

Designing with Vulcraft

Steel Joists, Joist Girders and Steel Deck



James M. Fisher, Michael A. West and Julius P. Van De Pas

NUCOR
VULCRAFT - GROUP

2nd edition

First Edition Copyright ©1991
Second Edition Copyright ©2002

by
Nucor Corporation

*All rights reserved. This book or any part thereof
must not be reproduced in any form without
the written permission of the publisher.*

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 herein, that this information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability arising from such use.

www.vulcraft.com

NUCOR
VULCRAFT - GROUP

ACKNOWLEDGEMENT

This book was produced under the guidance of Vulcraft managers and engineers. The authors wish to thank those Vulcraft personnel involved for their many suggestions and comments. They also wish to thank Dr. T.V. Galambos, Ph.D., P.E. for his careful reading of the manuscript and for his helpful comments. Appreciation is also due Carol T. Williams for the typing of the manuscript.

Authors:

James M. Fisher, Ph.D., P.E.
Michael A. West, P.E., AIA
Julius P. Van de Pas, P.E.

Computerized Structural
Design, Inc.
Milwaukee, Wisconsin

CHAPTER 1 INTRODUCTION	1		
1.1 PURPOSE	1		
1.2 HISTORICAL DEVELOPMENT	1		
1.3 CUSTOM DESIGNS	2		
1.4 CURRENT USAGE	2		
1.5 CODES AND SPECIFICATIONS	2		
1.6 OTHER SPECIFICATIONS	4		
1.7 REFERENCE STANDARDS	4		
1.8 SYNOPSIS OF CHAPTERS	5		
1.9 CONCLUSION	5		
CHAPTER 2 ROOFS	6		
2.1 INTRODUCTION	6		
2.2 ROOFING TYPES	6		
2.3 DECK TYPES	7		
Steel Decks	7		
Concrete Deck	10		
Wood Deck	10		
2.4 ROOF LOADING	11		
Dead Loads	11		
Collateral Loads	11		
Roof Live Loads	12		
Snow Loads	12		
Rain Loads	12		
Live Load Reductions	12		
Wind Uplift	12		
Load Combinations	13		
Concentrated Loads	13		
2.5 SERVICEABILITY CONSIDERATIONS	13		
Control of Roof Deflections	13		
Expansion Joints	14		
Area Dividers	14		
Roof Slope	15		
2.6 FRAMING CONSIDERATIONS	15		
Bay Size	15		
Direction of Joist Span	16		
Joist Spacing	16		
Joist and Girder Depth	16		
2.7 CONNECTIONS	16		
		Example 3.2.2 Composite Floor Slab with a Concentrated Load	22
		Concrete for Slabs on Steel Deck	23
		Pre-cast Slabs	23
		Wood Decks	23
		3.3 FLOOR LOADING	23
		Self-weight	23
		Collateral Loads	24
		Live Loads	24
		Live Load Reductions	24
		3.4 SERVICEABILITY CONSIDERATIONS	24
		Control of Deflections	24
		Other Deflection Considerations	24
		Vibrations	25
		Expansion Joints	25
		3.5 FRAMING CONSIDERATIONS	25
		Bay Size	25
		Joist Span Direction	25
		Joist Spacing	25
		Seat Depths	25
		Framing System Depth and Story Height	25
		Footnotes to Table 3.5.2	27
		Composite Joists	27
		3.6 SHEAR CONNECTORS	28
		3.7 CONNECTIONS	28
CHAPTER 3 FLOORS	19	CHAPTER 4 LATERAL LOAD SYSTEMS	29
3.1 INTRODUCTION	19	4.1 INTRODUCTION	29
3.2 FLOOR DECKS	19	4.2 DIAPHRAGMS	29
Cast-In-Place Concrete on Steel Deck	19	Introduction	29
Form Deck	19	Diaphragm Types	29
Composite Deck	20	Fastening of Steel Decks	29
Concentrated Loads	21	Fastening of Wood Systems	30
Example 3.2.1 Composite Floor Slab with a Line Load	21	Load Tables	30
		Design Procedure	31
		Diaphragm Connections	32
		Chord Force Connections	32
		Example 4.2.1 Diaphragm Chords	34
		Shear Collectors	35
		Attachment to Shear Walls	36
		Attachment to Vertical Bracing	37
		Example 4.2.2 Vertical Bracing with Joist Girder	38
		Expansion Joints	38
		4.3 HORIZONTAL BRACING	38
		Analysis Procedure	41
		Connections	41
		4.4 BRACED FRAMES	42
		Multistory Frames	42
		Single Story Frames	43
		4.5 RIGID FRAMES	43

CONTENTS

Design Considerations	43	Joists Spanning Parallel with Seam Roof	
Rigid Moment Connections	44	Standing Spans	99
Joist Girder Details	44	Tilted Joists	99
Joist Details	48	Extended Ends	99
Column Design	49	Folding Partitions	99
Drift Considerations	51	Seat Depth Changes	99
4.6 FRAMES WITH WIND CONNECTIONS	51	Unequal Joist Reactions on Joist	
Design Considerations for Wind		Girders or Beams	100
Connections	52	Weld Size	100
Connection Design	53	Expandable Walls	100
Column Design	54	Expansion Joints	100
Drift Considerations	55	Special Profile Joists	101
4.7 SELECTION OF THE LATERAL LOAD		Sloping Joists	101
SYSTEM	55	Example 5.10.1 Sloping Joists	103
Building Geometry	55	Joist Splices	103
Expansion Joints	55		
Roofing System	55		
Future Expansion	56		
4.8 DESIGN EXAMPLES	56		
Example 4.8.1 Building Braced			
at Walls	56		
Example 4.8.2 Rigid Frame Building	65		
CHAPTER 5 SPECIAL TOPICS	70		
5.1 INTRODUCTION	70	CHAPTER 6 SPECIFICATION OF	
5.2 HANGING LOADS	70	COMPONENTS	104
Cranes and Monorails	70	6.1 INTRODUCTION	104
Hangers and Bracing	71	6.2 JOISTS SUBJECTED TO UNIFORM	
Example 5.2.1 KCS Series Joist/Crane		GRAVITY LOADS	104
Support	72	Joist Selection	104
Beam Supports	75	Custom Joist Designs	104
Example 5.2.2 Beam Seat Design	75	6.3 JOISTS RESISTING CONCENTRATED	
Conveyors	76	LOADS	106
Sprinkler and Other Hanging Piping ..	77	Special Joists	106
5.3 HEADERS AND OPENINGS	78	Standard Joists	106
5.4 ROOF TOP UNITS	79	Example 6.3.1 Joist with a	
5.5 JOIST REINFORCEMENT	83	Concentrated Load	106
Introduction	83	KCS Series Joists	107
Load Distribution	84	Concentrated Load Reinforcement ..	108
Example 5.5.1 Load Distribution to		Beams	109
Joists	84	6.4 END MOMENTS AND CHORD	
Adding New Joists	86	FORCES IN JOISTS	109
Joist Reinforcement	86	Joist End Moments	109
Example 5.5.2 Joist Reinforcement ..	87	Joist Schedules	109
5.6 FLOOR VIBRATIONS	90	Chord Forces	109
5.7 SPANDREL SYSTEMS	93	6.5 JOISTS AND JOIST GIRDERS	
5.8 PONDING	94	SUBJECTED TO UPLIFT LOADING ..	109
5.9 FIRE RESISTANCE	97	6.6 JOIST GIRDERS SUBJECTED TO	
5.10 SPECIAL SITUATIONS	98	GRAVITY LOADS	110
Camber	98	6.7 BRIDGING CONSIDERATIONS	110
X-Bridging at Walls	98	6.8 SAMPLE SPECIFICATIONS	113
Cutting Bridging	98		
		CHAPTER 7 CONNECTION DESIGN	123
		7.1 THE BASIC CONNECTION	123
		Maximum Eccentric Top Chord	
		Force For Joist Girders	124
		Table 7.1.1	125
		Maximum Eccentric Top Chord	
		Force For Joists	125
		Rollover Capacity of Joist Girder	
		Seats	126
		Example 7.1.1 7/16 Inch Seat Angle	
		(Standard Seat)	127

CONTENTS

Example 7.1.2 1/4 Inch Seat Angle (Non-standard)	128	Example 7.3.1 Bottom Chord Force Transfer	138
Weld Requirements	128	7.4 TYPICAL CONNECTIONS	139
Summary	128	Typical Connection 7.4.1	139
7.2 MODIFICATIONS TO THE BASIC CONNECTION	128	Typical Connection 7.4.2	140
Joist Girder Modifications	129	Typical Connection 7.4.3	142
Design of "Fixed" Joist Girder Seats .	129	Typical Connection 7.4.4	143
Example 7.2.1 Joist Girder Fixed Seat	130	7.5 MOMENT PLATE CONNECTIONS ...	145
Increasing Joist Girder Chord Capacity with Seat Extensions	131	7.6 JOIST SEATS SUBJECTED TO ROLLOVER FORCES	146
Joist Modifications	131	Example 7.6.1 Joist Seat Rollover Resistance	147
Increasing Joist Moment Resistance with Joist Girder Seat Stiffeners	132	Summary of Test Results	148
Example 7.2.2 Joist Girder Seat Stiffeners	132	CHAPTER 8 RESPONSIBILITIES	150
Increasing Joist Chord Capacity with Seat Extensions	134	8.1 INTRODUCTION	150
Summary	135	8.2 SDI CODE OF STANDARD PRACTICE	150
7.3 BOTTOM CHORD EXTENSIONS	136	8.3 SJI CODE OF STANDARD PRACTICE	151
Geometrical and Tolerance Requirements	136	8.4 RESPONSIBILITY OF THE BUYER ..	152
Column Considerations	136	8.5 CONTENT OF PLANS	155
		8.6 CONCLUSION	155
		REFERENCES	156

CHAPTER 1

INTRODUCTION

1.1 PURPOSE

The purpose of this text is to provide a better understanding on the part of building designers in the proper use and employment of steel deck, steel joists and joist girders. It is not a manual which describes how these components are designed by their manufacturers. While steel deck, steel joists and joist girders have been in use for over a half century, recently they have been used in applications of greater complexity than initially contemplated. Their potential for innovative use has not yet been fully exploited. This manual will cover 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.

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. 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 Steel Joist Institute was formed in the 1920's. The establishment of the Standard Specification for Steel Joists allowed building designers to specify rather than design a structural component of the building frame. The acceptance of the Standard Specification by building codes and building officials, allow the use of steel joists in buildings without the need to reconfirm by engineering design the sizes and materials used in joists conforming to standard designations for given loads and spans. Over the years each Standard Specification has had an accompanying load table which gives 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, with the exception of the addition of tables for KCS joists which are discussed below.

While the standard load tables have always presented allowable capacities as uniform loads only, the application of load in the completed project rarely met this requirement to the letter. Over the years designers have used various strategies to account for concentrated and non-uniform loads. The principal method being 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 technical-

ly incomplete due to the fact that in some instances there were high localized top chord loadings and force reversals in some web members. This simple method was assisted by three features of the standard specification which have been eliminated from the current specification and Code of Standard Practice. They were:

1. The minimum shear capacity at any point along the web was never to be less than 50% of the required end reaction capacity.
2. The standard load table allowable uniform loads were allowed to be placed on the joist as a series of equivalent concentrated loads spaced at 33 inches on center down the length of the joist. This criteria allowed concentrated loads between panel points.
3. The Code of Standard Practice included a provision that a 400 pound concentrated load was to be expected anywhere along the top chord of the joist to accommodate headers.

The removal of these features meant that standard joists had to more closely adhere to the uniform load requirement as tabulated in the load tables, and also lead to the requirement that all concentrated loads and non-uniform loads be clearly specified in the construction documents so that the joists can be designed for these exact loadings. 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. Also many efficient floor framing layouts require joists for which the uniform load per foot exceeds the 550 plf limit for K-series joists or where the uniform load exceeds the tabulated safe loads for LH-series joists. In these cases custom designs are required. In 1994, the SJI added Joist Load Tables for KCS joists. The joists are part of the K-Series Specification. The CS stands for constant shear. These tables provide moment capacity and shear capacity for 40 different designations in depths ranging from 10 inches to 36 inches. If KCS joists can be selected for a given arrangement of loads, the need for a custom design is eliminated.

1.3 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 capacity for custom products. Currently the following products are offered which are custom designs. All of these products require the design engineer to specify loadings as opposed to using SJI standard designations. 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. VS series: These are joist substitutes and are used for relatively short spans. Their capacities for varying types and spans are tabulated in the Vulcraft catalog.³⁸
3. SLH series: These joists extend the standard load table for DLH-series joists by adding joist depths from 80 inches to 120 inches and increasing the longest available span from 144 feet to 240 feet. Even longer spans are available thru Vulcraft, but are not listed in their tables.
4. Special profile joists: Bow string (curved top chord), scissors and offset ridges on double pitched joists are now 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.
5. Joist Girders: Joist girders are standard SJI components having their own separate SJI specification.³⁴ However, each joist girder is custom designed using loadings specified in the contract documents.

1.4 CURRENT USAGE

At present the usage of steel deck, steel joists and joist girders has already expanded beyond the elementary use contemplated in the original SDI and SJI standard 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 multi-story 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 and malls
- Schools and other academic facilities
- Civic and institutional structures
- Large clear span structures such as fieldhouses and convention centers.

1.5 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, there are three major model codes which may be adopted with or without amendment. These are:

1. The "BOCA National Building Code" published by the Building Officials and Code Administrators International, Inc., Country Club Hills, Illinois.
2. The "Standard Building Code", published by the Southern Building Code Congress International, Inc., Birmingham, Alabama.
3. The "Uniform Building Code", published by the International Conference of Building Officials, Whittier, California.

Each of these model codes adopts the SJI Standard specification in the following ways:

1. BOCA 1999, para. 2205.1:
"General: Steel joists and joist girders used as structural members in floor and roof construction shall be designed and constructed in accordance with SJI *Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders* listed in Chapter 35."
2. SBC 1999, Para. 2205.1
"The design, fabrication and erection of open web steel joist construction shall com-

ply with the following SJI specifications:

1. Standard Specifications for Open Web Steel Joists, K-Series.
 2. Standard Specifications for Longspan Steel Joists, LH-Series, and Deep Longspan Steel Joists, DLH-Series.
 3. Standard Specifications for Joist Girders.”
3. UBC, para. 2205.7 and sec. 2221
 “Open-web Steel Joists and Joist Girders. The design, manufacture and use of steel joist, K, LH, and KLH series and joist girders shall be in accordance with Division IX.”

“In addition to the requirements in the building code, the design, manufacture and use of open web steel joists shall be in accordance with the Standard Specification for Steel Joists, K-Series, LH-Series, DLH-Series and Joist Girders, 1994, published by the Steel Joist Institute, 1205 48th Avenue, Suite A, Myrtle Beach, SC 29577, as if set out at length herein.”

In March of 2000, the International Code Council published the International Building Code (IBC). The IBC was published in co-operation with the Building Officials and Code Administrator International, Inc. (BOCA), the International Conference of Building Officials (ICBO) and the Southern Building Code Congress International, Inc. (SBCCI). The organizations previously published the following codes, respectively, the BOCA National Building Code, the Uniform Building Code and the Standard Building Code.

The requirements for the specification of steel joists are found in Section 2206, which states:

“The design, manufacturing and use of open web steel joists and joist girders shall be in accordance with one of the following SJI specifications:

- “1. Standard Specifications for Open Web Steel Joists, K-Series.
- “2. Standard Specifications for Longspan Steel Joists, LH-Series and Deep Longspan Steel Joists, DLH-Series.
- “3. Standard Specification for Joist Girders.”

There is also a cross reference to the IBC provisions of Section 2211. SJI Load Tables that provide allowable uniform loads per foot for varying spans of various Joist Designations are a part of the respective SJI Specifications by reference.

It is the adoption of the SJI Standard specifications^{35,36} (or their equivalents) 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 on the joist and joist girders in 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 notations.

As of the publication of this text, the Steel Joist Institute is in the process of preparing Load and Resistance Factored Design (LRFD) versions of the Specifications for steel joists and joist girders. Until these specifications are finalized and the publication “Specifications and Load Tables” is revised, SJI has published an interim publication entitled “Guide for Specifying Steel Joist with Load and Resistance Factor Design”. This pamphlet provides design examples and load tables for K-Series joists, LH-Series joists, KCS joists and joist substitutes. The pamphlet also provides information on bridging for K-Series and LH-Series joists and Joist Girder weight tables. The load tables provided in the interim pamphlet provide unit design strengths per foot for various spans for each joist and unfactored, service loads per foot that will provide an approximate deflection of 1/360 of the span. The design strengths are established using the current ASD load table values multiplied by (0.9 x 1.65).

For the design of steel deck, BOCA, SBC and UBC (amended) adopt the “Specification for the Design of Cold-Formed Steel Structural Members” published by the American Iron and Steel Institute.²⁸ The AISI specification is applicable to the design of steel decks and also references the following standards which give additional data:

1. “ASCE Standard, Specification for the Design and Construction of Composite Slabs”, published by the American Society of Civil Engineers, New York, New York.
2. “Design Manual for Composite Decks, Form Decks, and Roof Decks”, published by the Steel Deck Institute, Canton, Ohio.

The SDI Design Manual⁶ presents specifications, a Code of Standard Practice³ and load tables for common deck profiles to which SDI gives standard designations.

1.6 OTHER SPECIFICATIONS

Steel deck, steel joists and joist girders are frequently used in combination with structural steel, which is governed by the “Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design”³⁰ or the “Load and Resistance Factor Design Specification for Structural Steel Buildings”,¹⁷ both published by the American Institute of Steel Construction, Chicago, Illinois.

