$R_n = (0.80)(0.245\text{ in.})^2\left[1 + 3\left(\frac{6.0\text{ in.}}{8.06\text{ in.}}\right)\left(\frac{0.245\text{ in.}}{0.400\text{ in.}}\right)^{1.5}\right]\sqrt{\frac{(29{,}000\text{ ksi})(50\text{ ksi})(0.400\text{ in.})}{0.245\text{ in.}}}(1.0)$$

$R_n = 153$ kips

$\phi R_n$ = (0.75)(153 kips) = 115 kips

The strength of the connection is limited by the weld from the joist seat to the top of the Joist Girders. To achieve full strength (42 kips) the weld size or length must be increased.

## 7.5 MOMENT PLATE CONNECTIONS

The most efficient detail to transfer moment into a joist or girder is one in which a top moment plate is welded to the top chord of the joist or girder and to the column. The bottom chord is connected to the column in any of the ways discussed in Section 7.3. The use of the top plate significantly reduces the bending in the top chord. Details using the moment plate are shown in Figures 7.5.1 and 7.5.2.

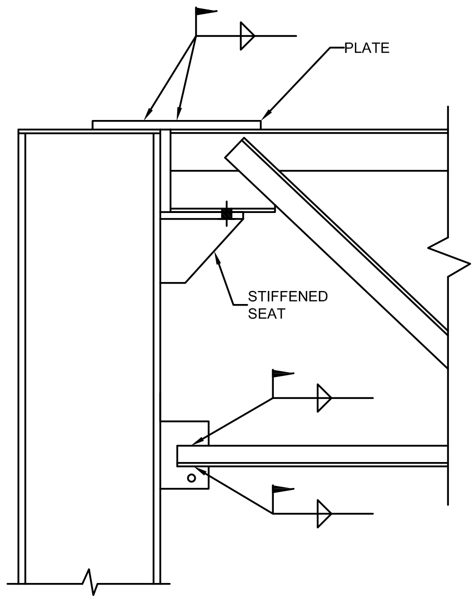



**Fig. 7.5.1 Roof Moment Plate**

The connection is ideally suited for floor girders and floor joists. To design the moment plate, the specifying professional needs to determine the maximum moment at the end of the member in question and divide the moment by the joist or girder depth to determine the maximum force, P, in the top plate. The plate and its attachments to the column and joist or girder are designed using standard AISC Specification equations. Maximum plate width can be determined from Table 7.1.2 so that down hand fillet welds can be made to the top chord. The plate must be installed prior to decking. The bottom chord can be left unwelded to reduce the continuity moments until dead loads are applied. The moment plate and the stabilizer plate must be checked for load reversal, and the columns must be checked for stiffener requirements.

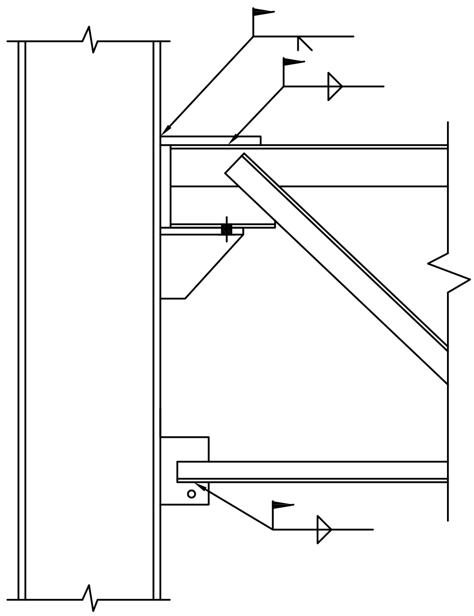


**Fig. 7.5.2 Floor Moment Plate**

The design of the seat supporting the joist or Joist Girder can be accomplished using the AISC Manual of Steel Construction tables for stiffened seats. For unstiffened seats, the tables cannot be used directly for Joist Girders. The angle bending capacity in the AISC Manual of Steel Construction tables is based upon the outstanding legs of the angles not exceeding 4 inches. Since the SJI Specification requires a minimum of 4 inches of bearing for Joist Girders, the outstanding leg will generally be 5 inches in length. The tables can be used to determine weld requirements for the vertical legs of the seat, but basic principles must be used to determine an unstiffened seat angle thickness. The AISC Steel Construction Manual tables are based on beam web thicknesses and not on joist or Joist Girder seats.

Reflected in Figure 7.5.3 is the reaction from a Joist Girder resting on an unstiffened seat.

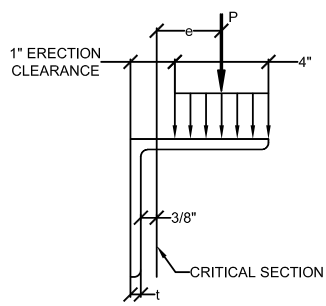



**Fig. 7.5.3 Unstiffened Seat**

If the vertical reaction of the girder is centered on the 4-inch bearing length, and the critical bending location in the angle is assumed to be 3/8 inches from the angle face.

$e = 3 \text{ in.} - t - \frac{3}{8} \text{ in.} = (2.625 \text{ in.}) - t$

The bending moment at the critical section:

$M_r = P_r e$

For a 5-inch bolt gage, the typical seat angle will be 8 inches wide ($b$ = 8 in.).

$M_n = ZF_y$

$Z \quad = bt^2/4$

$Z \quad = (8 \text{ in.})t^2/4 = (2 \text{ in.})t^2$

$M_n \; = (2 \text{ in.})t^2(36 \text{ ksi})$

$\quad\;\; = (72 \text{ kips/in.})t^2$

For ASD:

$M_n/\Omega = (72 \text{ kips/in.})t^2/1.67$

$\qquad\; = (43.1 \text{ kips/in.})t^2$

Equating:

$P_a e \quad = (43.1 \text{ kips/in.})t^2$

$(P_a)[(2.625 \text{ in.})\text{-}t] = (43.1 \text{ kips/in.})t^2$

To solve for a seat angle thickness for a given $P_a$, the quadratic equation must be solved for the thickness.

For LRFD:

$\phi M_n = 0.9(72 \text{ kips/in.})t^2$

$\qquad = (68.4 \text{ kips/in.})t^2$

Equating:

$P_u e = (68.4 \text{ kips/in.})t^2$

$(P_u)[(2.625 \text{ in.})\text{-}t] = (68.4 \text{ kips/in.})t^2$

To solve for a seat angle thickness for a given $P_{u,}$ the quadratic equation must be solved for the thickness.

On many occasions a stiffened seat or knife plate connections as shown in Figures 7.5.4 and 7.5.5 may be required.

Solutions for stiffened seats and knife plates can be found by downloading the Reference Manuals and the Spreadsheets from the SJI Website, www.steeljoist.org/product-category/design-tools/.