Metal building systems conform to the “Low Rise Building Systems Manual”¹⁹ 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: “Building Code Requirements for Reinforced Concrete”, ACI 318, published by the American Concrete Institute, Detroit, Michigan.
2. Masonry: “Building Code Requirements for Masonry Structures” ACI-ASCE 530.
3. Masonry: “Recommended Practice for Engineering Brick Masonry”, Brick Institute of America, McClean, Virginia
4. Masonry: “Specification for the Design and Construction of Load-bearing Concrete Masonry, National Concrete Masonry Association, Herndon, Virginia.
5. Wood: “National Design Specification for Wood Construction”, American Forest & Paper Association, Washington, District of Columbia.
6. Wood: “Plywood Design Specification”, American Plywood Association, Tacoma, Washington.

1.7 REFERENCE STANDARDS

Other reference standards are important and useful in designing structures employing steel deck, steel joists and joist girders. First, ASCE 7-98 “Minimum Design Loads for Buildings and Other Structures”, published by the American Society of Civil Engineers²⁰ 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

1. **TECHNICAL DIGEST #3** “Structural Design of Steel Joist Roofs to Resist Ponding Loads”
2. **TECHNICAL DIGEST #5** “Vibration of Steel Joist-Concrete Slab Floors”
3. **TECHNICAL DIGEST #6** “Structural Design of Steel Joist Roofs to Resist Uplift Loads”
4. **TECHNICAL DIGEST #7** “50-Year Digest”
5. **TECHNICAL DIGEST #8** “Welding of Open Web Steel Joists”
6. **TECHNICAL DIGEST #9** “Handling and Erection of Steel Joists and Joist Girders”

The Factory Mutual Research Corporation has published an annual “Approval Guide”² and a series of “Loss Prevention Data Sheets”.¹⁸ 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.

The International Conference of Building Officials publishes Evaluation Reports¹¹. These Reports present product descriptions and tabulated information which show conformity to the applicable UBC standard or standards.

Underwriters Laboratories of Northbrook, Illinois publishes the “Fire Resistance Directory”¹³ 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 particular building and are thus important in determining the required construction of floors and roofs.

1.8 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, capacity 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.

1.9 CONCLUSION

As stated initially it is the intention of this manual 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:

- Roofing types
- Decking types
- Roof loading
- Serviceability considerations.
- Framing considerations
- 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:

- Built-up roofing
- Single-ply roofing
- Liquid applied roofing
- 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 interply 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:

- Ethylene Propylene Diene Monomer (EPDM)
- Poly Vinyl Chloride (PVC)
- Polymer Modified Bitumen

The membranes are delivered to the site in rolls which are seamed together in the field to form continuous roofing. When these roofs are adhered to the substrate, the limitations on area are similar to those of built-up roofs but the requirements for deck stiffness can be somewhat relaxed. When these membranes are loose laid and ballasted, the requirements for both roofing area and deck stiffness can be much less restrictive than the adhered membranes.

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 manufacturers literature is advised.

Structural metal roofs are divided into two main categories: standing seam and through fastener. In both, 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 interlocking 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 fastener 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.

With the exception of structural metal roofs, all low slope roofs rely on deck for support.

Steep slope roofs must be sloped at least one inch per foot to four inches per foot depending on type to insure proper performance. It should be noted that low slope roofs can be used in steep slope applications when properly applied. Steep slope roofs are divided into four categories:

1. Asphalt products: roll roofing and shingles.
2. Clay and concrete tile, slate, and wood shingles.
3. Architectural metal roofs.
4. Structural metal roofs.

The first three of these types rely on a decking for support and many rely on a nailable substrate for attachment. This usually takes the form of a wood deck, nailable insulation on deck, nailing strips alternating with insulation on deck, or a composite of plywood, insulation and deck.

Steep slope roofs when used in the context of the buildings to which this manual is chiefly devoted would be considered feature roofs, whereas a low slope roof would be used in the main areas of roof.

2.3 DECK TYPES

As can be seen from the foregoing discussion all roofing with the exception of structural metal roofs rely on a decking for support. Such decking spans between joists or purlins and supports the weight of the roofing and insulation and the roof live and/or snow loads. Decks are made from steel, concrete or wood.

Steel Decks

Steel decks are formed from sheet steel into fluted units. Steel deck is manufactured from steel conforming to ASTM A611, Grades C, D or E or ASTM 653-94 Structural Quality grade 33 or higher. Steel deck is supplied as galvanized, aluminized or prime painted. The primer coat is only intended to protect the steel for a short period of ordinary atmospheric conditions. The Steel Deck Institute recommends the field painting of prime painted deck especially where the deck is exposed. It recommends the use of galvanized deck (G60 or G-90) 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" deep but deeper units are also available. The Steel Deck Institute identifies the standard profile for 3 inch deck as 3 DR. The Steel Deck Institute has also identified three standard profiles for 1-1/2" steel deck, which are narrow rib, intermediate rib and wide rib and has published load tables for each profile for gages varying from 22 to 18 gage. These three profiles NR, IR, and WR, correspond to the manufacturers' designations of A, F and B. A comparison of weights for each profile in various gages shows that weight to strength ratio for each profile is most favorable for wide rib deck and least favorable for narrow rib deck. 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. In the South, low roof loads and thinner insulation make the intermediate profile common. Where very thin insulation is used, narrow rib deck may be required although this is not a common pro-

file. In general, the lightest weight per square foot deck consistent with insulation thickness and span should be used.

In addition to the load, span and thickness relations established by the 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, see (Table 2.3.1). And lastly, Factory Mutual lists maximum spans for various profiles and gages in its Approval Guide, (see Table 2.3.2.).

Factory Mutual 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 "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 self drilling self tapping. The Steel Deck Institute in its "Specifications and Commentary for Steel Roof Deck"²⁷ requires a maximum attachment spacing of 18 inches along supports. Factory Mutual 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 carry lateral loads to the bracing.

Diaphragm strength is a function of the deck profile, thickness and attachments, both to the supports and at sidelaps. Diaphragm capacity tables have been developed by the Steel Deck Institute.⁷ Tables of diaphragm shear capacities for various Vulcraft decks are published in their catalog "Steel Floor and Roof Deck".³⁷ It should be noted that in addition to following these values, provisions must be made for diaphragm chords and a means to transfer the diaphragm reactions into the lateral bracing. The diaphragm tables also provide stiffness coefficients that can be used in calculations of diaphragm deflection. Further information on steel deck diaphragms is presented in Chapter 4.

ROOFS

Recommended Maximum Spans for Construction and Maintenance Loads Standard 1-1/2 Inch and 3 Inch Roof Deck				
	Type	Span Condition	Span Ft.-In.	Maximum Recommended Spans Roof Deck Cantilever
Narrow Rib Deck	NR22	1	3'-10"	1'-0"
	NR22	2 or more	4'-9"	
	NR20	1	4'-10"	1'-2"
	NR20	2 or more	5'-11"	
	NR18	1	5'-11"	1'-7"
	NR18	2 or more	6'-11"	
Intermediate Rib Deck	IR22	1	4'-6"	1'-2"
	IR22	2 or more	5'-6"	
	IR20	1	5'-3"	1'-5"
	IR20	2 or more	6'-3"	
Wide Rib	WR22	1	5'-6"	1'-11"
	WR22	2 or more	6'-6"	
	WR20	1	6'-3"	2'-4"
	WR20	2 or more	7'-5"	
	WR18	1	7'-6"	2'-10"
	WR18	2 or more	8'-10"	
Deep Rib Deck	3DR22	1	11'-0"	3'-5"
	3DR22	2 or more	13'-0"	
	3DR20	1	12'-6"	3'-11"
	3DR20	2 or more	14'-8"	
	3DR18	1	15'-0"	4'-9"
	3DR18	2 or more	17'-8"	

Table 2.3.1 Steel Deck Institute Recommended Spans

ROOFS

Vulcraft Division - Nucor Corp 2100 Rexford Rd Charlotte NC 28211	
Trade Name:	Type 1.5A
Type:	Type A Narrow Rib
Thickness:	18, 20 or 22 ga. (0.0474, 0.0358, 0.0295 in. [1.2, 0.91, 0.75 mm])
Depth:	1.5 in. (38 mm)
Max Spans:	4 ft 10 in. (1.5 m) for 22 ga. (0.0295 in. [0.75 mm]) 5 ft 3 in. (1.6 m) for 20 ga. (0.0358 in. [0.91 mm]) 6 ft 0 in. (1.8 m) for 18 ga. (0.0474 in. [1.20 mm])
Trade Name:	Type 1.5F
Type:	Type F Intermediate Rib
Thickness:	18, 20 or 22 ga. (0.0474, 0.0358, 0.0295 in. [1.2, 0.91, 0.75 mm])
Depth:	1.5 in. (38 mm)
Max Spans:	4 ft 11 in. (1.5 m) for 22 ga. (0.0295 in. [0.75 mm]) 5 ft 5 in. (1.7 m) for 20 ga. (0.0358 in. [0.91 mm]) 6 ft 3 in. (2.0 m) for 18 ga. (0.0474 in. [1.20 mm])
Trade Name:	Type 1.5B, 1.5BI
Type:	Type B Wide Rib
Thickness:	18, 20 or 22 ga. (0.0474, 0.0358, 0.0295 in. [1.2, 0.91, 0.75 mm])
Depth:	1.5 in. (38 mm)
Max Spans:	6 ft 0 in. (1.8 m) for 22 ga. (0.0295 in. [0.75 mm]) 6 ft 6 in. (2.0 m) for 20 ga. (0.0358 in. [0.91 mm]) 7 ft 5 in. (2.3 m) for 18 ga. (0.0474 in. [1.20 mm])
Trade Name:	B0.0334
Type:	Type B Wide Rib
Thickness:	0.0334 in. (0.85 mm)
Depth:	1.5 in. (38 mm)
Max Spans:	6 ft 3 in. (1.9 m)
Trade Name:	B0.0376
Type:	Type B Wide Rib
Thickness:	0.0376 in. (0.96 mm)
Depth:	1.5 in. (38 mm)
Max Spans:	6 ft 8 in. (2.0 m)
Trade Name:	B0.0398
Type:	Type B Wide Rib
Thickness:	0.0398 in. (1.01 mm)
Depth:	1.5 in. (38 mm)
Max Spans:	6 ft 10 in. (2.1 m)

Table 2.3.2 Vulcraft Factory Mutual Data

Concrete Deck

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

- a) Insulating lightweight concrete.
- b) Gypsum concrete.
- c) 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 100–120 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”. Aggregates for such concrete are covered in ASTM Specification “C332 Standard Specification for Lightweight Aggregates for Insulating Concrete”.

Roofs of insulating lightweight concrete rely on the substrate for the capacity 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 series. Because of the high moisture content of insulating concrete, it is necessary to provide slot vented decks so that water vapor can dissipate from both the top and bottom sides of the concrete. Vulcraft 0.6 and 1.0 C-series 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 I 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. Diaphragm values for both Type I and Type II

construction are presented in tables in the Vulcraft Deck Catalog. 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.

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 untopped. 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 capacity 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.

Wood Deck

Wood decks are available in the following general categories:

1. Plank and laminated wood decks.
2. Plywood 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 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 capacity. This capacity 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 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.¹ Plywood 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.

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 manufacturers 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 or 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 loads.

Roof dead loads represent the self-weight of the roof construction. They include the weight of the roofing mem-

brane, 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 Standard ASCE 7-98 "Minimum Design Loads for Buildings and Other Structures" in tables C3-1 and C3-2. This reference will be referred to in the remainder of this chapter as ASCE 7-98.

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

Roofing

Built-up roof gravel surface	5-1/2 to 6-1/2 lbs/sq. ft.
Adhered or attached single ply membrane	1 to 2 lbs/sq. ft.
Ballasted membrane	10 to 15 lbs/sq. ft.
Liquid applied	2 to 5 lbs/sq. ft.
Structural metal roofs	1 to 2 lbs/sq. ft.

Decking

Steel decking	2 to 3 lbs/sq. ft.
Lightweight insulation conc. (30 pcf) and steel deck	10 lbs/sq. ft.
Gypsum concrete (50 pcf)	15 lbs/sq. ft.
Precast concrete	10 to 20 lbs/sq. ft.
2" nominal wood planks	5 lbs/sq. ft.
3" nominal wood planks	9 lbs/sq. ft.
Plywood per 1/8 of thickness	0.4 lbs/sq. ft.
Structural wood fiber board	3 to 6 lbs/sq. ft.

Insulation (per inch of thickness)

Cellular glass	0.7 lbs/sq. ft.
Fibrous glass	1.1 lbs/sq. ft.
Fiberboard	1.5 lbs/sq. ft.
Perlite	0.8 lbs/sq. ft.
Polystyrene foam	0.2 lbs/sq. ft.
Urethane foam	0.5 lbs/sq. ft.

Collateral 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 is less than 150 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 150 pounds, they should be accounted for by special designs using concentrated loads.

Roof Live Loads

Code specified roof live loads are uniform loads used for the design of roofs to account for occupant use and environmental loads such as rain, snow or ice. Where no distinction is made in the code as to the source of the live load, it is incorrect to speculate as to the mix of load sources which are intended to make up the live load. When a live load is specified by code for a given locality it is reasonable to assume that the live load accounts for the total of superimposed loads which the roof is expected to carry. In some codes, procedures are given to increase roof live loads at roof level changes and obstructions. These procedures are intended to account for snow drifting.

Snow Loads

Many codes, such as SBC 2000 and ASCE 7-98, have eliminated the confusion caused by using roof live loads to account for snow by creating a specific category called snow load. This snow load is calculated using a weight of ground snow taken from a map which shows the expected annual accumulation with a given recurrence interval. However, these maps do not usually provide ground snow data for areas of higher variation such as mountainous regions, areas around the Great Lakes and areas of localized changes in terrain identified as high country. With these exceptions, the calculation of design 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. Some methods also account for the change in expected snow density in drifts. These procedures give a much more accurate picture of the anticipated loads on roofs due to snow.

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 being 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. Codes vary as to whether this relief may be other drains, special relief drains or overflow at 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 particular drain must be designed for the depth of water which accumulates if that particular drain is plugged. It is the opinion of the authors that the plugged drain loading is a load case which is to be taken separately from other snow or live loadings.

ASCE 7-98 also recognizes the potential for rain on snow in certain localities and recommends the addition of 5 psf to snow loads where $P_g \leq 20$ psf (but not zero) and roof slopes are less than 1/2" per foot. This requirement is not generally part of local codes or the national model codes.

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.

Live Load Reductions

Live load reductions are allowed by building codes to account for the probability of the occurrence of full live loading on a particular element. Thus live load reductions vary with the type of expected load and the tributary area of the element to be designed. In general, codes do not allow the reduction of live load or snow load for framing members in the roof. The one exception is when codes give a minimum live load based on tributary area. This amounts to a live load reduction on members with relatively large tributary areas.

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 buildings, but frequently these forces are intended for gross design of the lateral force resisting system. More detail is provided by Factory Mutual in three Loss Prevention Data

Sheets 1-7 “Wind Forces on Building and Other Structures”, 1-28 “Roof Systems and Roof Deck Securement”, and 1-29 “Above-Deck Roof Components”. These guides cover the securement of the roof deck to the supports and the attachment of the roof membrane and insulation to the roof deck. They divide roof and deck assemblies into two classes, Class 1 and Class 2. The field of roof uplift pressures are determined from maps and charts and the pressure is a result from the basic wind speed, building height and exposure. The roof approval rating is determined by multiplying the field roof pressure by 2.0. The minimum approval rating is I-60 and the approval ratings increase in increments of 15. The higher approval ratings are required for edge and corner zones.

Vulcraft decks are approved by Factory Mutual and can be used in FM Class I, I-90 roof construction.² Additionally they can be used following Underwriters Laboratories (UL) constructions which are rated for wind uplift – Class 90. The most common UL constructions are listed below.

1. Construction No. 143: C deck.
2. Construction No. 155: C deck, galvanized.
3. Construction No. 157: 1.5 F and B deck.

These constructions are listed in the “Roofing Materials and Systems Directory”, published by Underwriters Laboratories, Inc.

ASCE 7-98 gives similar criteria for edges and corners. Ballasted roofs present a different situation. With ballasted schemes the edges and corner zones have increased ballast to counter balance 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 requirements: “Steel deck units shall be anchored to supporting members ... to resist the following gross uplifts: 45 pounds per square foot for eave overhang; 30 pounds per square foot for all other roof areas.” It should be noted that these pressures are minimums and may not be appropriate for edges and corners. Roof decks are attached to supports by fasteners or welds. The capacity of fasteners can be evaluated using manufacturer’s published test values. Welds can be evaluated using the procedure in the AISI Specification. The

evaluation is based on Equation E2.2.2-1 thru E2.2.2-3 which is located in Section E2 of the Specification.

A frequently cited standard for resistance of a roofing system to uplift is UL 580 “Tests for Uplift Resistance of Roof Assemblies.” This standard gives procedures to test sample roof assemblies and establishes four rating classifications: Class 15, Class 30, Class 60 and Class 90. These Classes have associated with them positive and negative test pressures and durations along with test failure criteria. The Class values of 15-90 should not be confused with wind velocities or design uplift pressures. The Class values are only a measure of relative performance and are only a prediction not a guarantee of in place performance.

The engineer of record must specify net uplift loads required for the joist manufacturer to incorporate in the design.

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.

Concentrated Loads

The treatment of concentrated loads and the specification of loading on joists and joist girders is covered in Chapters 5 and 6.

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 and 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.

Steel Deck Institute:

- a. Span over 240, uniformly distributed live load.
- b. Span over 240, 200 lb. concentrated load at mid-span on a one foot section of deck.

Steel Joist Institute:

- a. Span over 360, design live load where plaster

- ceilings are attached or suspended.
- b. Span over 240, design live load in all other cases.

National Roofing Contractors Association (NRCA):²¹

- a. Span over 240, deck deflection at full uniform load.
- b. Span over 240, deck deflection, 300 pound concentrated load at midspan.

The NRCA also points out that the roof structure in its cambered or deflected geometry must provide positive slope to drains. 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.

Factory Mutual:

Span over 200, 300 pound concentrated load at mid-span.

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.

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 gives the following recommendations for the location of roof expansion joints.

- “• Where expansion or contraction joints are provided in the structural system.
- Where steel framing, structural steel, or decking change direction.
- Where separate wings of L, U, T or similar configurations exist.
- Where the type of decking changes; for example, where a precast concrete deck and a steel deck abut.
- Where additions are connected to existing buildings.
- At junctions where interior heating conditions change, such as a heated office abutting unheated warehouse, canopies, etc.
- Where 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”.¹² 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 T_w , the temperature exceeded only 1% of the time during summer months, T_m , the mean temperature during the normal construction season and T_c , the temperature exceeded 99% of the time during winter months. The design temperature change is the larger of the two temperatures 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.”

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

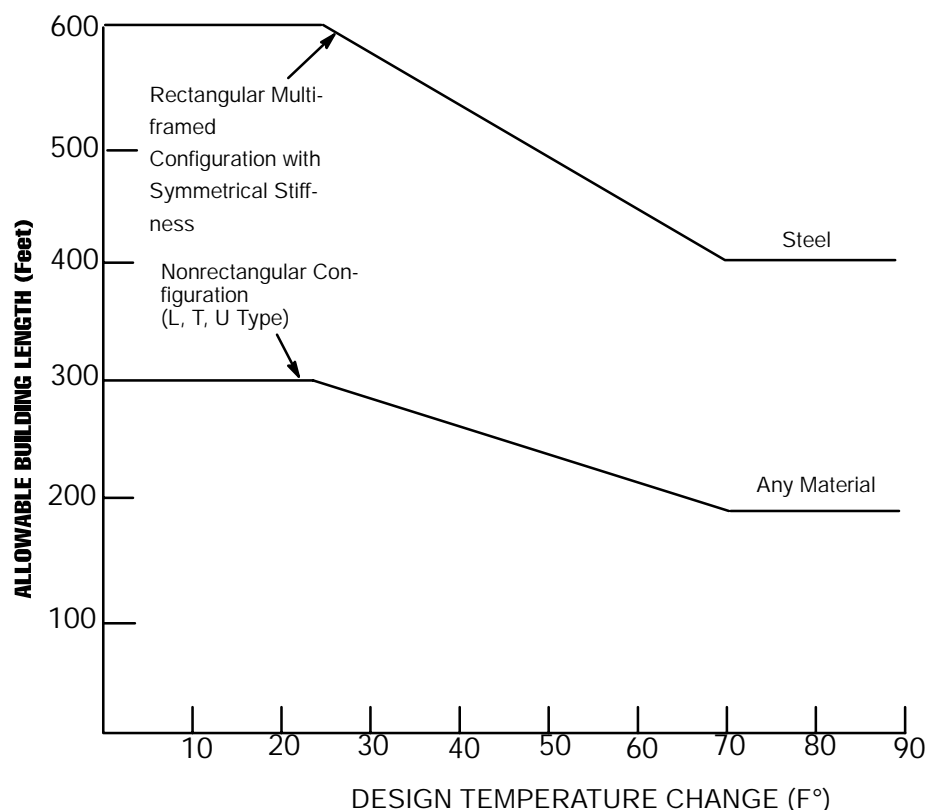


Fig. 2.5 Expansion Joint Spacing Graph

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

simple squares and rectangles. In large roofs they are placed at intervals of 150–250 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 fastener 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.

Roof Slope

Roof slope is also a factor in roofing performance. For membrane roofs, 1/4" pitch per foot is generally recommended. For structural steel roofs the minimum pitches are on the order of 1/4" per foot for standing seam roofs and 1/2" per foot for through fastener roofs.

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 50'x50' 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 due to the fact that 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 due to the fact that 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.

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. Vulcraft has provided examples of bay weight per square foot for various

combinations of joist and girder spans in their catalog. These examples can provide the designer with valuable insight as to bay size costs and span directions. However, the designer is encouraged to examine alternate framing schemes for a given project, and to contact Vulcraft to discuss the least expensive system. Prices can vary for joists and joist girders depending upon plant work load 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. The joist should be spaced to maximum values of the deck, but with spaces not greater than those recommended for construction practice as contained in the Steel Deck Institute specification. In addition, the designer should check to see if Factory Mutual requirements must be followed. If so, then the Factory Mutual recommended joist spacings should be followed.

If a standing seam roof is being used, typically a 5 foot joist spacing is used. This is due to the fact that 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'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 catalog; 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.

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 Fig. 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 specifications. 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.

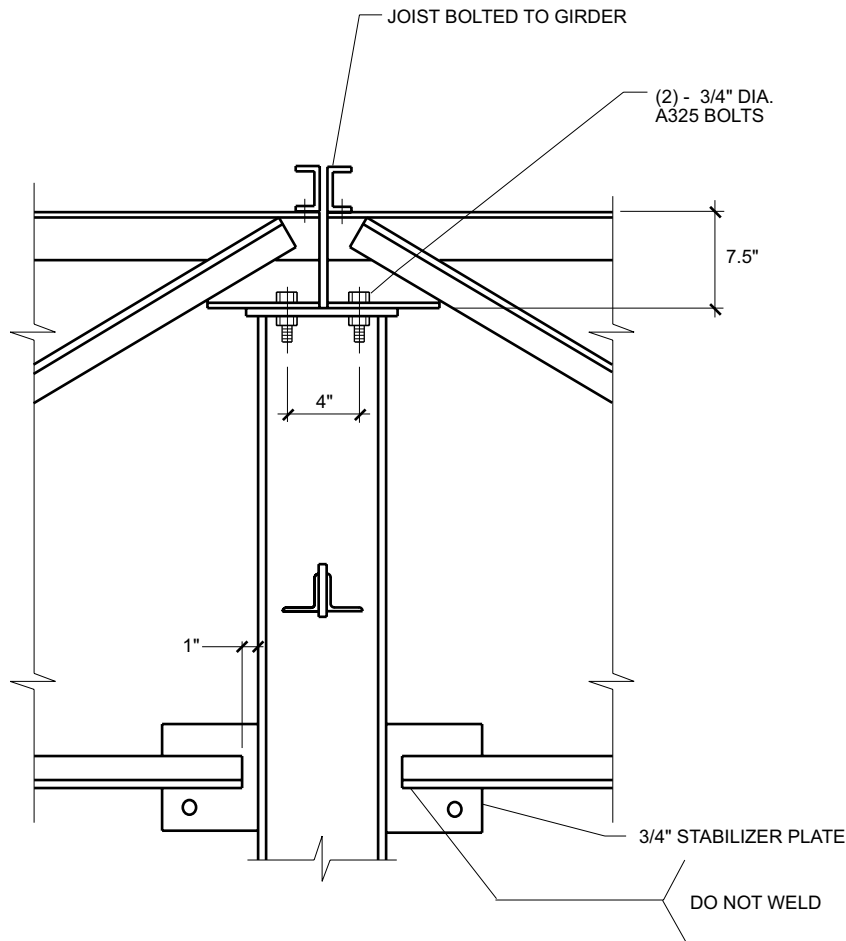


Fig. 2.7.1 The Basic Connection

The Basic Connection is also used when framing joist girders into the weak axis of wide flange columns. This condition is shown in Fig. 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.

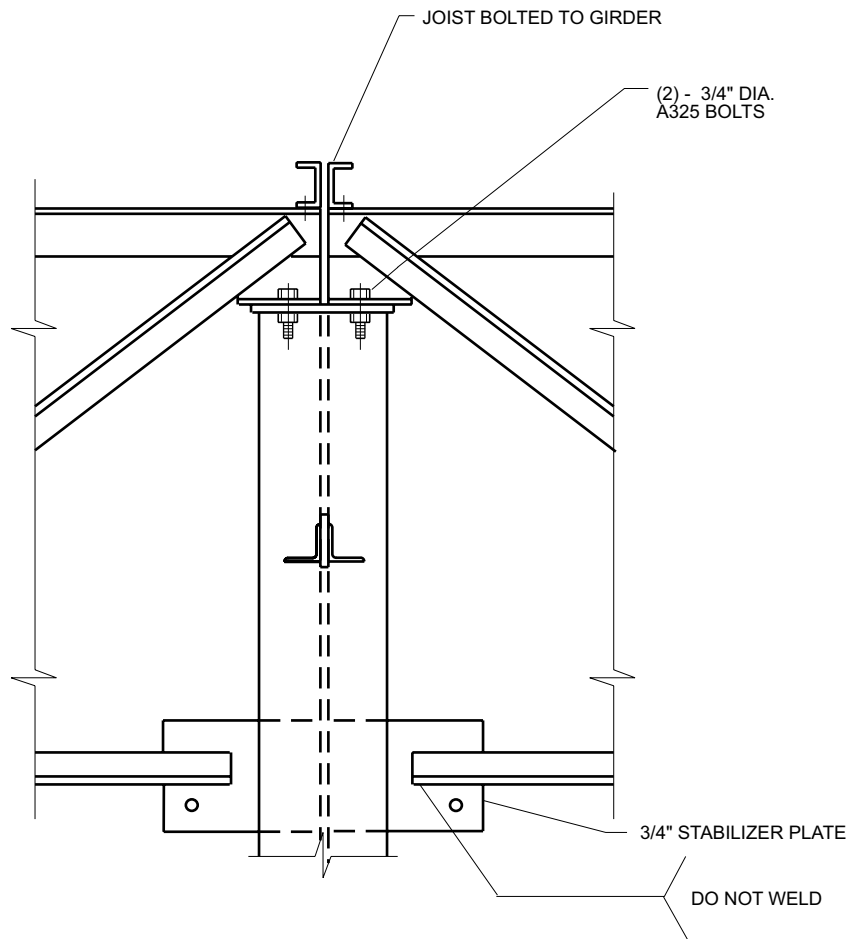


Fig. 2.7.2 The Basic Connection

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 design 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.

1. Cast-in-place concrete on steel deck.
2. Precast slabs.
3. Plywood and wood decking.

Cast-In-Place Concrete on Steel Deck

The following steel deck types are used in floor construction:

1. Vulcraft C-series, Conform 0.6", 1.0", 1.3", 1.5", 2" and 3" deep steel form deck.
2. Vulcraft VLI-series, 1.5", 2" and 3" composite floor deck.
3. Vulcraft VLP-series 1.5", 2" and 3" composite decks with bottom cover plates which form UL rated electrical raceways under the deck profile.
4. Vulcraft VLPA-series, 1.5", 2" and 3" composite decks with perforated bottom cover plates. Batts of acoustical insulation can be inserted in the cells for sound control.

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^{6,26} require-

ment for loads during construction is to design the deck 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 3/4" 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, in its catalog "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 it is appropriate for most situations. Neither painted roof nor floor deck is appropriate in certain high moisture environments. Unfinished metal 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 catalog based on an unshored condition to carry the weight of concrete and construction loads. The Vulcraft catalog gives allowable uniform load capacities using four criteria: allowable stress of 36,000 psi, deflection of span over 240, deflection of span over 180, and "WI", the maximum weight of concrete and deck for single, double and triple spans.

In the past, many building codes and the SJI Code of Standard Practice limited the spacing of joists in floors to two feet on center. At that time the industry standard form was a 9/16" deep, 28 gage corrugated metal form deck (.6C28). It was commonly identified as centering. The usual concrete depth was 2-1/2" overall. Current practice is to space the joists further apart to minimize the number of pieces to be erected. Also thicker concrete slabs are now recommended to control floor vibration. This increased span and load has prompted greater use of 26 gage centering and the one inch deep profile, although 28 gage is still

used in many situations. The spacing of joists is sometimes limited by the load capacity of standard K-series joists, but this can be overcome by the use of custom designed joists as will be seen later.

The Vulcraft catalog also presents load tables for the finished slabs. For form decks, flexure reinforcement for superimposed loads is provided using welded wire fabric. 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 insure the correct positioning of the fabric, must be specified on the drawings to insure 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. The building designer should consider the applicability of the ACI criteria when selecting reinforcement for the concrete slab. The SDI recommends a minimum reinforcement area of 0.00075 times the area of concrete above the deck. The minimum reinforcement allowed is 6x6-10/10 (6x6-W1.4xW1.4) welded wire fabric. This minimum reinforcement would not normally be adequate to resist negative moments at supports or transverse negative moments over girders against which the deck bears.

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 is 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". ACI 318 should be used as the standard for acceptable reinforcement types.

As discussed in the section on form deck, 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 the girders.

Vulcraft has published load tables giving superimposed live load capacities for various slab thicknesses, gages, profiles and spans for both normal weight and light-weight concrete. These tables also give the maximum span using the SDI criteria for one, two and three span conditions in an unshored 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 which are presented elsewhere in the Vulcraft catalog. Example 3.1 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 unshored deck is almost universally preferred. The tables should be used to select a proper thickness and profile based on an unshored condition.

The SDI Specification and Commentary 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 de-icing 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.

2. Cantilevers require special top reinforcement. 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.

Manufacturer's load tables such as those in the Vulcraft catalog give 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 "Composite Deck Design Handbook" published by the Steel Deck Institute. As its title indicates it will cover other areas as well as the treatment of line and concentrated loads.

Concentrated Loads

The SDI "Composite Deck Design Handbook" provides a method for analyzing concentrated loads composite deck in Part E of the Handbook. The method provided (see, also Reference 10) 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 defined parameters and effective widths are:

P = concentrated load

b_2 = load dimension, perpendicular to span, inches

b_3 = load dimension, parallel to span, inches

t_c = thickness of concrete over top of deck, inches

t_t = thickness of any permanent durable topping, inches

h = dimension, top of concrete to bottom of deck, inches

b_m = effective slab width, inches

$$= b_2 + 2t_c + 2t_t$$

b_e = effective slab width, inches

$$= b_m + 2(1.0 - x/L)x, \text{ for single span bending}$$

$$= b_m + 4/3(1.0 - x/L)x, \text{ for continuous span bending}$$

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

where,

L = the center to center span, inches

x = the distance from the center of the support, $\geq h$, inches

$$b_e \leq 8.9 (t_c/h), \text{ feet}$$

The transverse moment is equal to $(Pb_e)/(15w)$,
 $w = L/2 + b_3 \leq L$.

Examples 3.2.1 and 3.2.2 illustrate key issues in proper deck selection, i.e. unshored construction, web crippling, uniform load capacity and live load capacity and points out the importance of attentive use of the Tables.

Example 3.2.1 Composite Floor Slab with a Line Load

Design a composite steel floor deck with a clear span of 10'-0" to support an 80 psf live load and a 650 plf concentrated line dead load. The line load runs perpendicular to the deck span and is located 2 feet from the left support. A two hour restrained assembly fire rating is required. 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 catalog select a 3-1/4 inch lightweight concrete thickness above the deck. This thickness can be used unprotected.
2. Try a 3VLI20 deck. From the 3VLI load tables it can be seen that the Type 20 deck can support a uniform live load of 149 psf.
3. Determine if the 3 VLI20 deck can safely support 80 psf with the concentrated line load of 650 plf.

Using statics determine the equivalent uniform load for bending:

Reactions:

$$V_L = (80)(5) + (650)(8)/(10) = 920 \text{ lbs./ft.}$$

$$V_R = (80)(5) + (650)(2)/(10) = 530 \text{ lbs./ft.}$$

Using statics the point of zero shear is located 6.63 feet from the right support. Therefore the maximum moment equals:

$$M_{\max} = (6.63)(530) - (80)(6.63)^2/2 \\ = 1756 \text{ ft.-lbs./foot}$$

The equivalent uniform load based on moment is found by equating the maximum moment to $WL^2/8$.

$$W_{eq.} = 8M_{max}/L^2$$

$$W_{eq.} = (8)(1756)/(10)^2 = 140 \text{ psf}$$

Since the equivalent uniform load is less than the allowable uniform load for the system (149 psf) the VLI20 deck is adequate for moment.

Check the deck for shear:

A conservative value for the maximum shear capacity can be determined by examining the load tables. For the 3VLI20 deck with 6-1/4 inch light weight concrete the maximum tabulated end shear is found from the uniform load tabulated for the shortest clear span. For this case:

$$V_{max} = (221)L/2 = (221)(8)/2 = 884 \text{ lbs./ft.}$$

Since the 3VLI20 deck does not check for shear using the simplified procedure, use the SDI deck shear equation:

$$V_{conc} = (1.1)(f'_c)^{1/2}(0.75)$$

The 0.75 factor is a reduction factor for lightweight concrete.

The line load is continuous thus b_e equals 12 inches and V equals 920 lbs./ft.

Based on the 3VLI profile, $A_c = 44.3 \text{ in.}^2/\text{ft.}$

Solving:

$$V_{conc} = 1.1(3000)^{1/2}(0.75)44.3 = 2002 \text{ lbs.}$$

$V_{deck} = 2140 \text{ lbs.}$ per Figure 9 of the SDI Composite Deck Design Handbook

$$V_{total} = 2002 + 2140 = 4142 \text{ lbs.} > 884 \text{ lbs.}$$

Therefore the 3VLI20 deck is adequate.

The designer should also check the allowable reaction for the deck for web crippling.

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

Based on this criterion the end reaction for the 10 foot span (assuming a three span condition) equals $0.4wL = (0.4)(46+20)(10) = 264 \text{ lbs./ft.}$ The interior reaction

equals $1.1wL = (1.1)(46+20)(10) = 726 \text{ lbs./ft.}$ From the web crippling data in the Vulcraft deck catalog these reactions require the minimum bearing lengths as follows:

For VLI 20 deck:

Exterior reactions: 1.5 inches.

Interior reactions: 2.5 inches.

The designer must make sure that suitable bearing is provided for the design.

Example 3.2.2 Composite Floor Slab with a Concentrated Load

Determine if a concentrated dead load of 1000 pounds 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.