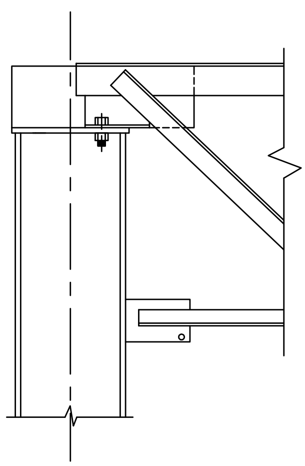


**Fig. 7.5.4 Knife Plate Roof Connection**

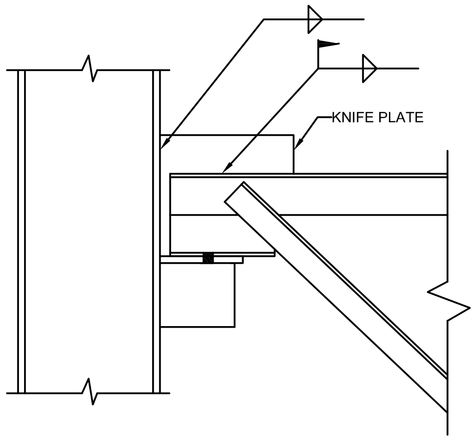



**Fig. 7.5.5 Knife Plate Floor Connection**

The check for column stiffeners is identical to the check at the stabilizer plate location. Vulcraft must fabricate the end seat to fit over the knife plate. The knife plate should be centered on the centroid of the top angles. It can extend down to the top of the seat. However, as discussed in Chapter 5 it should not be welded to the seat angle because the welds will interfere with the bearing of the joist or girder seat to the seat angle.

The specifying professional should check with Vulcraft prior to using the knife plate connection.

## Tools for the Design of Joist Girder Moment Connections

Many of the forgoing examples indicate the complexity for the design of moment connections. As discussed earlier, the "Basic Connection" is used for simple spans and is one where the Joist Girder seat rests on the column cap and the bottom chord angles slide over the stabilizer plate. Vulcraft designs the Joist Girder seat for the imposed vertical concentrated loads. The Basic Connection becomes a moment connection when the bottom chord of the Joist Girder is welded to the stabilizer plate. This connection has very limited moment capacity. Its strength is limited by bending stresses, induced in the top chord by load path eccentricities (Figure 7.5.5).

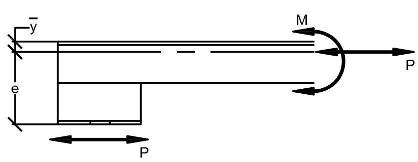



**Fig. 7.5.5 Chord Bending (use 7.1.2)**

Also pointed out earlier, is that the SJI has developed design tools that assist the specifying professional by making the design process more timely and complete. SJI provides six different spreadsheets to assist in the design of moment connections. Each can be used to calculate connection strength based on the necessary limit states. A reference manual is provided with each spreadsheet, explaining the calculations. As indicated the spreadsheets are:

1. Connection to the Strong Axis of Wide Flange Columns
2. Connection to the Strong Axis of Wide Flange Columns- Intermediate Levels
3. Connection to the Weak Axis of Wide Flange Columns
4. Connection to HSS Columns- Top Plate
5. Connection to HSS Columns- Knife Plate
6. Connection to Wide Flange Columns - Knife Plates

It is expected that the spreadsheet user be familiar with the SJI "Standard Specification for K-Series, LH-Series, DLH-Series and For Joist Girders" (SJI, 2015a), and the AISC Specifications. Before using the spreadsheets, the user should perform a structural analysis to determine that the column has the available strength to resist the applied loads.

### Design Checks for: Joist Girder Moment Connection to the Strong Axis of a Wide Flange Column

The following focuses on the Joist Girder Moment Connection to the Strong Axis of a Wide Flange Column with additional information available in the reference manuals and spreadsheets. The other Spreadsheet tools follow a similar presentation. In this detail, the Joist Girder vertical reaction is supported by a stiffened seat welded to the column flange. If the Joist Girder is modeled as a truss, the chord forces are obtained directly from the model. However, if Joist Girders are modeled as a beam elements, chord forces are determined by resolving end moments into force couples. Top chord force is transferred to the column by a top plate field welded to the chord and to the column cap plate. For Joist Girders framing to both sides of the column, the top plate is also used to transfer continuity forces from one Joist Girder to the other. Bottom chord force is transferred to the column via the stabilizer plates. Numerous limit states, which must be examined, are discussed below.

### Top Chord Connection

The required top plate strength is determined from the axial force in the top chord ($P_u = M_r/d_e$), where $M_r$ is the required end moment of the Joist Girder and $d_e$ is taken as the distance from the top of the Joist Girder to the half depth of the bottom chord leg. The required top plate area is $P_u/\phi F_y$ ($\phi = 0.90$). Plate length is based on the required length of fillet welds attaching the plate to the column cap plate and the top chord. Shear lag must be checked per the 2015 AISC Specification Table D3.1 "Shear Lag Factors for Connections to Tension Members." The Spreadsheet requires the top plate to the top chord weld length to be a minimum of two times the width of the top plate. This criterion minimizes the shear lag for the top plate. For the top plate connection to the column cap, the spreadsheet reduces the strength of the top plate for any shear lag. When the top chord is in tension, Vulcraft has the responsibility to check the top chord angles for shear lag. Case 2 from Table D3.1 is applicable for this check. For reference, the shear lag factor is calculated for the top chord based on the input of the angle leg size and the angle thickness. Shear lag factors greater than 0.92 do not have an affect on the Joist Girders. Providing longer fillet welds will reduce shear lag effects.

Many times the size of the Joist Girder chord angles is unknown when designing the connection. When the chords are subject to axial compression, a good estimate of the angle sizes can be obtained using Table 2-1 in the SJI Technical Digest 11, "Design of Lateral Load Resisting Frames Using Steel Joists and Joist Girders."(SJI, 2007a) The digest can be ordered from the SJI Website, www.steeljoist.org. From the structural analysis, the table can be entered with the chord force, unbraced length, to determine the angle size based on the avaiable strength. A representative sample of Table 2-1 is shown in Table 1.

**CHORD STRENGTHS (kips)**

| Angle Size | Unbraced Length | | | | Area |
|---|---|---|---|---|---|
| | L = 4 ft. | L = 5 ft. | L = 6 ft. | L = 7 ft. | in.$^2$ |
| 2L 6 x 6 x 1 | 939 | 911 | 879 | 842 | 22.0 |
| 2L 6 x 6 x 7/8 | 825 | 809 | 781 | 749 | 19.5 |
| 2L 6 x 6 x 3/4 | 702 | 694 | 679 | 651 | 16.9 |
| 2L 6 x 6 x 5/8 | 570 | 564 | 556 | 547 | 14.3 |
| 2L 6 x 6 x 9/16 | 499 | 494 | 487 | 480 | 12.9 |
| 2L 2-1/2 x 2-1/2 x 3/16 | 49 | 48 | 41 | 34 | 1.80 |

**Table 1 Available Strength (LRFD) for Chord Angle Sizes**

### Cap Plate to Column Weld

The weld of the cap plate to the column must also be determined since the top plate force must be transferred into the column web. The spreadsheet uses the column T-distance as the weld length. On occasion, the base metal strength may be less than the weld strength. If this occurs, the user can select a deeper column, a column with a thicker web and/or additional weld can be placed beyond the T distance.