Given: $b_2 = b_3 = 4 \text{ in.}$, $t_c = 3.25 \text{ in.}$, $t_t = 0 \text{ in.}$

Span = 10 ft., $h = 3.25 + 3.0 = 6.25 \text{ in.}$

Solution:

1. Determine the effective slab width for flexure and shear.

For shear:

$$b_m = b_2 + 2t_c + 2t_t$$

$$b_m = 4 + 2(3.25) = 10.5 \text{ in.}$$

$$b_e = b_m + \left(1.0 - \frac{x}{\ell}\right)x$$

$$x = h$$

$$b_e = 10.5 + (1.0 - 6.25/120) 6.25 = 16.42 \text{ in.}$$

For flexure:

$$b_e = b_m + 2 \left(1.0 - \frac{x}{\ell}\right)x$$

$$x = \frac{\ell}{2} = 60 \text{ in.}$$

$$b_e = 10.5 + 2 \left(1.0 - \frac{60}{120}\right) 60 = 70.5 \text{ in.}$$

$$b_{e(max)} = 8.9 (t_c/h)$$

$$b_e = 8.9 \left(\frac{3.25}{6.25}\right) = 4.6 \text{ ft.} = 55.54 \text{ in. (controls)}$$

2. Check the shear strength:

For the uniform load: $R =$

$$(43 + 80)10(0.5) + 1000 \left(\frac{120 - 6.25}{120} \right) \left(\frac{12}{16.42} \right)$$

$$= 615 + 693 = 1308 \text{ lbs.} < 4142 \text{ lbs. from the previous example. } \mathbf{o.k.}$$

3. Check bending:

For the uniform load:

$$M_u = wL^2/8 = (80)(10)^2/8 = 1000 \text{ ft.-lbs./ft.}$$

For the concentrated load:

$$b_e = 55.54 \text{ inches} = 4.628 \text{ feet.}$$

$$M_c = PL/4 = (1000)(10)/(4) = 2500 \text{ ft.-lbs.}$$

$$M_c/\text{ft.} = 2500/4.628 = 540 \text{ ft.-lbs./ft.}$$

$$M_{\max} = M_u + M_c = 1540 \text{ ft.-lbs./ft.}$$

The equivalent uniform load:

$$\begin{aligned} W_{eq} &= 8 M_{\max}/L^2 = (8)(1540)/(10)^2 \\ &= 123 \text{ psf} < 149 \text{ psf } \mathbf{o.k.} \end{aligned}$$

4. Determine required distributional steel

$$\begin{aligned} w &= \frac{\ell}{2} + b_3 \\ &= 64 \text{ in.} \end{aligned}$$

$$\begin{aligned} M &= P b_e / 15w = \frac{1000(55.24)(12)}{(15)(64)} \\ &= 690.5 \text{ in.-lbs./ft. or } 1.2(690.5) \\ &= 828.6 \text{ factored} \\ &= \text{Check SDI minimum reinforcement} \\ &6X6-W1.4X1.4, \text{ As per foot} = 0.029 \text{ in.}^2/\text{ft.} \end{aligned}$$

w/mesh @ 2" from top of slab

$$\begin{aligned} a &= A_s(f_y)/0.85(f'_c)b \\ &= 0.029(60000)/0.85(3000)12 = 0.057 \end{aligned}$$

$$\begin{aligned} \phi M_n &= \phi(A_s F_y (2 - a/2)) \\ &= 0.85(0.029)60000 \left(2 - \frac{(0.057)}{2} \right) \\ &= 2915 \text{ in.-lbs./ft.} \end{aligned}$$

$$2915 > 828.6 \text{ } \mathbf{o.k.}$$

For a detailed discussion and other design examples refer to the SDI Handbook.

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 light weight concrete). Normal weight concrete is most commonly used. However, the lesser density of structural lightweight concrete is often used to 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" 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 catalog "Steel Floor and Roof Deck".

Pre-cast Slabs

The pre-cast 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 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 catalogs. The self-weight of framing must be computed on a job by job basis.

Collateral 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. That is 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 capacity but also shear capacity. In the case of concentrated loads shear may govern the design when the load is placed near the support. The shear capacity 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

Building codes provide 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.

American Society of Civil Engineers (ANSI/ASCE 3-91).²⁹

- a. Span over 180 not more than $3/4"$, uniformly distributed weight of wet concrete and steel deck.
- b. Span over 240 to span over 480 including creep for service loads depending on susceptibility of collateral elements to damage.

Steel Joist Institute:

- a. Span over 360, design live load.

It should be noted that many building codes also give deflection limits.

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 of 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 criteria 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 1/4 to 1/2 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 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. 3¹⁴ that framing members be held to a maximum deflection of span over 360 (1" 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 200-250 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 multi-story buildings little if any options exist for the engineer to select the optimum bay size. Architectural requirements and building foot print 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' 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.

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

When custom designs are used for floor joists it is likely that the joists will require 5 inch seat depths since the joist loads due to wide spacings will require relatively larger chords. The specifying engineer can refer to Tables 6.1 and 6.2 to determine seat depth requirements or the engineer can check with the local Vulcraft representative.

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. .

FLOORS

JOIST SPAN (FT.)	GIRDER SPAN (FT.)	JOIST SPACING (FT.-IN.)	JOIST DEPTH (IN.)	GIRDER DEPTH (IN.)	TOTAL WEIGHT OF JOISTS, GIRDERS AND BRIDGING (PSF) FOR LOADS BELOW			
					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

Provided in Table 3.5.2 are duct sizes which can be accommodated in standard joists of various depths.

VG type joist girders can often be used to advantage when mechanicals are run between the joists and through

the joist girders. This joist girder type aligns an open panel in the girders with the space between joists. VG girders are not practical for floor systems where close joist spacings are used.

Joist Depth (Inches)	Panel Length (Inches)	Maximum Span (Feet)	Round Inches	Square Inches	Rectangular (Ins. x Ins.)	Flat Oval (Ins. x Ins.)
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 Joists - 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 particular 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.
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.

Composite Joists

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 must take into account:

- a. The potential for dead load deflection of the unshored non-composite section.
- b. Floor vibration considerations.
- c. The added cost of the shear connectors required for composite action.

Because the use of composite joists requires custom design of the joists, it is important for the building designer 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.

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 such studs results from water which accumulates between the deck and the framing members. It is usually not recommended that shear connectors be welded through two deck thicknesses. 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 num-

ber 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 reduced if structural lightweight concrete is used. 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 catalog. In the catalog the maximum and minimum flute widths are given so that the average may be computed. The selection of the type and number of shear connectors is the responsibility of the building designer. The construction documents should state if full or reduced values were used and what deck profile was used in establishing the stud values. They 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.

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 4 of AISC Manual of Steel Construction (ASD), and in Part 9 of the AISC Manual of Steel Construction (LRFD). 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.

CHAPTER 4

LATERAL LOAD SYSTEMS

4.1 INTRODUCTION

In this chapter the various means of providing lateral load resistance systems for single story and multistory joist and joist girder buildings are presented. There are several systems of lateral support available to the structural engineer. These include:

1. Roof and floor diaphragms used in combination with vertical steel bracing or shear walls.
2. Horizontal roof bracing used in combination with vertical steel bracing or shear walls.
3. Rigid frames with moment connections.

The most economical system to be used is dependent upon several variables. These include the building geometry, roofing types, the type and magnitude of loading, and the number of stories in the structure. The loadings that must be considered in the design of the lateral load resisting system include wind, seismic, earth pressure and column stability loads. Each of the above systems and the choice of the “best” system is discussed in this chapter.

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 in-plane 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

The diaphragm can be classified based upon the type of materials used to comprise the diaphragm. Those 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 the diaphragm system is controlled predominantly by:

1. The panel configuration, i.e. the height of the major corrugations and spacing of corrugations within the panel.
2. The span of the individual 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 and is 300 lbs. at 3 feet on center for K-series joists and ranges from 120 lbs. per foot to 250 lbs. per foot based on chord size for LH and DLH joists.

Most commonly, welding is used to connect the deck to the structural members. The Steel Deck Institute requires 5/8 inch arc spot welds (puddle welds) or a 3/8x3/4" elongated weld. The 3/8x3/4" elongated welds are required for A and F decks because the 5/8 inch arc spot welds cannot be made in the narrow rib of these decks. For ICBO approvals, the arc spot weld size requirements vary somewhat between various manufacturers. Weld patterns vary de-

pending 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 inch spacing. The maximum number of welds per 36 inch sheet width is 7 or one every 6 inches. These patterns are commonly designated as 36/3 and 36/7. Other patterns are 36/5 and 36/4. The reader is referred to the Vulcraft Steel Floor and Roof Deck Catalog for a full description of fastener patterns.

A final comment should be made regarding the welding of the deck to structural members using welding washers. The SDI, AISI, and the AWS specifications do not require the use of welding washers for decks equal to or greater than 0.028 inches in thickness. Since 22 gage deck is approximately 0.0295 inches in thickness welding washers are not required. The SDI Specifications go further, in fact, 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.

Power driven fasteners and self drilling screws are also used for the deck-to-structural attachment. 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. Questions concerning quality control are basically eliminated since their strength is very dependable once they are installed.

The major 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 is generally more as compared to welding the deck.

Sidelap connections are made by welding, button punching, or self drilling screws. Vulcraft, and other manufacturers of steel deck, do not recommend the welding of sidelaps of decks of 22 gage or less. Sidelap welds can be made on 22 gage deck. However, extreme care must be exercised even with proper welding equipment. 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 the use of wood nailers which are generally bolted to the joists. 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 designer and the joist manufacturer 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 designer 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 longitudinal force transfer from the nailer to the joist.

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. When attaching wood decking directly to joists, care must be taken to insure that the fasteners used 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 would require nailers on the panel joists which run 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. At about the same time the Army, Navy and Air Force published what has been referred to as the Tri-Services design manual on seismic design.²⁴ The manual includes information on steel and concrete diaphragms. The Steel Deck Institute published its first diaphragm design manual and load tables in 1981. A revised and expanded diaphragm manual⁷ was published by the SDI in 1987. In addition to these organi-

zations, various manufacturers of steel deck conducted their own research and published diaphragm strength and stiffness values of their own.

The American Iron and Steel Institute in its 1996 Specification for the Design of Cold-Formed Steel Structural Members included recommended factors of safety for light gage diaphragms. Basically these factors of safety are in agreement with the ICBO Evaluation Reports for welded diaphragms, and the Steel Deck Institute for mechanically fastened systems.

Currently, designers of steel deck systems principally rely on two organizations for diaphragm values. These are the Steel Deck Institute and the International Conference of Building Officials (ICBO). Both organizations provide load tables with strength and stiffness/flexibility criteria. ICBO Evaluation Reports which contain diaphragm strength and flexibility values are used predominantly by engineers 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 designer when comparing the two different sets of load tables. This is due to the fact that different researchers compiled the data and they used different empirical equations to establish diaphragm tables. In addition the ICBO published values contain a factor of safety of 3.0 whereas the Steel Deck Institute values contain a variable factor of safety depending upon the type of diaphragm fastening system.

It should be noted that in all cases the values established have already taken into account the one-third stress increase for wind or seismic loads. Thus the engineer is not permitted an increase in these published values when dealing with wind and seismic lateral loads.

The SDI Diaphragm Design Manual 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, timber decking, laminated timber decking, and 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", Englewood, Colorado.
- 2) Plywood Diaphragms: American Plywood Association, "Plywood Diaphragm Construction", V310, Tacoma, Washington.
- 3) Board Diaphragms: Western Wood Products Association, "Western Woods Use Book", Portland, Oregon.

The remainder of this chapter is devoted solely to diaphragm design using steel decks because of their predominant use.

Design Procedure

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

1. The deck thickness.
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, 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 general rule the authors have found that the maximum number of structural connections and three sidelap fasteners 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 designer provides for the diaphragm strength requirements by specifying the type and number of connectors from load tables. For optimum design, connection patterns or the deck thickness can be altered to achieve the strength requirements at any given location within the plane of the roof. This is analogous to the placement of extra shear reinforcement in a concrete beam as required by the magnitude of shear. When these procedures are used it is necessary for

the engineer to provide a diagram indicating the locations of the deck thicknesses and fastener spacing. An example (shown in Figure 4.2.1) uses a change in the weld and screw

pattern to meet the strength requirements of the diaphragm system. In addition to strength considerations, the deflec-

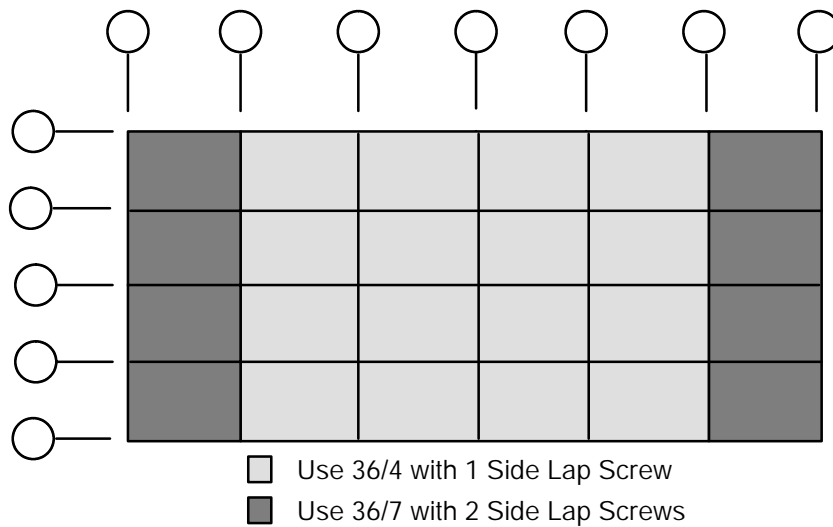


Fig. 4.2.1 Roof Diaphragm Key Plan

tion 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 the various ICBO 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 Reference 14. Generally, serviceability limits are not contained in the building codes since they are not life safety issues. The designer 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 ICBO 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 formula below:

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

where:

- 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 formula.

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 designer in giving thought to the diaphragm deflection.

Diaphragm Connections

Once the diaphragm 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

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 diaphragm chord force along with any other imposed loads. If the pe-

rimeter member is a joist or joist girder, the forces resulting from diaphragm action must be provided to the manufacturer, unless it can be determined that the perimeter member will not be overstressed by the diaphragm chord force. Chord forces from all load cases in combination with the diaphragm chord force must be specified. It is not enough to simply provide the manufacturer with the diaphragm chord force because of the need to check specific code load combinations which include the diaphragm force.

Suitable connections are also required for the diaphragm chords. 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.

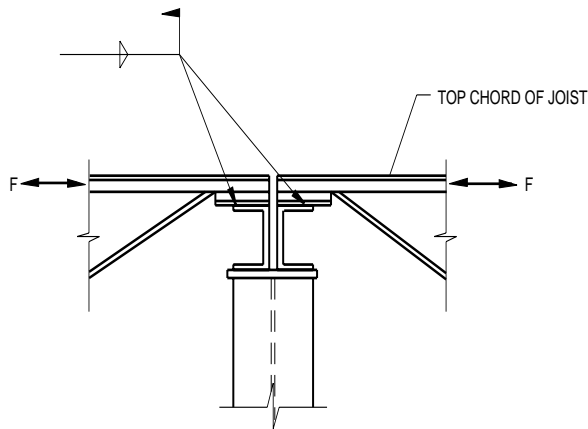


Fig. 4.2.2 Diaphragm Chord

Represented in the figure 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 seat, across the top of the joist girder seat, to the adjacent joist. This is a legitimate force path, but each component must be designed to resist the force F . The capacity of this connection is limited. The strength would probably be controlled by the strength of the joist top chord. The chord is not only subjected to force F , but also the bending moment shown in Figure 4.2.3, where $M = Fe$.

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. The joist manufacturer could design the joist to accommodate the axial force and bending moment, if the proper information is provided to the manufacturer; however, the result would probably be a substantial increase in the size and cost of the

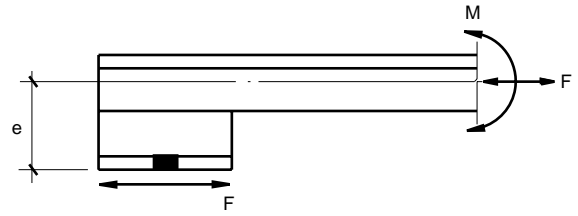


Fig. 4.2.3 Joist Chord Bending

perimeter joists. A better force path would be created if a top plate or tie angles were added to connect the adjacent joists. For roofs, either the detail shown in Figure 4.2.4 or 4.2.5 could be used to create this force path.

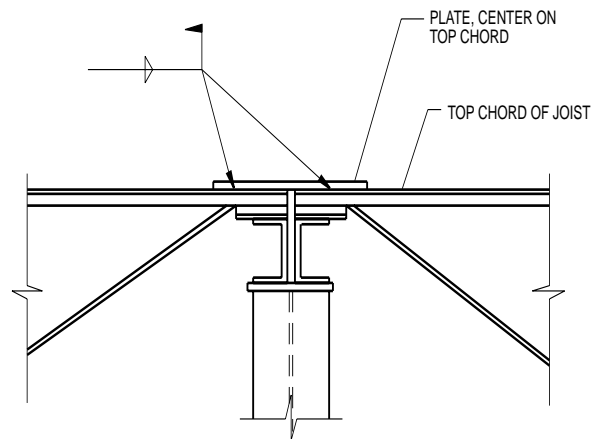


Fig. 4.2.4 Joist Tie Plate

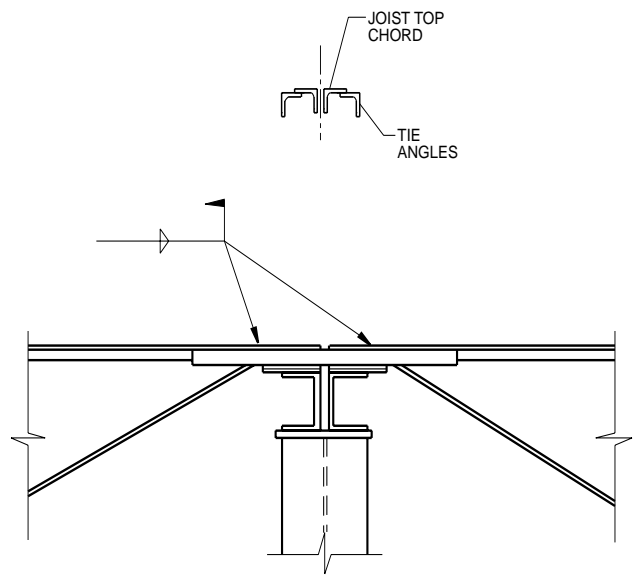


Fig. 4.2.5 Joist Tie Angles

The tie connection and weld sizes are based on the calculated chord force. As mentioned, the manufacturer must be provided with the top chord force unless it is determined that the perimeter joist will not be overstressed by the diaphragm chord force. The following example illustrates the design of the continuity tie and a procedure to check the joist chord for the diaphragm chord force.