### Column Web Shear

The nominal shear strength, $V_n$, is determined using the provisions of AISC Specification Section G2.1 ($\phi$ = 1.0 for rolled shapes when Eq. G2-1 controls, otherwise $\phi$ = 0.90). If the web does not have the available strength for shear, then it is generally economical to either select a deeper W shape or one with a thicker web. The column web shear yielding is checked at the Joist Girder top chord connection independent of the column web panel zone shear.

### Stiffened Seat Connection

The seat width can be determined from the minimum bearing length and, N, from the SJI Specifications (Table 5.4-3). The reaction is located N/2 from the interior edge of the seat.

Additionally, for the stiffened seat connection, the stiffener shall be finished to bear under the seat (AISC Steel Construction Manual, Table 10-8).

### Column Web Checks

The spreadsheet checks the following column web limit states:

1. Web Local Yielding
2. Web Crippling
3. Web Compression Buckling
4. Web Panel Zone Shear

The spreadsheet does not check the web panel zone shear below the bottom chord.

Web compression buckling is applicable when a pair of single-concentrated forces is applied at both flanges of a member. This condition does not exist at the exterior columns.

When unequal depth Joist Girders frame into both sides of the column web, compression buckling is checked when the stabilizer plates overlap one another. In cases when the web does not have sufficient strength for the compressive or tensile forces delivered by the stabilizer, the strength can be increased by:

- Selecting a W Shape with a thicker web
- Adding a stiffener to the web of the column
- Adding a doubler plate

### Bottom Chord Connection

The bottom chord of the Joist Girder must be attached to the stabilizer plate to resist and transfer the chord force to the column. Stabilizer plates are normally sized based on a ¾ -inch thickness. Using a ¾-inch plate allows the plate to fit between the bottom chord angles allowing fillet welds to be made to the heels and toes of the chord angles. Economically, the stabilizer plates can usually be connected to the column using only fillet welds. Stabilizer plates must be welded to the column flange to resist the compression and tension forces. The specifying professional must specify that the Joist Girder bottom chords be a minimum thickness to accommodate the required weld size. As is required for the top chord, Vulcraft has the responsibility to check the bottom chord angles for shear lag. Case 2 from Table D3.1 is applicable for this check. For reference, the shear lag factor is calculated for the bottom chord based on the input of the angle size, bottom chord leg size, and bottom chord thickness. Providing longer length fillet welds will reduce shear lag effects.

### Stabilizer Plate Checks

The following strength checks are made:

1. Determine the weld between the bottom chord and the stabilizer
2. Check the Whitmore width for stabilizer (AISC Manual Section 9-3)
3. Check stabilizer yielding
4. Check stabilizer Block Shear Rupture Strength
5. Determine the weld between the stabilizer and the column

The spreadsheet uses the Joist Girder bottom chord forces to determine the weld requirements. Some designers prefer to provide enough weld to develop the full strength of the stabilizer.

### Minimum Member Thicknesses (Weld Compatibility)

Throughout the spreadsheet, checks are made for the minimum thicknesses of base metal to match the weld strength. From the AISC Specification, Section J2.4, the design strength, $\phi R_n$, and the allowable strength, $R_n/\Omega$, of welded joints shall be the lower value of the base material strength according to the limit states of tensile rupture, shear rupture, and the weld metal strength based on the limit state of rupture.

The moment connection design tools can be downloaded at no cost from the SJI website:

steeljoist.org/product-category/design-tools/.

## 7.6 JOIST SEATS SUBJECTED TO ROLLOVER FORCES

It is often necessary to evaluate the resistance of a joist seat relative to a lateral force applied to the top of the joist seat. Shear collector members between the joists will often be required because of the low rollover strength of the joist seats.

Green P.S. and Sputo T (2004) provide a theory to calculate uplift forces on joist seats. The yield line analysis in the paper can be adjusted to determine joist seat rollover.

Based on the theory the tension force, $T_u$, at the seat toe equals:

$$T_u = \frac{M_p L_{YL}}{a}$$

where

$T_u$ = Ultimate uplift force at the seat toe angle, kips

$M_p = F_y Z$, kip-in./in.

$L_{yL}$ = Length of yield line where $L_{YL}$ is the lesser of ($L_w + \pi a$) and $L_s$, in.

$L_s$ = length of bearing seat, in.

$a = 2.3t$ = Distance from toe of angle to yield line, in.

$Z = t^2/4$ = Plastic section modulus of unit length of plate, in.$^2$/in.

The rollover resistance can be determined by multiplying, $T_u$, by a lever arm equal to the distance, $m$, taken as the distance from the tension force to an assumed compression reaction and dividing by the seat height, $h$.

where

$m$ = seat angle leg length, plus the gap, plus $k$ for the seat angle. See Figure 7.6.2.

Thus,

$\phi V = T_u$(m/h)

Twenty-four tests were conducted by Green and Sputo on various K-series joist seats. The test values compared to theory had a mean value of 0.985 with a standard deviation of 0.149.

Green and Sputo indicate the following limitations exist in applying the design equations:

(1) The joist seat must be welded to the steel anchorage plate or supporting steel beam or joist girder with approximately equal length fillet welds on each side. The fillet weld must have a minimum equivalent throat equal to that of a 5⁄32-inch equal leg fillet weld. While this exceeds the SJI minimum ⅛-inch leg weld, in practice most field applied anchorage welds exceed this minimum. Each weld provided must be a minimum of one (1) inch long.

(2) The maximum thickness of the horizontal (bearing) leg of the seat angle must not exceed ¼- inch. It is unknown whether the 5⁄32-inch nominal fillet weld is adequate to develop the yield line mechanism for thicker seats based on the scope of this research.

(3) The joist seat length must be a minimum four (4) inches long and must not exceed eight (8) inches in length.

(4) For seat configurations where the seat angles overlap the top chord angles, the seat angles must be welded to the top chord from both the inside and outside. The result of not providing the outside weld is clearly illustrated in Figure 8. This outside weld is

necessary to prevent rigid body rotation of the seat angles that will prevent development of the yield line mechanism.

Sixteen proprietary tests were conducted by Vulcraft. The results of these tests compare favorably with the above theory.

### Example: 7.6.1 Joist Seat Rollover Resistance

Determine the resistance to rollover of the seat shown in Figures 7.6.1 and 7.6.2.

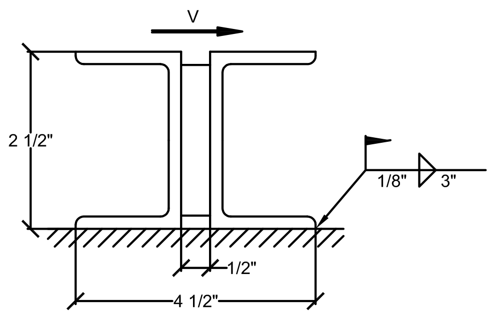



**Fig. 7.6.1 Joist Seat**

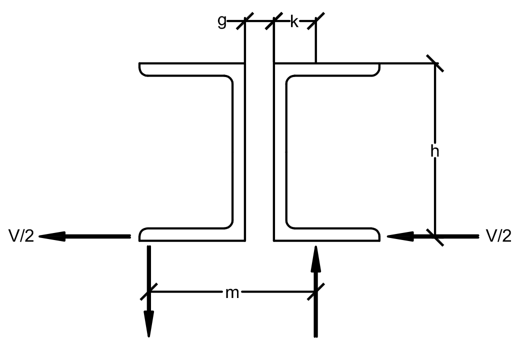



**Fig. 7.6.2 Resisting Forces on Joist Seat**

**Given:**

A seat is fabricated from 2x2x1/8 in. angles with a specified yield strength of 50 ksi. The gap between the angles is ½ in., $k$ =0.375 in. for the seat angles.

**Solution:**

$Z$ = (1.0 in.)(0.125 in.)$^2$/4 = 0.0039 in$^3$/in.

$M_p$ = (50 ksi)(0.0039 in.$^3$/in.) = 0.195 kip-in./in.

$a$ = 2.3(0.125 in.) = 0.288 in.

$L_{YL}$ = 3 in. + $\pi$(0.288 in.) = 3.90 in.

$T_u$ = (0.195 kip-in./in.)(3.90 in.)/0.288 in.) = 2.64 kips

$V = T_u(m/h)$

where

$m$ = 2.0 in. + 0.5 in. + 0.375 = 2.875 in.

$h$ = 2.5 in.

$V$ = (2.64 kips)(2.875 in)/(2.5 in.) = 3.04 kips/in.

$\phi V$ = 0.9(3.04 kips) = 2.74 kips

$V/\Omega$= (3.04 kips)/1.67 = 1.82 kips

Check the weld strength:

$R_u$ = (3 in.)(0.707)(0.6)($F_{EXX}$)(1/8 in.)

= (3 in.)(0.707)(0.6)(70 ksi)(1/8 in.)

= 11.1 kips (Directional increase not used

$\phi R_u$ = 0.9(11.1) = 10.0 kips > 2.64 kips **o.k.**

# Chapter 8

# Responsibilities

## 8.1 INTRODUCTION

The furnishing of deck, joists and Joist Girders is a commercial transaction involving buyer and seller. The identification of and relations between these two parties is established by contract, negotiation and by two standard documents. These documents are:

1) "Code of Recommended Standard Practice for Composite Deck, Form Deck and Roof Deck Construction," published by the Steel Deck Institute (SDI, 2017a).

2) "Recommended Code of Standard Practice for Steel Joists and Joist Girders," published by the Steel Joist Institute (SJI, 2017a).

The seller in these documents is the party which manufactures and distributes steel deck, steel joists, Joist Girders and accessories. The other side of the relation is the buyer. The buyer is that party which orders, receives and pays for steel deck, steel joists, Joist Girders and accessories. On the buyer's side are numerous parties with varied responsibilities. Only one of these parties directly pays the seller but all are involved. This group includes:

- The Owner
- The Architect/Engineer
- The General Contractor/Construction Manager
- The Joist and Deck Erector

Depending on the nature of the construction, some or all of the following parties may be involved:

- Structural Steel Fabricators and Erectors
- Light Gage Steel Fabricators and Erectors
- Pre- Engineered Building Manufacturers and Erectors

Lastly, other trades may be involved, such as:

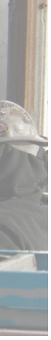
- Masons
- Concrete Contractors
- Carpenters, Lathers, Insulators, etc.
- Mechanical and Electrical Trades

All the duties and responsibilities on a given project must add up to 100% coverage. What is not done by one party must be done by another, so it is very important that all parties have a clear understanding of the division of responsibilities. The Codes of Standard Practice of SDI and SJI govern in the absence of other contract requirements. They represent a good summary of what the industry expects under usual conditions and thus a review of these standards is a useful starting point.

## 8.2 SDI CODE OF STANDARD PRACTICE

The SDI code covers eight major sections: 1) General, 2) Materials, 3) Quality and Scope Responsibility, 4) Estimating and Bidding, 5) Drawings and Specifications, 6) Handling and Protection, 7) Installation of Deck and Accessories and 8) Concrete Design and Placement.

The buyer is expected to provide "complete architectural and structural drawings and specifications prepared by the designer, all correctly dimensioned." The plans and specifications are expected to show details and be complete as to the extent of deck and accessories to be furnished. The basis of design is the current applicable SDI specification unless specified otherwise.





The “base bids for roof deck shall include the deck as shown in plan on the structural drawings. Base bid shall also include ridge, hip and valley plates which are specifically designated on the structural drawings, which are not part of the vertical load resisting system, and sump pans per architectural drawings and specifications. No other deck or accessories shall be included unless specified.”

The base bid for composite floor deck and non-composite floor deck “shall include deck as shown in plan on the structural drawings and only those sheet steel accessories specifically designated on the structural drawings and called for in the appropriate division of the specifications. No other deck or accessories shall be included unless specified.”

Prior to the fabrication of deck, erection layouts showing the location of all sheets are submitted to the buyer for review and approval. Shop work does not begin until final approval is received from the buyer, unless this approval is waived. After such final approval by the buyer, changes initiated by the buyer are subject to extra charges. The building plans are taken as correct except in the case of discrepancy between the building plans and structural steel or purlin (joist) spacing plan, in which case the steel plans are given precedence.

The code excludes the following from the base bid:

1. Sheet Metal Work: All closures, flashing and other similar items shall be detailed and furnished by others.
2. Shear Connectors: shear connectors and related placement plans
3. Mechanical Fasteners: screws or power-actuated fasteners and their installation tools
4. Welding Materials: all consumables used for field welding
5. Support Material: deck supporting members
6. Touch-up Paint: paint used for field touch-up

The construction phase involving the site storage and installation of steel decking is covered in the code. All construction phase activity is done by the buyer or his agents.

## 8.3 SJI CODE OF STANDARD PRACTICE

The SJI code covers eight sections: 1) General, 2) Joists Joist Girders and Accessories, 3) Materials, 4) Inspection, 5) Estimating, 6) Plans and Specifications, 7) Handling and Erection and 8) Business Relations.

As in the case with steel decking, the seller in the joist code is the party “engaged in the manufacturer and distribution of steel joists, Joist Girders and accessories.” The buyer is that party which orders, receives and pays for the joists, Joist Girders and accessories. As cited before, the buyer is a group of parties with duties and responsibilities defined in their own agreements.

The SJI code is the governing standard in the absence of specification requirements to the contrary. The code states that it is expected that the design prepared by architects and engineers be in accord with the specifications and load tables of the Steel Joist Institute. The seller furnishes steel joists, Joist Girders and accessories to the specifications provided, or in the absence of other requirements, to the specifications and code of the Steel Joist Institute. The seller must identify all material as to size and type. The seller is not responsible for the erection of items furnished.