Example 4.2.1 Diaphragm Chords

Determine whether the 30K7 perimeter joist shown in Fig. 4.2.6 is overstressed for the wind loads shown. Also design the continuity ties for the perimeter joists. Assume the perimeter joists have an uplift load of 150 plf (0.6D+W per the IBC) acting simultaneously with the diaphragm forces.

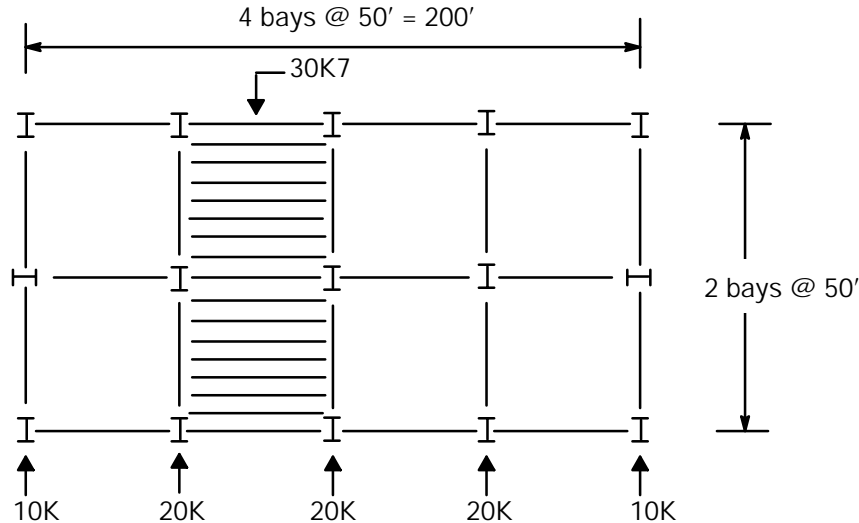


Fig. 4.2.6 Example 4.2.1

Solution:

1. Determine the maximum chord force:

Determine the chord force from the lateral loads:

$$M = (30)(100) - (20)(50) = 2000 \text{ ft.-kips}$$

$$F_{\text{chord}} = M/100 = 20 \text{ kips}$$

Determine the chord force from uplift:

$$M_u = wL^2/8 = 0.150(50)^2/8 = 46.9 \text{ ft.-kips}$$

$$F_u = 12M_u/(30-2 \times 0.5) \approx 19.5 \text{ kips (tension)}$$

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

The maximum chord force = $19.5 + 20 = 39.5$ kips (tension).

2. Determine the allowable tensile chord forces: Based on the joist load tables a 30K7 can support 203 lbs/ft., thus the allowable moment $M = 63.4$ ft.-kips. Conservatively the allowable tensile chord force = M/d
 $M/d = (12)(63.4)/(30-2 \times 0.5) = \pm 26.2$ kips.

Since the chord force is greater than the allowable force, the chord forces should be specified to the manufacturer, or a larger edge joist can be specified. By

proportioning of the load tables a 30K11 is **o.k.**, i.e. $(333/203)(26.2) = 43.0$ kips > 39.5 kips.

3. Tie Plate Design: (use A36 steel)

$P = 20$ kips (Force to be transferred from joist to joist):

$$A_{\text{req'd}} = P/F_t = 20/(22) = 0.91 \text{ in.}^2$$

where,

F_t = the allowable tension stress increased for wind loading.

Use a tie plate $1/4" \times 4"$. $A = 1.0 \text{ in.}^2$

4. Determine weld requirements: (E70 electrodes)

Length req'd of 3/16 fillet weld/joist:

$$(20)/(0.707 \times 21 \times 0.1875) = 7.18 \text{ in.}$$

Use 8" of 3/16" fillet weld to connect the tie plate to each joist.

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

The detail shown in Fig. 4.2.7 represents the typical situation at a building end wall where the joist sits on a joist girder.

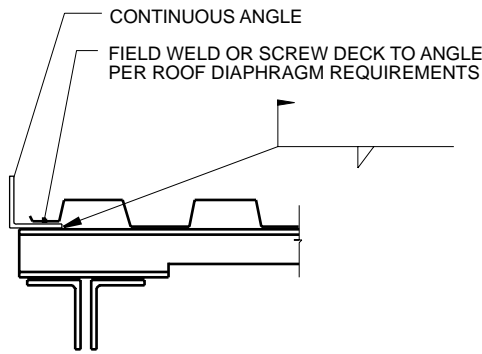


Fig. 4.2.7 Deck Support Angle

The continuous angle shown can be designed as the diaphragm chord, or the diaphragm horizontal shear forces can be transferred down to the top chord of the girder, whereby the girder top chord can furnish the required chord strength. The need for the angle as a deck support member should be determined first. In many buildings the angle is provided to support the deck and attached roofing materials from tearing due to construction and foot traffic on the roof. If the angle is to be provided for this purpose it can then also be sized for the diaphragm chord requirements. A word of caution is appropriate here. If the edge angle is to be continuous as shown then an additional detail illustrating how the angle pieces are to be joined should be provided. If this detail is not shown it is likely the angles will simply be butted together and not connected.

The angle may also be required because of the diaphragm shear requirements for loads in the perpendicular direction, i.e. if the shear forces are such that sidelap connectors are required between the deck sheets, then sidelap connectors are also required at the edge of the diaphragm. The need for sidelap fasteners does not stop at the diaphragm edge. If the edge connectors are omitted then the fasteners connecting the deck to the joist will be overstressed. The joist seat shown in Fig. 4.2.7 can be used as a shear collector, i.e. a shear transfer device. The rollover capacity of typical joist seats is about 1920 pounds. This capacity is discussed in Chapter 7. If additional capacity is required then some other kind of shear transfer device must be used. Once the load is in the joist girder top chord a proper force path to connect girder to girder must be provided. A detail similar to that shown in Fig. 4.2.5 can be used if required.

Shear Collectors

Details must be provided to transfer the diaphragm shears into the vertical bracing system. A variety of details have been used. In the preceding paragraphs the transfer of

deck shears to joist girder top chords was briefly discussed. For relatively small shears it was pointed out that the joist seat could be used for this transfer. If sidelap fasteners are required then a “drag strut” or shear transfer device would likely be required between the diaphragm deck and the joist girder top chord or edge beam. The details shown in Figs. 4.2.8 and 4.2.9 have been used successfully. A joist substitute (VS joist) can be used in lieu of the 2-1/2 inch square tube shown in Figure 4.2.8. Care must be taken to position the 2-1/2 inch square tube or the channel so that the deck flat rests against the top flange of these members. This is necessary to provide for the arc spot welds or other fasteners to the shear collector. Fastening can be clustered at selected shear collectors but care must be taken not to overstress the diaphragm by funneling all of the horizontal shear into or out of the system at one location. The AISI specification provides the designer with equations for calculating allowable arc spot weld stresses for shear.

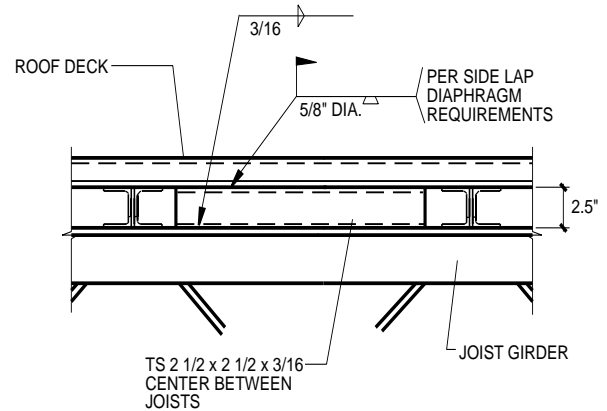


Fig. 4.2.8 Shear Collector with K Joist

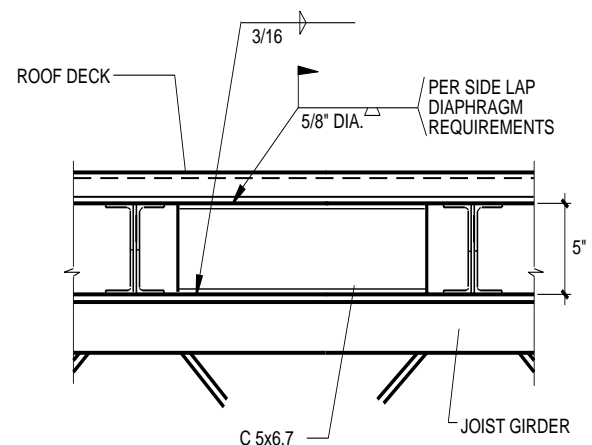


Fig. 4.2.9 Shear Collector with LH Joist

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. 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 horizontal load. In Figs. 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 Fig. 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 can be placed on the arc spot welds.

The detail shown in Fig. 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 multi-purpose function of deck support, shear transfer device, and diaphragm chord member.

A similar detail is shown in Fig. 4.2.14 where joists are parallel to the wall. In this detail the continuous angle also serves as a structural member to support deck gravity loading. This detail could be used when a joist is not placed next to the wall. It has the advantage of accommodating joist camber, in that the flexibility of the deck permits the edge angle to be placed horizontally. The deck will flex enough to accommodate most camber conditions in K series joist. The accommodation of camber is discussed further in section 5.10. The edge angle does save the cost of the edge joist; however, the cost of installing the edge angle can be significant when scaffolding must be used.

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

For buildings with long walls, steel diaphragm force attachments should be made at the wall’s mid point so that the roof diaphragm can expand and contract independently from the wall. Horizontal as well as vertical slip joints should be provided.

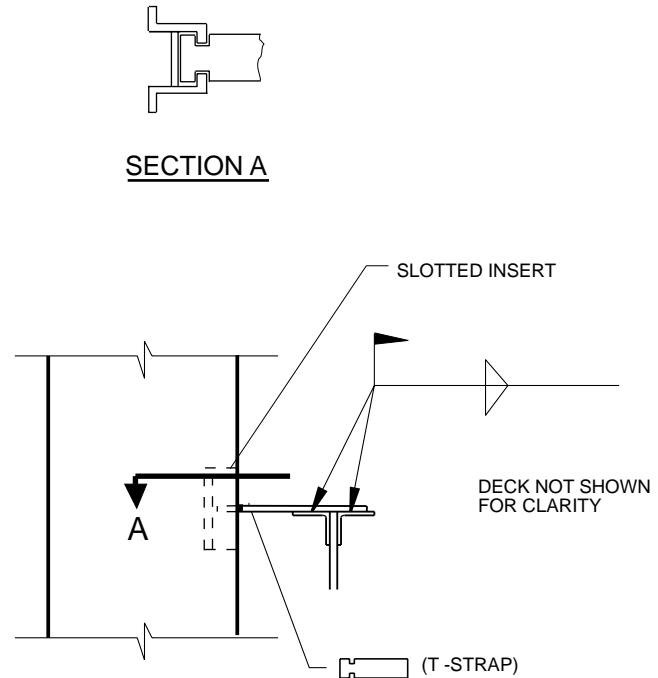


Fig. 4.2.10 Shear Transfer to Precast

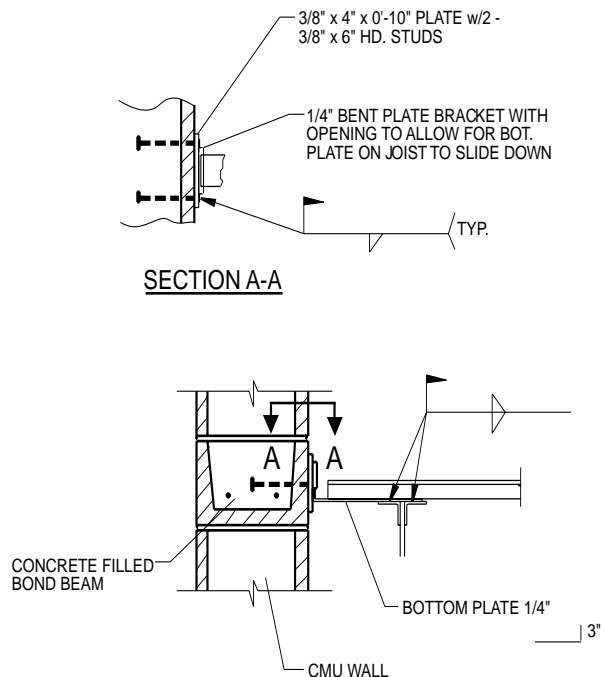


Fig. 4.2.11 Shear Transfer to Masonry

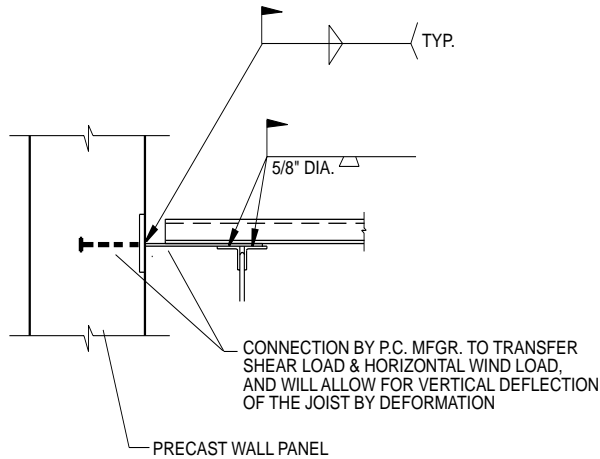


Fig. 4.2.12 Shear Transfer to Precast

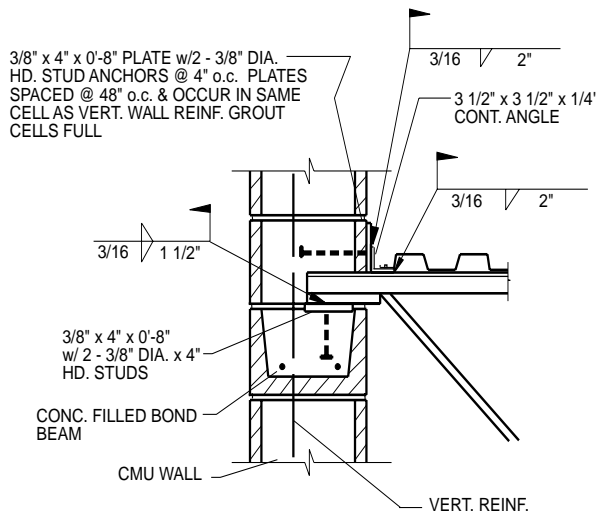


Fig. 4.2.13 Gravity and Shear Load Transfer to Masonry

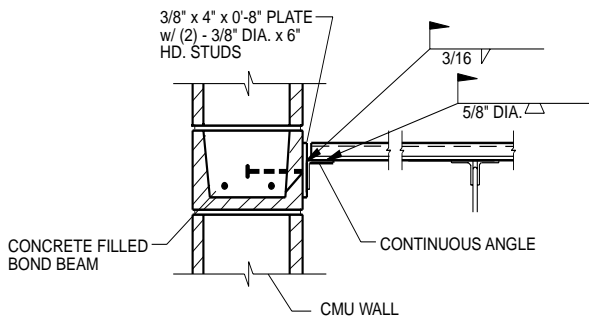
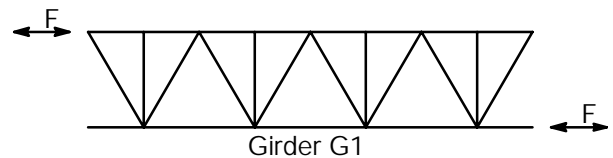


Fig. 4.2.14 Gravity and Shear Load Transfer to Masonry

The designer 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 steel frame. This condition is discussed further in Chapter 8.

Attachment to Vertical Bracing

When vertical steel bracing is used, the perimeter members of the diaphragm must transfer the horizontal 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 joist girder bottom chord. In these cases, the bottom chord of the girder can serve as the compression strut or the column can be designed to transfer the horizontal force to the bracing in column bending. For the column to work in bending, the top chord force must be transferred thru the joist girder seat and then into the column. The seat capacity is limited to 8 to 16 kips (ASD, see Chapter 7), unless reinforcement is provided. If the bottom chord is used as the strut and column bending is not desired, then the top chord force must be transferred to the bottom chord through the web members. The designer must provide the force information to the manufacturer so that the manufacturer can check the web members and can also determine the lateral bracing requirements for the bottom chord of the joist girder. The force diagram shown in Fig. 4.2.15 can be used to convey this type of information to the manufacturer.



Note:

Design the web system to transfer the force F from the top chord to the bottom chord. See schedule for load combinations.

Fig. 4.2.15 Joist Girder Note

In addition to the information shown in Fig. 4.2.15, the manufacturer must be informed as to how to combine the force system (F) with other load combinations. Sample load schedules are discussed in Chapter 6.

In lieu of specifying the chord forces, the designer may wish to make a calculation to see if this load case will control the design of the joist girder. An example of such a calculation is shown below:

Example 4.2.2 Vertical Bracing with Joist Girder

A 60 ft. long 60G12N10K joist girder is required for live load. If a 20 kip chord wind force must be transferred thru the web system, determine what size joist girder should be specified.

Solution:

Approximate gravity load chord force:

$M \cong wL^2/8$; $w = 2$ kips/ft. (10 kips \div 5 ft. joist spacing)

$M = 900$ ft.-kips

Chord Force $\cong (900)(12)/60 = 180$ kips

Thus, the top and bottom chord force is 200 kips, (180 + 20). Try a 60G12N12K girder. The allowable chord force equals:

$M \cong wL^2/8$; $w = 2.4$ kips/ft. (12 kips \div 5 ft. joist spacing)

$M = 1080$ ft.-kips

Chord Force $\cong 216$ kips

$\therefore 200$ kips < 216 kips o.k.