Section two of the code gives certain physical requirements for joists, Joist Girders and accessories. The section also addresses the proper specification of design loads by the specifying professional, bridging and bridging anchors, bottom chord bracing for Joist Girders and connections.

Section three of the code states that the steel used in the manufacture of steel joists and Joist Girders is to comply with the latest edition of the applicable SJI Specification, and that paint for steel joists, Joist Girders and accessories, when specified, shall conform to the latest edition of the appropriate SJI Specification.

Section four of the code states that inspection of all joists and Joist Girders will be made in accordance with the latest edition of the appropriate SJI Specification.

Section five of the code deals with estimates. It requires that plans "show the character of the work with sufficient clarity to permit making an accurate estimate" and should include the following information:

1. Designation and location of materials, including any special design or configuration requirements
2. Location and elevations of supports
3. Location and lengths of joist extended ends
4. Location and size of openings in floors and roofs
5. Location of all partitions
6. Location and magnitude of concentrated loads
7. Construction and thickness of floor slabs, roof decks, ceilings and partitions
8. Joists or Joist Girders requiring extended bottom chords
9. Paint, if different from manufacturer's standard

The expected scope of estimated work includes the following:

1. Steel Joists
2. Joist Girders
3. Joist Substitutes
4. Joist Extended Ends
5. Ceiling Extensions
6. Extended bottom chord used as strut
7. Bridging
8. Joist Girder bottom chord bracing
9. Headers
10. One coat of shop paint, when specified, shall be in accordance with Section 3.2

The following are not expected to be included in the scope of an estimate but would be included if specifically designated in the plans and specifications:

1. Headers when end reactions exceed 10,000 lbs.
2. Headers for DLH-Series joists
3. Reinforcement in slabs over joists
4. Centering material, decking and attachments
5. Miscellaneous framing between joists for openings
6. Loose individual or continuous bearing plates and bolts or anchors for such plates

7. Erection bolts for joists or Joist Girder end anchorage
8. Horizontal bracing in the plane of the top and bottom chords
9. Bridging anchors and anchorage
10. Wood nailers
11. Moment plates
12. Special joist configuration or bridging layouts
13. Shear studs

These items which are related to the joist and Joist Girders, but not included in the joist and Joist Girder estimate, must be contracted for from others unless specifically specified otherwise. This requires the coordination of someone other than the seller (joist manufacturer).

Section six of the code covers plans and specifications. The plans and specifications provided by the buyer give the information required as listed above as well as the live loads to be used, wind uplift (if any), the weights of partitions and the elevations of finished floor and roof and bearing elevations.

In addition to the SJI Code of Standard Practice, the International Building Code (IBC) and other building codes, have requirements for information on contract documents. These items are necessary to properly estimate and design the project. They include the following:

1. Building Code and year (i.e. IBC 2015)
2. Method of joist and Joist Girder design, ASD or LRFD
3. $S_{DS}$ factor
4. Deflection criteria
5. Layout and spacing of joists and Joist Girders, including dimension to starting point of layout
6. End supports
7. All special loading (concentrated loads, non-uniform loads, net uplift loads, axial loads, end moments, and connection forces)
8. Profiles for non-standard joist and Joist Girder configurations
9. Oversized or other nonstandard web openings

The $S_{DS}$ factor needs to be specified on the plans. It is used in the load combinations with seismic load and will impact the design of the joists and Joist Girders ($E = E_h + E_v$, $E_v = 0.2S_{DS}D$).

The plans furnished by the seller include the steel joist placement plans to show the materials specified on the construction documents and are to be utilized for field installation. Detailed plans and lists showing the number, type, location, spacing, anchorage and mark of all joists, Joist Girders and accessories.

Prior to shop work, the seller submits the detailed plans to the buyer for review and approval. Shop work does not commence until final approval is obtained from the buyer. After such final approval, changes initiated by the buyer are subject to extra charges. The building plans presented by the buyer are assumed to be correct unless written notice is given to the contrary.

Section seven of the code covers handling and erection, doing so chiefly by reference to SJI Technical Digest 9 “Handling and Erection of Steel Joist and Joist Girders.”

Section eight of the code covers business relations. Among other items concerning presentation of proposals and acceptance of proposals, it gives the following regarding billing and payment:

a) Lump sum contracts are to be billed proportionately to shipments.

b) Payments are due in full without retention. It should be noted that many construction contracts require retainage between the owner and contractors, and so this requirement (if not modified), could be out of sync with the remainder of the contracts.

Lastly, section eight states that disputes will be settled by means of binding arbitration.

## 8.4 RESPONSIBILITY OF THE BUYER

The foregoing discussion of the two codes is intended to illustrate the division of responsibility between buyer and seller in transactions with deck, joists, Joist Girders and accessories. What follows is a discussion of the division of responsibilities on the buyer's side of the relation. As has already been stated the parties on the buyer's side are numerous. The relations on the buyer's side can best be illustrated by using the example of five different joist support types as shown below and by considering relations among the owner, the architect/engineer and the contractor.

1. Structural steel frame
2. Pre-engineered metal building frame
3. Light gage bearing walls
4. Concrete frame
5. Masonry bearing walls

Buildings involving the support of joists and Joist Girders by structural steel represent the largest proportion of all construction with joists and Joist Girders. The American Institute of Steel Construction has published the "Code of Standard Practice for Steel Buildings and Bridges" (AISC, 2016a). This code gives trade practices relating to the fabrication and erection of structural steel. It contains this definition of structural steel: "'*Structural Steel,*' shall consist of the elements of the structural frame that are shown and sized in the structural *design documents*, essential to support the design loads as used to define the scope of work in the contract documents, consists of steel elements of the structural steel frame essential to support the design loads and described as: [the code then provides a list of elements]." Cold-formed steel products [deck] and open- web joists and Joist Girders" are not included in the list of structural steel, but rather they are listed in "Other Steel, Iron or Metal Items," a category of items not included in structural steel "even where such items are shown in the structural *design documents* or are attached to the *structural steel frame*."

Since the requirements for the erection of structural steel and the erection of steel deck, steel joists and Joist Girders do not overlap and are not mutually cross referenced, a common way to have these components erected into one uniform structural framework is to have one party erect all these components under one contract. This ensures that the two categories of material will be erected as one. Secondly, the ordering of steel deck, steel joists and Joist Girders can be made part of the steel fabrication contract. In this way the coordination of bearing elevations, seats, stabilizer plates, bolt holes, etc. can be done under one responsibility and it is the best way to resolve discrepancies between the fabricated steel and the steel deck, steel joists and Joist Girders. While it is important to have good coordination in the fabricated elements, it is critical to have it in the erection work. As stated previously, the AISC Code of Standard Practic does not cover the erection of steel deck, steel joists and Joist Girders. The principal document which does is SJI Technical Digest 9, "Handling and Erection of Steel Joists and Joist Girders," which is incorporated by reference in the SJI Code of Standard Practice. This digest gives a thorough presentation of the issues involved in handling and erecting steel joists.

One aspect of steel erection is the employment of temporary bracing. Some steel frames do not rely on any element other than the structural steel for strength or stability. Thus, when the work required to erect and finish the steel framework is complete, the temporary bracing can be removed at the erector's discretion. Other steel frames rely on elements of the building to stabilize the structural steel framework. These other elements can be steel deck diaphragms, shear walls, and, as was seen above, steel joists and Joist Girders. Non-structural steel elements required for the strength and/or stability of the steel frame are to be identified in the Contract Documents. The installation schedule for non-structural steel elements of the lateral load resisting system and connecting diaphragm elements are to be provided by the Owner's Designated Representative for design to the erector prior to bidding. The erector supplies temporary bracing consistent with this information and removes the bracing when appropriate to these conditions. The other building elements are expected to be provided in a timely fashion consistent with the contract documents.

The AISC Code of Standard Practice gives the following definition of Contract Documents; "The documents that define the responsibilities of the parties that are involved in the bidding, fabricating and erecting structural steel. These documents normally include the design documents, the specifications and the contract." These documents may be prepared by different parties. The plans and specifications are prepared by the building designer, an architect or engineer. The contract may be prepared by the designer or by the owner, a construction manager, a general contractor or a subcontractor.

Steel deck and steel joists can also be supported by light gage steel, pre-engineered metal buildings, masonry walls, or concrete beams and walls. Among other organizations, these materials and systems are represented by:

- Metal Lath/Steel Framing Association
- American Iron and Steel Institute
- Metal Building Manufacturer's Association
- National Concrete Masonry Association
- American Concrete Institute

These organizations publish codes, technical bulletins and standards which apply to these materials and systems. None of these organizations has comparable documents relating to steel deck, steel joists and Joist Girders developed to the same degree as the AISC Code of Standard Practice, the SJI Code of Standard Practice and the SDI Code of Standard Practice. All these groups have addressed issues of material and workmanship standards, but not specifically in relation to joists and steel deck.

In each of these systems, the coordination concerns relate to the function of the deck and joists in the completed structure and the details at the interface with the supporting elements. Examples of the functional uses requiring coordination would be roof and floor deck diaphragms, steel joist and Joist Girder rigid frames, and steel joist and Joist Girder wind struts.

As these functions could relate to building stability, it would be useful to the various contractors and the deck and joist erector for these functions to be given in the plans. This would then be a situation analogous to the listing of nonstructural steel and connecting diaphragm elements required by the AISC Code of Standard Practice.

As for the coordination of details, this must be addressed in the subcontractor to contractor contracts, and must cover four general areas:

1. Attachment of joist end seats: The bearing surface must be appropriately designed and detailed to receive the joist end seat. This requires checking for bearing length, bearing width clearance, provision of holes, welds or embedded weld plates.
2. Attachment of Joist Girder end seat and stabilizer plate: As with joists, the bearing surface must be appropriate to receive the Joist Girders. Additionally, the provision of an adequately designed and detailed stabilizer plate must be accounted for.
3. Attachment of Bridging: Bridging must be anchored at its ends. This anchorage may require expansion bolts or other anchors where the bridging cannot be terminated by

welding to the last structural element.

4. Attachment of deck perimeter: The deck perimeter is often supported by angles or other loose material which must be detailed and installed to receive the steel deck. This material would not normally be provided by the deck supplier nor would it be installed by the deck erector.

The foregoing discussion of responsibilities among the parties in the design and contraction process can be summarized as follows:

**Owner:** The owner is the key to the entire process. It is the owner who initiates the process and defines the building to be constructed. The owner provides the building usage which establishes the building function and characteristics. This includes the nature of the use of the building and any special requirements beyond the minimum requirements of the building code for the type and size of the proposed building. The owner establishes the level of quality of the building. The owner hires the architect/engineer and general contractor/construction manager.

**Architect/Engineer:** The architect/engineer is responsible for taking the owner's requirements and, in the context of codes and other regulations, preparing plans and specifications which conform to these requirements. The contents of these plans relative to joists was discussed above and is given in the SJI Code of Standard Practice. The purpose of these plans is to show the completed structure in enough detail that competent parties can understand what materials and labor are required to complete their work. The architect/engineer reviews shop drawings (also discussed above) which show the supplier's understanding of the materials required as well as fabrication and erection details. The review of the architect/ engineer is for conformity to his design concept only. The review or approval of shop drawings does not approve deviations from original specifications. There are separate procedures for substitutions. Nor does the review check for dimensions or fit up. The architect/engineer makes periodic visits to the site to examine the construction for conformity with the design. These visits are not intended to be detailed inspections as part of a quality control program unless such a program is explicitly contracted with the owner.

**General Contractor/Construction Manager:** Amongst other duties these parties are in charge of carrying forward the work of construction to completion. In the context of steel deck, steel joists and Joist Girders the general contractor/construction manager solicits proposals for this work. A key activity in this is the apportionment of work among subcontractors so that each has a clear understanding of what must be done and when. This division of work must be clear in the subcontractor's contracts. The plans and specifications should not be relied on solely to establish the required division and assignment of work to subcontractors.

**Fabricators:** The structural steel fabricator prepares fabrication (shop drawings) and erection drawings showing the work required for the steel framework consistent with the AISC Specifications and Code of Standard Practice. If the procurement of steel deck, steel joists and Joist Girders is part of his contract, the fabricator coordinates his work with that of the steel deck, steel joists and Joist Girder suppliers. Even if the procurement is not part of this contract, coordination can be made part of it. Otherwise the required coordination must be performed by the general contractor/construction manager. It is sometimes the case that fabricators or others procure joists and Joist Girders by means of bills of material and joist erection plans which are sent to the manufacturer. In this case, the task of interpretation of the plans and specifications requirements is in the hands of the party who prepares the bills of material. The only responsibility of the manufacturer is to provide material conforming to the requirements given on the bills of material, not the plans and specifications. When bills of material are prepared by others for the manufacturer, the special skills that the manufacturer has in reading plans and specifications considering the unique requirements for his product, are not taken advantage of. The direct use of plans and specifications by the manufacturer is preferred over the provision of bills of material prepared by others.

**Manufacturer:** The manufacturer prepares erection drawings consistent with the requirements of the building design requirements. The content of the drawings is as presented in the SJI Code of Standard Practice. After these drawings are approved, shop orders are prepared which provide the details of fabrication to the shop. The material is manufactured and shipped. This terminates the manufacturer's work unless errors in his work are discovered which must be corrected.

**Erector:** Steel deck, steel joists and Joist Girders are usually received at the site by the erector who checks for shipping damage and quantities and directs their storage and temporary protection at the site. As required, the erector erects the material consistent with the specifications and the AISC and SJI Codes of Standard Practice. The erector makes all field connections and provides temporary bracing, as was discussed above.