If it is assumed that only the end diagonals transfer the 20 kips shear to the bottom chord then each diagonal must resist 10 kips horizontal load plus the end shear in the girder. The end shear equals $wL/2 = 2(60)/2 = 60$ kips. Because of the geometry of the girder, the end diagonals are angled approximately 45 degrees, thus the maximum diagonal force is $(10 \times 1.414 + 60 \times 1.414) \cong 99$ kips < $(1.414)(2.4)(60)/2 \cong 102$ kips. Thus, the load combination of live load plus wind does not govern the 60G12N12K joist girder.

The same procedures as described above 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 al-

lows the expansion joint to perform its intended function and yet is capable of shear transfer will work. The details shown in Figs. 4.2.16 and 4.2.17 have been used. In Fig. 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. However, if insufficient capacity exists, a shear collector can be used on the joist girder lines to carry the force.

In Fig. 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 both framing directions or in only one direction. In Fig. 4.3.1 a roof plan is shown in which horizontal bracing is used to resist lateral forces in only one framing direction.

In Fig. 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.

The bracing members shown in Figs. 4.3.1 and 4.3.2 can take several forms. That is, 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 and 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 1/4 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, i.e. either adding additional bracing or choosing an alternate lateral load system. This rule of thumb is based on the fact that the forces in straps of 1/4"x6" become so large that connections to structural elements are not practical.

LATERAL LOAD SYSTEMS

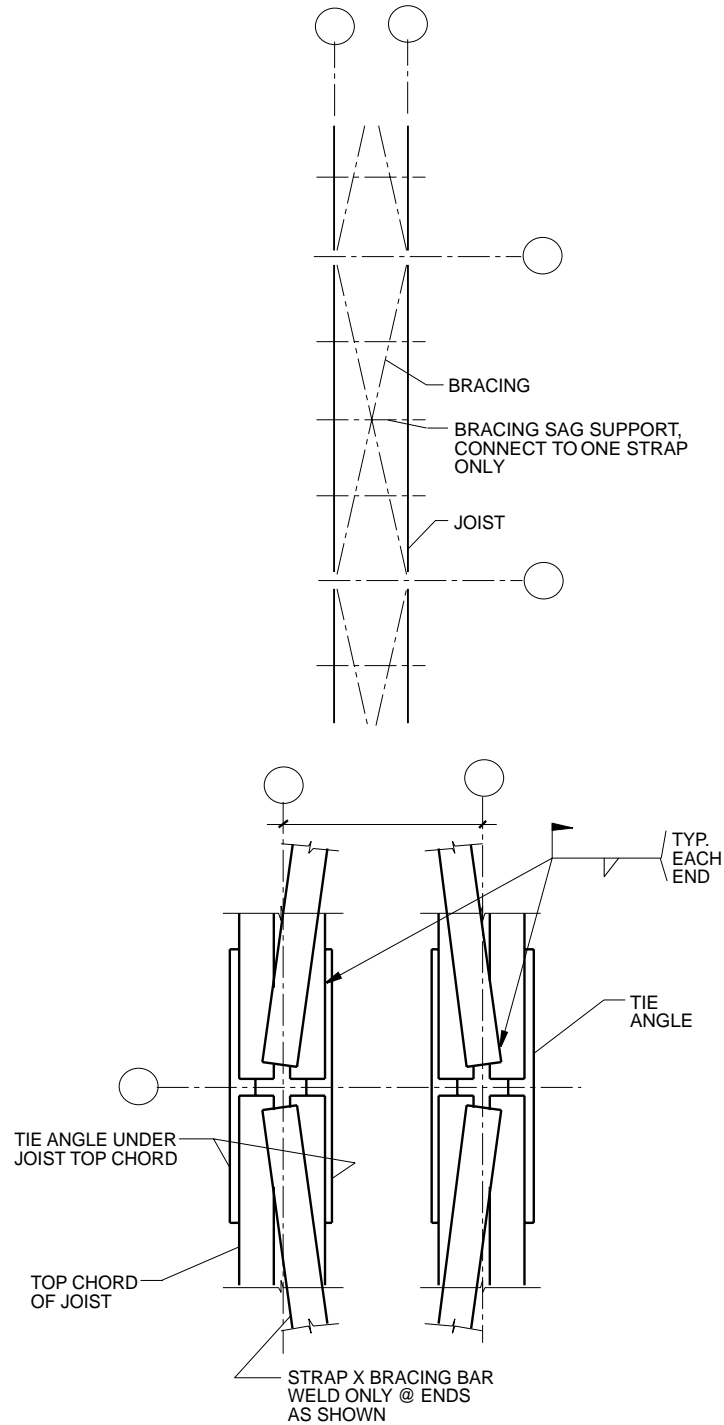


Fig. 4.2.16 Expansion Joint Shear Transfer

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 of the joists at each crossing to hold them in place.

LATERAL LOAD SYSTEMS

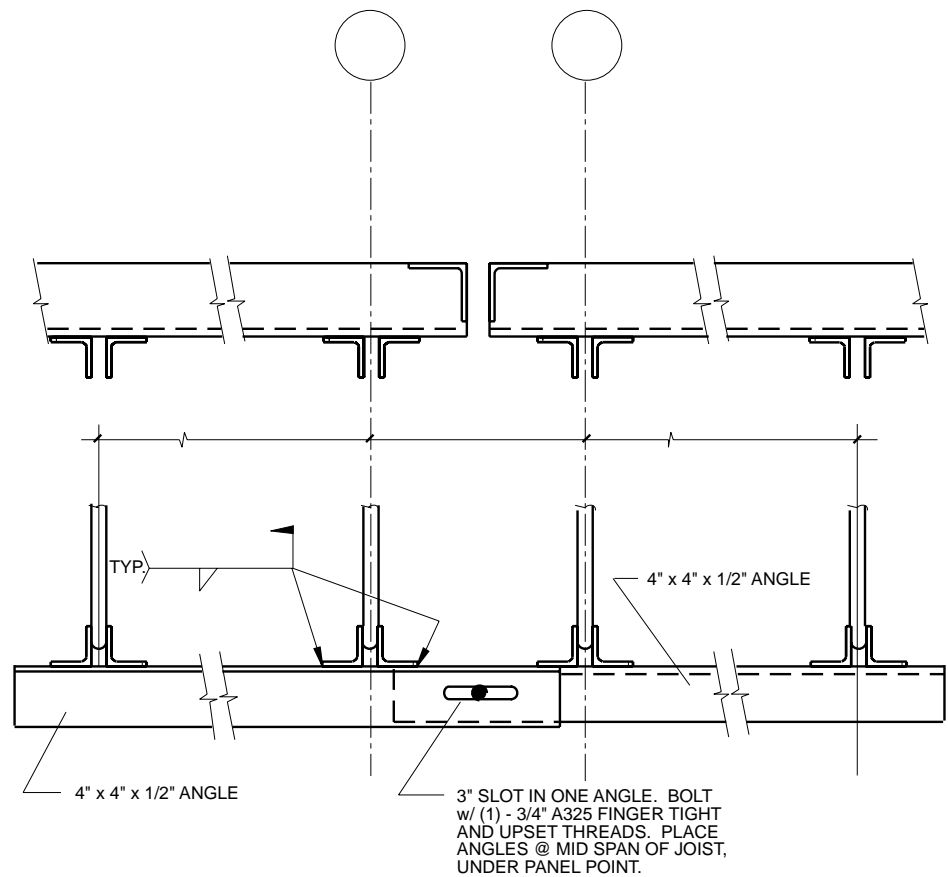


Fig. 4.2.17 Expansion Joint Shear Transfer

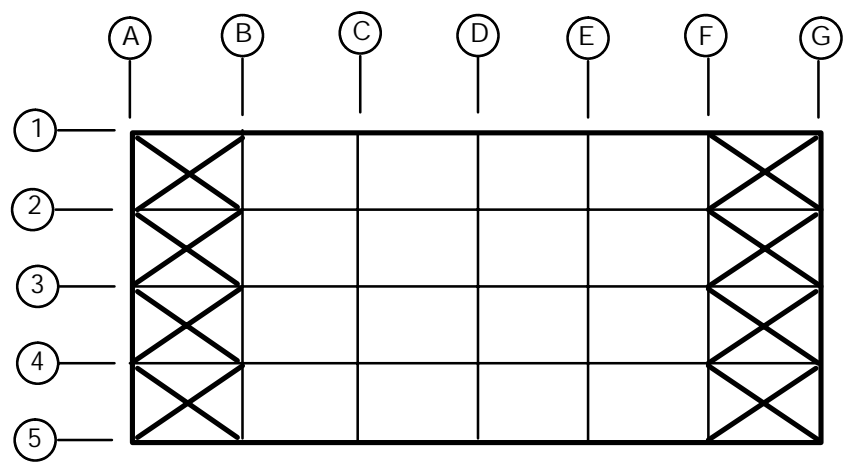


Fig. 4.3.1 Roof Bracing, One Direction

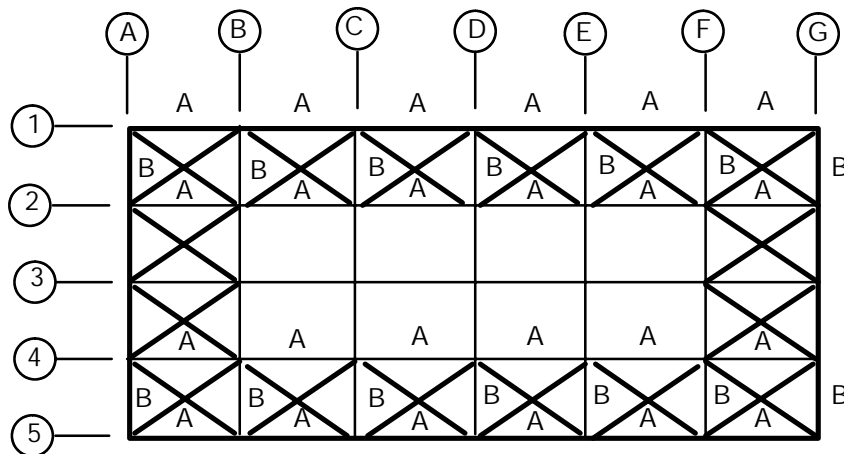


Fig. 4.3.2 Roof Bracing, Two Directions

Analysis Procedure

Most horizontal bracing systems are analyzed assuming that the horizontal bracing in combination with the joists and joist girders forms 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 Figs. 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 designer 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 on the structure. This can be seen by referring to the truss layout shown in Fig. 4.3.2. If the joists run left to right, then the joists marked A in Fig. 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 the manufacturer for sizing or the designer can check to see if the lateral load chord forces will control the joist top chord size. The procedure shown in Example 4.2.1 can be used for this determination. If it is apparent that the joists must be increased in size to accommodate the lateral loads then it may be more cost effective to provide the horizontal bracing shown in Fig. 4.3.3. By positioning the bracing as shown in Fig. 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 (See Fig. 4.2.4) is reduced. 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 into the bracing system.

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 the manufacturer. A detail such as the one shown in Fig. 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.

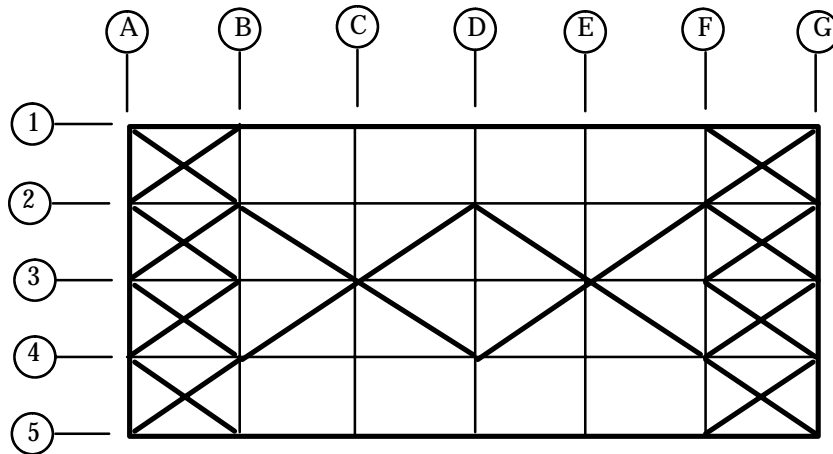


Fig. 4.3.3 Horizontal Bracing

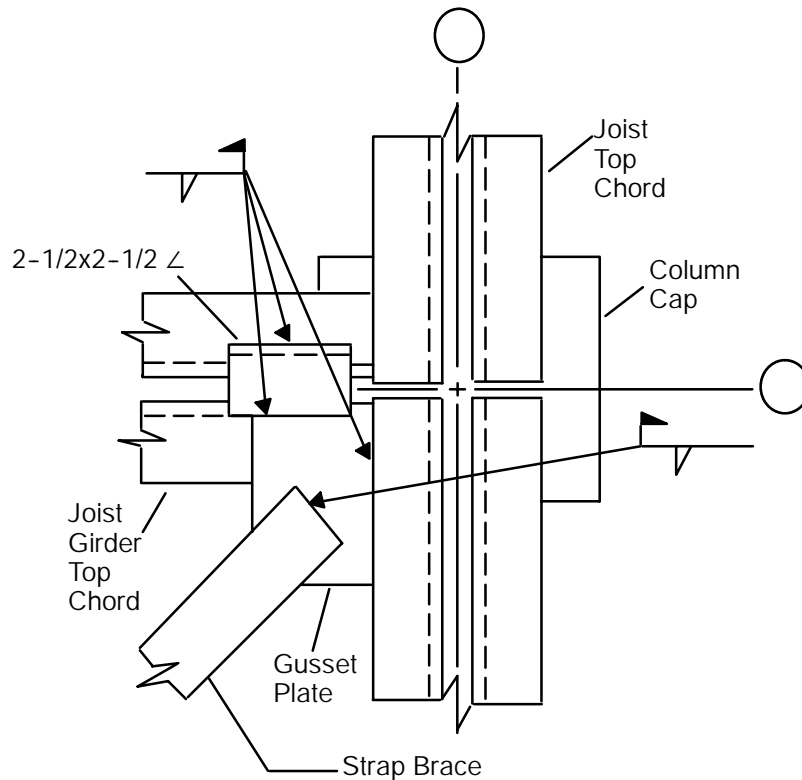


Fig. 4.3.4 Strap Bracing Detail

4.4 BRACED FRAMES

Diaphragms or horizontal bracing systems can be used in conjunction with either braced frames or rigid frames or a combination of both. The purpose of this section is to discuss transferring of loads from diaphragms or horizontal bracing into a vertical bracing system. The design of the vertical bracing system for joist and joist girder

buildings is basically the same as for buildings framed with beams and girders.

Multistory Frames

For multi-story steel frames, vertical bracing systems provide very economical framing. However, bracing is generally not allowed around the perimeter of the building

because of its interference with windows. On occasion, certain bays can be braced where stair wells and elevator shafts are positioned at the perimeter bays. An option usually exists as to whether the vertical bracing system should consist of steel members or whether use should be made of concrete or concrete block walls. No clear cut answer exists as to which system is best. Using steel bracing has the distinct advantage that the frame is totally dependent for its stability on only the steel frame. Scheduling problems are often minimized since the frame can be constructed independently. Using concrete or masonry shear walls eliminates some steel tonnage from the structure; however, the cost savings may be offset by scheduling delays, increased cost of reinforcement in the walls, and the details of connections between frame and shear walls.

As discussed earlier, the joists and joist girder chords can serve as struts in bracing systems; however, when the frames are over two or three stories in height, the strut forces become so large that it is usually simpler and more efficient to use wide flange beams in combination with the vertical bracing. Also, since the bracing is most often located around stair well and elevator shafts, beams may already be present because they are often used to frame these openings.

As with any steel frame most basic bracing configurations can be used with joist and joist girder framing. Chevron, K, single diagonal and X bracing are all practical and common. Where beams are substituted for joists or joist girders eccentric bracing can be used as well.

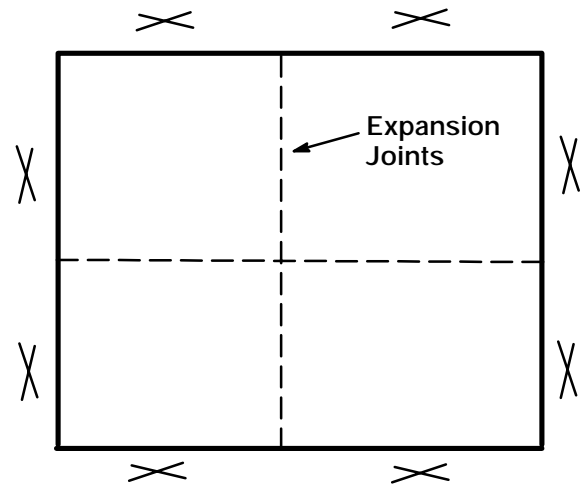
Single Story Frames

Unlike multistory frames, single story frame bracing is almost always located at the perimeter of the structure. Generally only a few windows exist and their locations can be avoided. Only overhead doors and exits must typically be avoided. All of the bracing types mentioned with respect to multistory structures can be used, but X-bracing either with rods or angles is most common.

Much like diaphragms, the economical use of vertical bracing is dependent a great deal on the building geometry. When the length-to-width ratio between braces exceeds about 4 to 1, bracing forces become quite large. Strut forces transferring the forces to the braced bays also become large. With this ratio, significant column uplift forces are also developed, affecting foundation costs. To avoid uplift forces, it is recommended that bracing be placed in adjacent bays rather than separated so that uplift forces can be minimized. When the length to width ratios exceed 4 to 1, the designer should discuss the cost advantages of interior braced bays with the client. The client's first reaction is almost always negative. Sketches should be prepared of K

bracing or eccentric bracing so that the client can see that forklift trucks or pedestrian traffic can be permitted thru the braced bays. If interior bracing is simply not permitted then an alternate lateral force resisting system such as rigid frames must be considered.

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



Note: Structure as shown is torsionally unstable.

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 RIGID FRAMES

Designing joist and joist girder structures as rigid frames is no more difficult than designing rigid frames with wide flange beams and girders. To obtain a cost effective design the engineer must be aware of the inter-relationships 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.

Design Considerations

The first consideration relative to the design of the structure is to determine if rigid frame action is required in

both framing directions. When rigid frames are required in only one direction, the joist girders should be selected to resist the lateral loads. If rigid frame action is required in both directions, the framing scheme that creates the smallest end moments in the joists should be examined first.

For single story buildings, if the Basic Connection (see Chapter 7) can be used without modification to resist the lateral loads, then it is the system that will most likely be the most economical one. Using moment connections on each frame line at the perimeter columns typically provides the most economical system when the Basic Connection cannot be used. If perimeter columns do not exist in the structure such as in the case of load bearing perimeter wall buildings, then the rigid frame connections must be designed by modifying the Basic Connection or by providing brackets on the columns and using top plate moment connections.

For multistory projects, rigid 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.

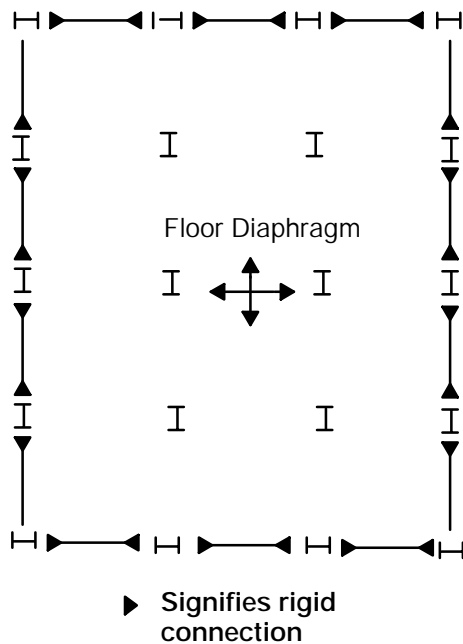


Fig. 4.5.1 Multistory Framing

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

Rigid Moment Connections

As mentioned earlier, the Basic Connection is the most economical connection for rigid frames, provided it has the capacity to resist the imposed lateral loads. This capacity is generally limited by the 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 Basic Connection. Other types of moment connections are also discussed in detail.

As an aid to the designer, typical moment details are summarized and discussed below. After determining the moments that exist at the connections in the frame, the designer 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 due to the fact that the top chord force generally limits the capacity of the connection. To determine the chord forces the designer 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 of the details presented below, the column web must be checked by the building designer to determine if web stiffening is required. The design of the welds connecting the joist girders to the columns is the responsibility of the building designer. See Chapter 7 for further details.

Detail A - (Fig. 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 as indicated on the drawings. 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 engineer may want to specify the welding of the bottom chords after dead loads are applied to the joist girders.

LATERAL LOAD SYSTEMS

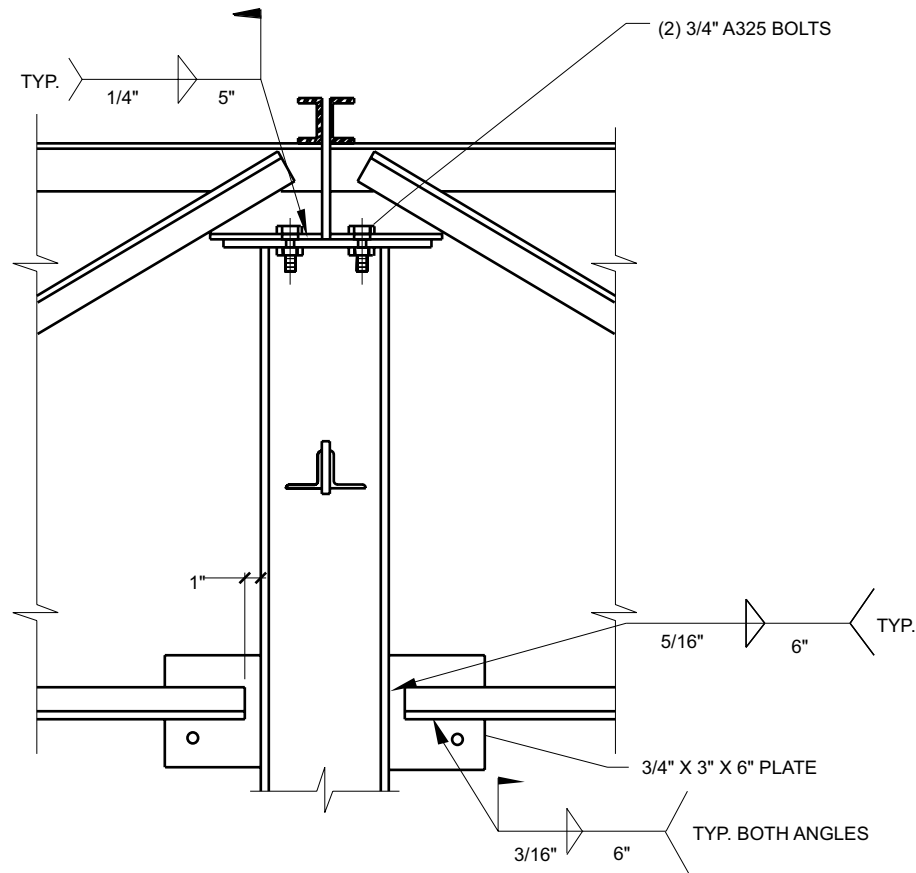


Fig. 4.5.2 Detail A - The Welded Basic Connection

Detail B - (Fig. 4.5.3)

Fig. 4.5.3 illustrates a modification which can be made to the Basic Connection. The modification connects the top chords of adjacent joist girders together 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 allowable load 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.

LATERAL LOAD SYSTEMS

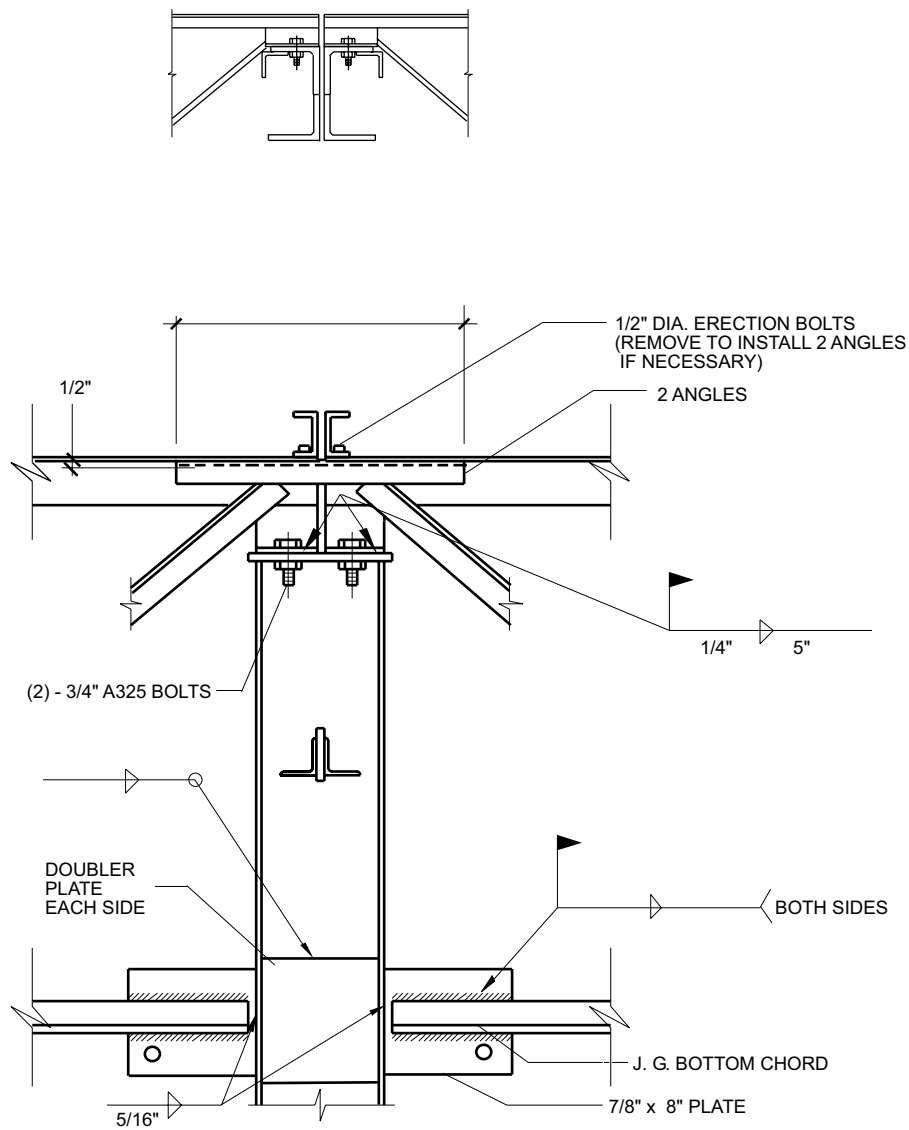


Fig. 4.5.3 Detail B - The Welded Basic Connection with Ties

Detail C - (Fig. 4.5.4)

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 building designer sparingly. Shown in Fig. 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 using this connection type.

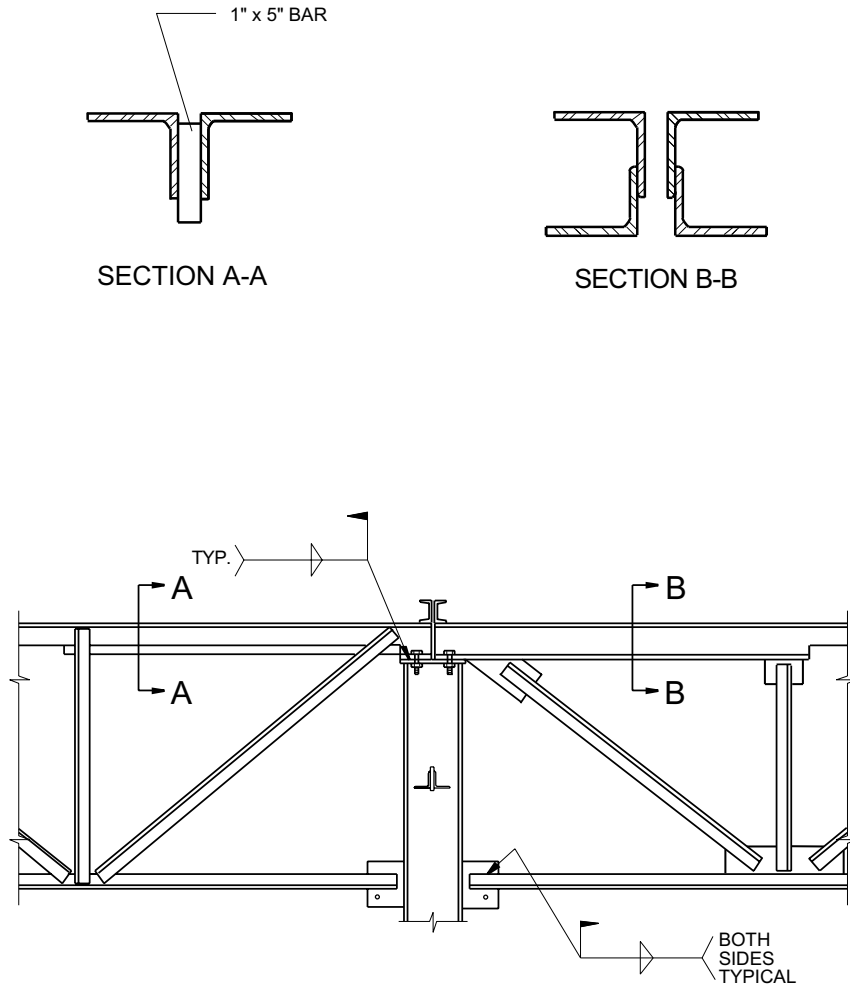


Fig. 4.5.4 Detail C - Reinforced Chords

Detail D - (Fig. 4.5.5)

The special girder seat condition shown in Fig. 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 also 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 centroidal distance between the top and bottom chords.

Detail E - (Figs. 4.5.6 and 4.5.7)

Detail E is capable of developing larger joist girder chord forces than Details A thru D. It also has the advantage

of giving the designer more control over the design, thus less coordination with the manufacturer is required. A disadvantage is that a seat must be attached to the column in order for the moment plate to be welded to the column. The detail shown in Figure 4.5.7 is the most common and most effective connection for multistory frames because the seats are necessary on continuous columns to pick up the joist girders. The force couple lever arm for this detail is from the centroidal distance between the top and bottom chords. The strength of Detail E is limited by the top chord axial load capacity. A chord load limit of 200 kips for ASD and 300 kips for LRFD are recommended for this detail.

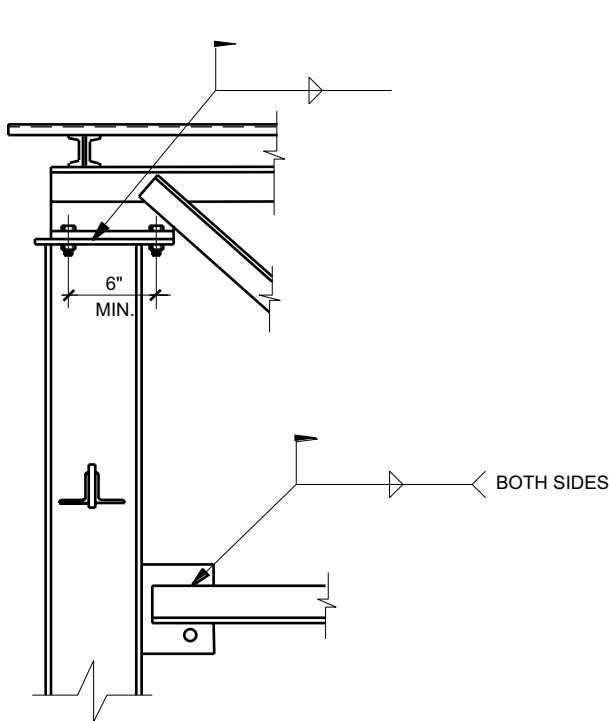


Fig. 4.5.5 Detail D - Rigid Seat Connection

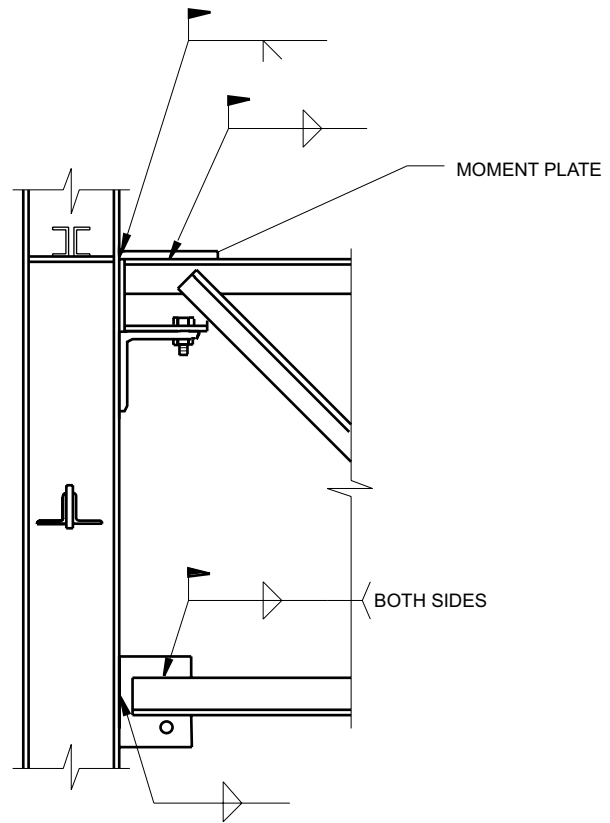


Fig. 4.5.7 Detail E - Moment Plate Connection

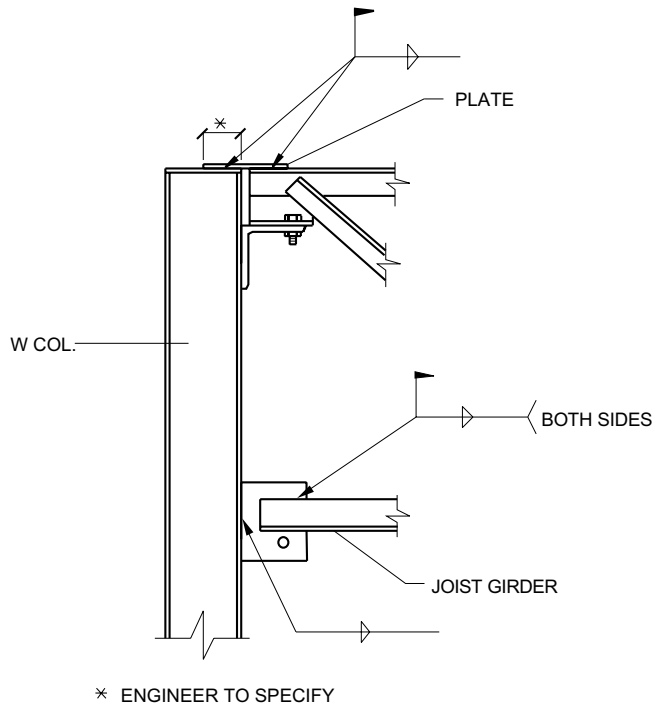


Fig. 4.5.6 Detail E - Moment Plate Connection

Detail F - (Fig. 4.5.8)

A modification to Detail E is shown in Figure 4.5.8. This knife plate connection has been used successfully in multistory moment frames. An advantage of the knife plate connection is that it eliminates some of the field welding re-

quired with Detail E. A disadvantage is that the manufacturer must give special attention in the joist girder seat design to accommodate the knife plate. The knife plate should be shop welded only to the column flange. Welding the knife plate to the column bracket will interfere with the joist girder seat to bracket connection. The designer is encouraged to contact the local Vulcraft sales engineer regarding the use of the knife plate connection. 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.