**Other trades:** Once the erection is complete, other trades attach to or otherwise load the steel frame with other elements required to complete the building. They must at all times take care not to damage the structure by:

1. Excessive construction loads
2. Cutting or notching the structural elements
3. Applying concentrated loads in excess or in different locations of design specified loads

This means that they must read and understand the plans and exercise care and judgment.

## 8.5 CONTENT OF PLANS

The contents of the plans for bidding was discussed previously. The requirements are presented in the SJI Code of Standard Practice in Section 5.1. Also, the Standard Specification requires that wind uplift forces must be shown on the documents. Specifications should be consistent with the Code of Standard Practice and the Standard Specifications. When conflicting or stricter requirements are given, it is a source of confusion and extra expense, because it is a departure from normal operating procedures.

On some projects it is desirable to prepare preliminary designs, pricing drawings, scope drawings or phased drawings to expedite the work.

Preliminary designs and pricing drawings are prepared to establish budgets, determine feasibility and compare framing approaches. As such, they must never be considered complete or binding. On the other hand, they should be treated with enough care to be useful. The key element is that they contain a good description of both standard and special conditions, and the emphasis must be on the special, nonstandard aspects. The usual form is to present typical bays or bents and a description of the frequency of these typical conditions, plus a description of special bays and loadings. This should include:

1. Decking selection and attachment requirements
2. Roof drainage and roof slopes
3. Size and spacing of joists
4. Depth and loading of Joist Girders
5. Bracing scheme used and location of bracing
6. Designation of joists used as struts
7. Sample column connections, especially special connections
8. Special depth joist seats
9. Location and framing approach for major openings
10. Wind uplift loads
11. Special bridging requirements

12. Special deflection requirements
13. Special clearance requirements
14. Design loads and material strengths
15. Contemplated start and completion dates

Certain fast track or negotiated projects begin the construction process with scope documents. These drawings and specifications are intended to be complete in terms of the scope of work involved, but incomplete as to final details and design. These documents are issued to solicit proposals and can form the basis of contracts. Because of this, they must be as complete as possible and must contain indication of where they are incomplete or not final. A procedure must be established in the contract to resolve differences between the scope documents and the final documents. No one should consider the documents as final. The content of scope documents is the same as those for the preliminary drawings given above, but the contents are presented more formally. It is desirable that the scope documents be as complete as possible, especially about special, nonstandard requirements.

Another way to expedite the pace of design and construction is the use of phased documents and phased construction. In this process the total work is designed and bid in multiple bid packages. A typical breakdown of a project would be:

1. Foundations
2. Superstructure and primary mechanical systems
3. Interior development and secondary mechanicals

In this approach, it is intended that each bid package be complete and final and be integrated into preceding and subsequent bid packages. This does not always turn out to be the case, because subsequent bid packages may bring to light unforeseen conditions which may require modification of previously bid and constructed work. It is very important in phased documents to distinguish the work covered in each package. The work of previous packages becomes an existing condition with respect to the current package, and in some cases special provisions must be made for the installation of the work of subsequent packages. This determination requires extra effort on the part of the architect/engineer and general contractor/construction manager.

## 8.6 CONCLUSION

This presentation of the content of plans, specifications and the responsibility of the various parties was given to promote a clear understanding of the process by which buildings involving steel decks, steel joists and Joist Girders are designed and constructed. An understanding of these issues is essential for a smooth-running project.

## REFERENCES

ACI (2014), ACI 318, “Building Code Requirements for Reinforced Concrete and Commentary,” American Concrete Institute, Farmington Hills, MI.

ACI (2011), “Building Code Requirements and Specification for Masonry Structures and Related Commentaries,” 530/530.1-11, American Concrete Institute, Farmington Hills, MI.

ACI (2006), “Guide for Cast-in Place Low Density Concrete”, ACI 523.1, American Concrete Institute, Farmington Hills, MI.

AISC (2016a) “Code of Standard Practice for Steel Buildings and Bridges,” American Institute of Steel Construction, Chicago, IL.

AISC (1989), “Specification for the Design, Fabrication and Erection of Structural Steel Buildings with Commentary,” American Institute of Steel Construction, Chicago, IL.

AISC (2016b), Design Guide 11, “Vibrations of Steel-Framed Structural Systems Due to Human Activity,” Second Ed., American Institute of Steel Construction, Chicago, IL.

AISC (2016c), “Seismic Provisions for Structural Steel Buildings,” American Institute of Steel Construction, Chicago, IL.

AISC (2010), “Specification for the Design, Fabrication and Erection of Structural Steel Buildings with Commentary,” American Institute of Steel Construction, Chicago, IL.

AISC (2016d), “Specification for the Design, Fabrication and Erection of Structural Steel Buildings with Commentary,” American Institute of Steel Construction, Chicago, IL.

AISC (2017) 15th Edition, “Steel Construction Manual,” American Institute of Steel Construction, Chicago, IL.

AISI (2016a), “North American Specification for the Design of Cold-Formed Steel Structural Members,” American Iron and Steel Institute, Washington, D.C.

AISI (2016b), “North American Standard for the Design of Profiled Steel Diaphragm Panels,” American Iron and Steel Institute, Washington, D.C.

AITC (2012), “Timber Construction Manual,” American Institute of Timber Construction, Tigard, OR.

APA (2007), “Diaphragms and Shear Walls, Design/Construction Guide,” V310, American Plywood Association, Tacoma, WA.

APA (2016), “APA Construction Guide,” American Plywood Association, Tacoma, WA.

ASCE (1987), “Guide for Design of Steel Transmission Towers,” Task Committee on Updating Manual 52 of the Committee on Electrical Transmission Structures, Structural Division, American Society of Civil Engineers, New York, NY.

ASCE/SEI (2016), “Minimum Design Loads for Buildings and Other Structures,” ASCE/SEI 7-16, American Society of Civil Engineers, Reston, VA.

ASTM (2018) “Standard Methods of Fire Tests of Building Construction and Materials,” ASTM E119-00, West Conshohocken, PA.

ASTM (2017), “Standard Specification for Light Weight Aggregates for Insulating Concrete”, ASTM International, West Conshohocken, PA

AWC (2015) “Special Design Provisions for Wind & Seismic (SDPWS)” ANSI/AWC, American Wood Council, Leesburg, VA.

AWS (2015), "Structural Welding Code" (AWS D1.1/D1.1M), American Welding Society, Miami, FL.

AWS (2017) "Structural Welding Code – Sheet Steel" (AWS D1.3/D1.3M), American Welding Society, Miami, FL.

AWS (2016), "Structural Welding Code-Seismic Supplement (AWS D1.8/D1.8M)," American Welding Society, Miami, FL.

CMAA (2015a), Specification for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes- No. 70, Crane Manufacturers of America, Inc., Charlotte, NC.

CMAA (2015b), Specification for Single Girder Cranes- No. 74, Crane Manufacturers of America, Inc., Charlotte, NC.

Doyle, C.G. (2010), "Behavior of Open Web Steel Joist Seats Subjected to Lateral Loads," Master's Thesis, Villanova University.

FM Global (2018), Approval Guides, "A Guide to Equipment, Materials, and Services Approved by Factory Mutual Research for Property Conservation," FM Global, Johnston, RI.

FM Global (various dates), "Property Loss Prevention Data Sheets 1-54, FM Global Data Sheets," FM Global, Johnston, RI.

Fisher, J.M. and West, M.A. (2019), "Serviceability Design Considerations for Low- Rise Buildings, Steel Design Guide Series 3," American Institute of Steel Construction, Chicago, IL.

Fisher, J.M. and Sputo, T. (2017), "Are your roof members overstressed?" Structure Magazine, March, 2017

Green, P.S. and Sputo, T. (2004), "Uplift Capacity of K-Series Open Web Steel Joist Seats," ASCE, Structures Library.

IAS (2019), "International Accreditation Service," Brea, CA

IAPMO (2018), "International Association of Plumbing and Mechanical Officials" (IAPMO), Ontario, CA

IBC (2015), "International Building Code," International Code Council, Inc. (ICC), Country Hills II, IL.

MBMA (2012), "Low Rise Building Systems Manual," Metal Building Manufacturers Association, Cleveland, OH.

MIA (2017), "Reinforced Masonry Engineering Handbook," 8th Edition, (MIA, 2017), Brick Institute of America, Torrance, CA.

NAS (1974), "Expansion Joints in Buildings, Federal Construction Council, Technical Report No. 65," National Academy of Sciences National Research Council, Washington, D.C.

NFPA (2019), "Standard for the Installation of Sprinkler Systems," NFPA 13- 2019, National Fire Protection Association, Quincy, MA.

NRCA (2015), "Roofing Manual Membrane Roof Systems," The National Roofing Contractors Association, Chicago, IL.

SDI (2017a) “Code of Standard Practice 2017,” Steel Deck Institute, Glenshaw, PA.

SDI (2017b) ANSI/SDI C-2017 “Standard for Composite Steel Floor Deck-Slabs,” Steel Deck Institute, Glenshaw, PA.

SDI (2017c) ANSI/SDI NC-2017 “Standard for Non-Composite Steel Floor Deck,” Steel Deck Institute, Glenshaw, PA.

SDI (2017d) ANSI/SDI RD-2017 “Standard for Steel Roof Deck,” Steel Deck Institute, Glenshaw, PA.

SDI (2004), “Diaphragm Design Manual” (DDM03), Third Edition, Steel Deck Institute, Glenshaw, PA.

SDI (2015), “Diaphragm Design Manual” (DDM04), Fourth Edition, Steel Deck Institute, Glenshaw, PA.

SDI (2017e), “Floor Deck Design Manual (FDDM),” First Edition, Steel Deck Institute, Glenshaw, PA.

SDI (2017f), “Manual of Construction with Steel Deck (MOC3),” Third Edition, Steel Deck Institute, Glenshaw, PA.

SDI (2017g), “Roof Deck Design Manual (RDDM),” First Edition, Steel Deck Institute, Glenshaw, PA.

SDI (2001), “Standard Practice Details (SPD2),” Steel Deck Institute, Glenshaw, PA.

SDI (2018), “Steel Deck on Cold-Formed Steel Framing Design Manual (SDCFSFDM),” First Edition, Steel Deck Institute, Glenshaw, PA.

Seely, F.B. and Smith, J.O. (1962), “Advanced Mechanics of Materials,” John Wiley and Sons, Inc., New York, NY.

SJI (2003), “Design of Fire-Resistive Assemblies with Steel Joists”, TECHNICAL DIGEST 10, Steel Joist Institute, Florence, SC.

SJI (2007a), “Design of Lateral Load Resisting Frames Using Steel Joists and Joist Girders´ TECHNICAL DIGEST 11, Steel Joist Institute, Florence, SC.

SJI (2007b), “Evaluation and Modification of Open Web Steel Joists and Joist Girders”, TECHNICAL DIGEST 12, Steel Joist Institute, Florence, SC.

SJI (2008b), “Handling and Erection of Steel Joists and Joist Girders”, TECHNICAL DIGEST 9, Steel Joist Institute, Florence, SC.

SJI (2017a), “Recommended Code of Standard Practice for Steel Joists and Joist Girders,” Steel Joist Institute, Florence, SC.

SJI (2019), “Standard Specification Composite Steel Joists Catalog,” First Edition, Steel Joist Institute, Florence, SC.

SJI (2015a), “Standard Specification for Steel Joists, K-Series, LH-Series, DLH-Series and For Joist Girders,” Steel Joist Institute, Florence, SC.

SJI (2018a), “Structural Design of Steel Joist Roofs to Resist Ponding Loads,” TECHNICAL DIGEST 3, Steel Joist Institute, Florence, SC.

SJI (2012), “Structural Design of Steel Joist Roofs to Resist Uplift Loads”, TECHNICAL DIGEST 6, Steel Joist Institute, Florence, SC.

SJI (2015b), "Vibration of Steel Joist Concrete Slab Floors", TECHNICAL DIGEST 5, Steel Joist Institute, Florence, SC.

SJI (2008a), "Welding of Open Web Steel Joists", TECHNICAL DIGEST 8, Steel Joist Institute, Florence, SC.

SJI (2017b), "44th Edition K- Series, LH- Series, DLH- Series, Joist Girders," SJI 100-2015/ANSI, Steel Joist Institute, Florence, SC.

SJI (2018), "90 Year Download Open Web Steel Joist Construction (1928-2018)," Steel Joist Institute, Florence, SC.

UL (2018a) "Guide Information for Roofing Materials and Systems," Underwriters Laboratories, Inc., Northbrook, IL.

UL (2018b) "Standard for Tests for Uplift Resistance of Roof Assemblies," Underwriters Laboratories, Inc., Northbrook, IL.

UL (2019) "Fire Resistance Directory," Underwriters Laboratories, Inc., Northbrook, IL.

Vulcraft (2016a), ECOSPAN®, Composite Floor System Design Manual" Nucor, Vulcraft/Verco Group, Nucor Corporation, Charlotte, NC.

Vulcraft (2017a), "Dovetail Roof Deck, Welded Support Connections," Nucor, Vulcraft/Verco Group, Nucor Corporation, Charlotte, NC.

Vulcraft (2016b), "Punchlok II® Roof Deck, Weld and Screw Support Connections," Nucor, Vulcraft/Verco Group, Nucor Corporation, Charlotte, NC.

Vulcraft (2018), "Steel Roof & Floor Deck," Nucor, Vulcraft/Verco Group, Nucor Corporation, Charlotte, NC.

Vulcraft (2017c), "Steel Joist & Joist Girder Systems," Nucor, Vulcraft/Verco Group Nucor Corporation, Charlotte, NC.

Young, W.C. (2011), "Roark's Formulas for Stress and Strain," Amazon.com/books, Eighth Edition, 2011.

Ziemian, R. X. (2018), "Removal of Bridging from Vulcraft's CJ and Ecospan® E-Series Floor Joists following 7 Day Concrete Curing", White paper submitted to Vulcraft, 2018.