Joist Details

In the details presented below the building designer 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 designer. See Chapter 7 for further details.

Detail G - (Fig. 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 4.0 kips (ASD), and 6.0 kips (LRFD). Use of the rod bottom chord extensions reduce the continuity chord forces as explained in Chapter 7.

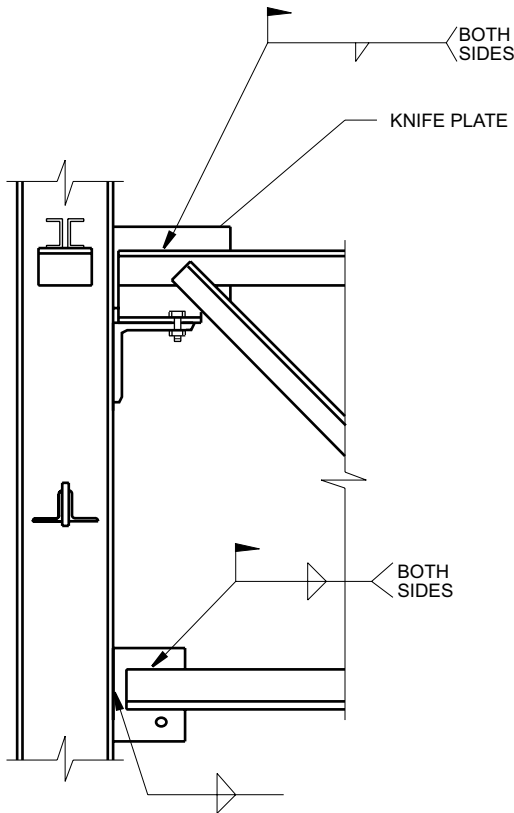


Fig. 4.5.8 Detail F - Knife Plate Connection

Detail H - (Fig. 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.

Detail I - (Fig. 4.5.11)

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 the manufacturer is significant, an alternative to Detail I should be used.

Detail J - (Fig. 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 designer must be carefully considered.

Detail K

The bracketed connection shown in Fig. 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.

Column Design

Columns in joist and joist girder rigid frame buildings are designed in the same manner as columns in any other type of rigid frame structure. The column must satisfy the AISC specification requirements for axial loads and moments.

To determine the effective length factor K for the column, the nomographs which are contained in the AISC Specification Commentary are used. In order to use the nomograph the moment of inertia for the members framing into the column must be known. For joists, the moment of inertia is easily determined from the joist load tables. The Vulcraft catalog 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. L equals (span - 0.33), in feet. This moment of inertia given is a "flexural" moment of inertia in that it does not include a reduction for web deformations. The loads relating to deflections listed in the load tables are calculated based on a moment of inertia equal to I_J divided by 1.15. This approximate moment of inertia equation can also be used for LH and DLH joists. The moment of inertia is listed in the Vulcraft load tables for the KCS joist series.

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 use of the inelastic K method as outlined in the column section of the AISC manual is appropriate for joist and joist girder frames.

It also should be mentioned that the effect of "leaner" columns should be considered in the design of the rigid frames. "Leaner" columns are those which depend upon other columns or bracing to provide for their stability. For example, the interior columns shown in the frame in Fig. 4.5.13 have hinged connections at their tops and base.

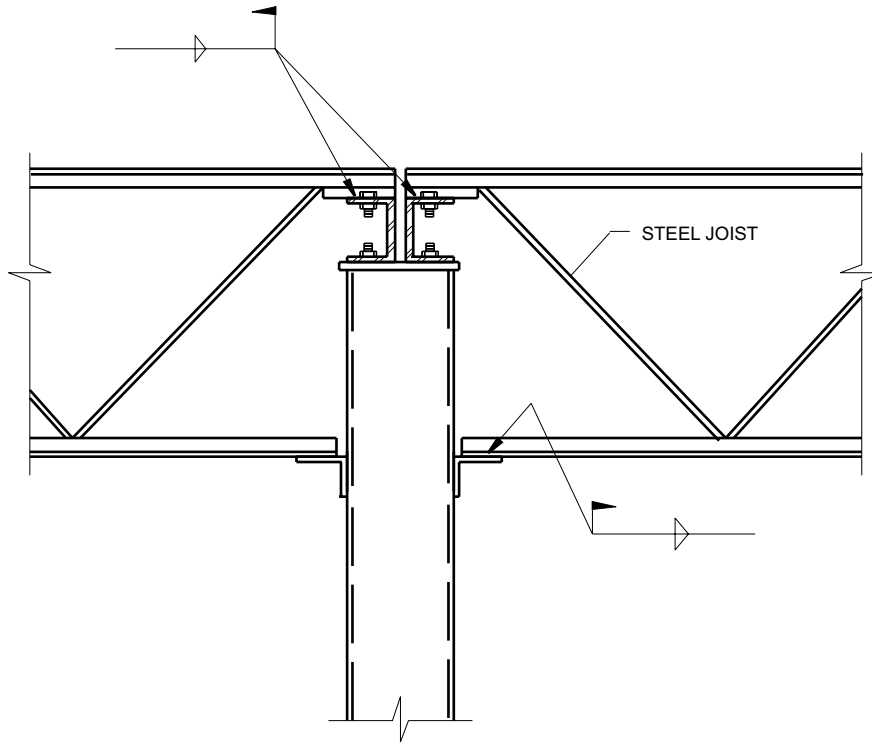


Fig. 4.5.9 Detail G - The Basic Connection

These columns depend on the exterior rigidly connected columns to provide the necessary resistance to prevent lateral sway buckling. Another way of describing the problem is to consider the fact that if interior columns are designed with a K factor of 1.0, it is required that the top of the column be laterally braced. To laterally brace the top of a column requires a brace force and a brace stiffness. The rigid frame must provide the necessary strength and stiffness.

For a proper design using “leaner” columns, story stability must be checked. The check is that the summation of the applied axial loads must be less than the summation of the allowable axial loads i.e.,

$$\Sigma P \leq \Sigma P_{\text{allowable}}$$

For the frame shown in Fig.4.5.13, $P_A + 3P_B + P_C$ must be less than the summation of the allowable buckling loads for columns A and C. The allowable buckling loads for columns A and C are determined in the direction of sway stability; that is, if the girders are framed into the column flanges, then the allowable buckling loads for columns A and C are determined based on their strong axis.

In addition to the story stability check, for ASD the AISC beam-column Equation (H1-1) must be modified because of the “leaner columns”.

Equation H1-1 is modified as shown:

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{\Sigma P}{\Sigma P_e}\right) F_b} \leq 1.0$$

where F_a is based upon the greatest slenderness ratio for the column, i.e. KL/r_y or KL/r_x . The term $1/(1 - \Sigma P/\Sigma P_e)$ is the moment magnifier for the bending moments in the member. The $\Sigma P/\Sigma P_e$ is used in lieu of the normal f_a/F'_e term to account for the effects of the “leaner” columns. ΣP is the sum of the axial loads in the story and ΣP_e is the sum of the Euler buckling loads in the story. For the frame shown in Figure 4.5.13, only columns A and C would contribute to the ΣP_e . The P_e terms are based on the direction of sway. Some engineers prefer to perform a second order frame analysis in lieu of using the moment magnifier term.

A point often overlooked by engineers is that the beam end-moments in a rigid frame are also increased due to column magnification effects. In stiff frames, this effect is often negligible; however, when the frame consists of many “leaner” columns or when the frame is flexible, the second order beam moments should be considered. Since joint equilibrium is always required, the beam end moments can be determined from the magnified column end moments by distributing them to the beams framing into the joint in proportion to the beam stiffnesses.

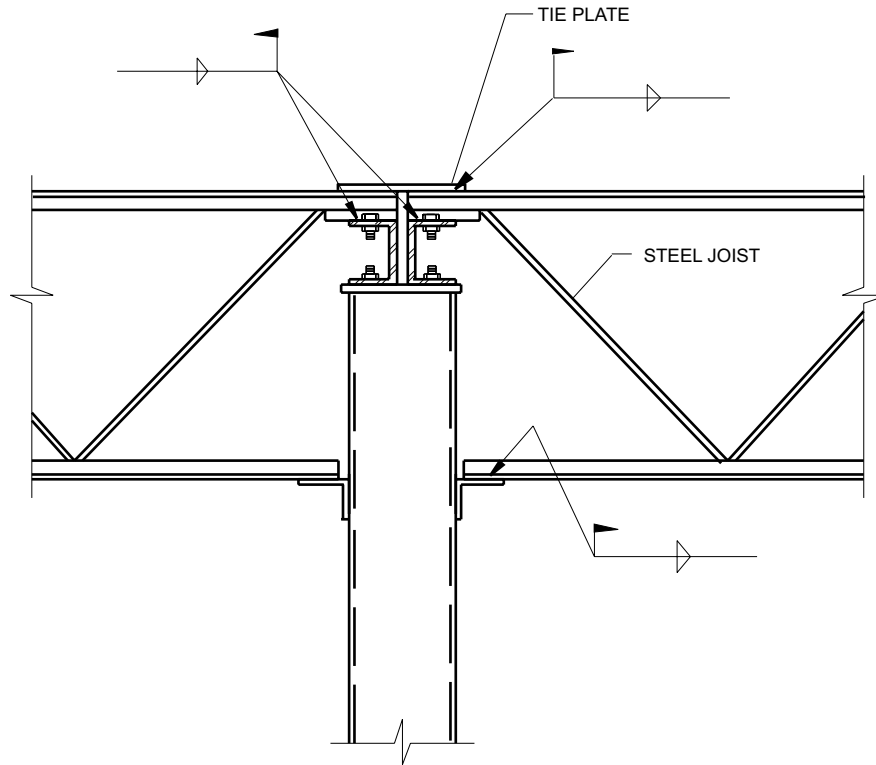


Fig. 4.5.10 Detail H - The Basic Connection with Tie

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. Table 4.5.1 gives a summary of typical drift limitations for single story industrial buildings.

For multistory frames, most engineers 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.

Reference 14 is an expanded treatise on serviceability requirements in low-rise steel buildings.

4.6 FRAMES WITH WIND CONNECTIONS

Although not recognized by the AISC LRFD Specification, for many years the AISC ASD Specification has recognized three basic connection types, each of which will

govern in a specific manner the size of members and the strength of connections in a given frame. The three types are currently described in the 1989 Allowable Stress Design Specification. They are:

“Type 1, commonly designated as ‘rigid-frame’ (continuous frame), assumes that beam-to-column connections have sufficient rigidity to hold virtually unchanged the original angles between intersecting members.

Type 2, commonly designated as ‘simple framing’ (unrestrained, free-ended), assumes that, insofar as gravity loading is concerned, ends of beams and girders are connected for shear only and are free to rotate under gravity load.

Type 3, commonly designated as ‘semi-rigid framing’ (partially restrained), assumes that the connections of beams and girder possess a dependable and known moment capacity intermediate in degree between the rigidity of Type 1 and the flexibility of Type 2.”

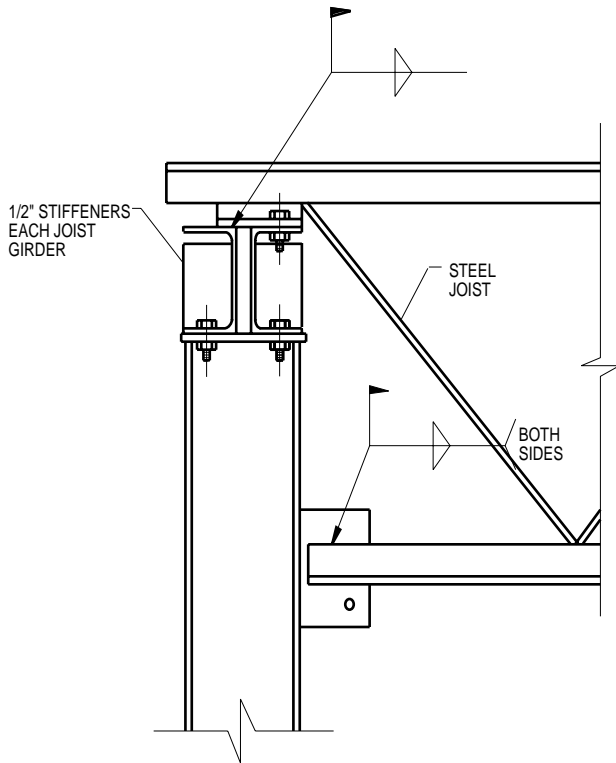


Fig. 4.5.11 Detail I - Stiffeners in Girder Seat

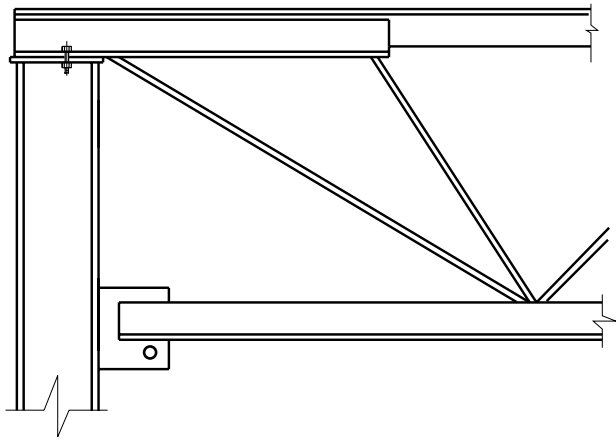


Fig. 4.5.12 Detail J - Joist E Member Extension

The Specification further states the following with respect to Type 2 construction:

“In buildings designed as Type 2 construction (i.e., with beam-to-column connections other than wind connections assumed flexible under gravity loading) the wind moments may be distributed among selected joints of the frame, provided:

1. Connections and connected members have adequate capacity to resist wind moments
2. Girders are adequate to carry full gravity load as ‘simple beams’.
3. Connections have adequate inelastic rotation capacity to avoid overstress of the fasteners or welds under combined gravity and wind loading.”

Type 2 connections which are designed to resist wind moments are often referred to as wind connections. Type 2 construction requires the beams to be designed assuming simple supports and columns to be designed for gravity loads plus wind forces. Wind forces are usually obtained using standard portal analysis or by elastic computer analysis. Beam-to-column connections must be strong enough to resist wind moments but flexible enough to relieve gravity moments via inelastic deformation. At the current time the Uniform Building Code has no provisions for the use of Type 2 wind connections for the resistance to seismic loadings. Type 2 construction is a natural for joist and joist girder framing, since it is the practice of most joist and joist girder manufacturers to design the joists and girders for full gravity load and then to check the strength of these joists or girders for specified wind moments.

The advantage of using wind connections is that smaller column moments and less field welding are generally required at the connections. In addition large continuity forces may be eliminated from the joists and girders.

Several types of wind connections have been used by engineers for beam construction. These include web framing angles, seat angles with top angles, combinations of framing angles with top and bottom angles, tee sections and yieldable flange plates. For joist and joist girder construction the use of a yieldable top flange plate is the ideal wind connection. Top flange plate connections are shown in Figures 4.6.1 and 4.6.2. These connections are ideally suited for joists and joist girders because they can be used in conjunction with the normal stabilizer plate at the bottom chord. They are also ideal because they possess a very predictable linear load deformation curve, and they are quite stiff up to the point of yielding.

Design Considerations for Wind Connections

The same design considerations exist for Type 2 wind frames as for rigid connections. The authors have found that wind connections are ideal for one story structures when there are no perimeter columns. This would be the case in load bearing precast, tilt-up or masonry load-bear-

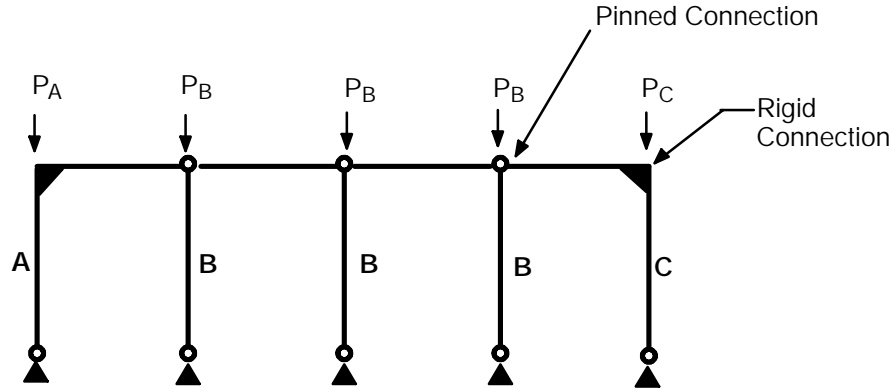


Fig. 4.5.13 Frame with Leaner Columns

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

Table 4.5.1

* Note: These drift indices can be increased with proper detailing. See Reference 14.

ing wall structures. In these cases, if moment connections are required, the interior framing is designed to resist the lateral loads. The wind connections eliminate large continuity moments. With smaller wind end moments the need for stiffeners, web doublers, etc. in the columns is less likely than with full moment connections. A disadvantage for single story frames is joists and girders must be placed on column brackets which is an extra expense.

Connection Design

The design of the wind connection is a relatively simple task. The joist or girder is assumed to create a force couple as shown in Figure 4.6.3.

Force F as shown in Fig. 4.6.3 equals the wind moment divided by the distance d . The required area of top plate is equal to the force F divided by the allowable tensile stress for the plate. The top plate must be connected to the column with enough strength to fully yield the plate. This will require a full penetration weld to the column flange, or a partial penetration weld with a fillet weld reinforcement. The connection of the bottom chord to the column should be of sufficient strength to guarantee that the top plate yields before the bottom connection yields. It is suggested that the bottom chord connection and the welds connecting the top plate to the column and to the joist or girder top chord be designed with a strength equal to 1.25 times the plate

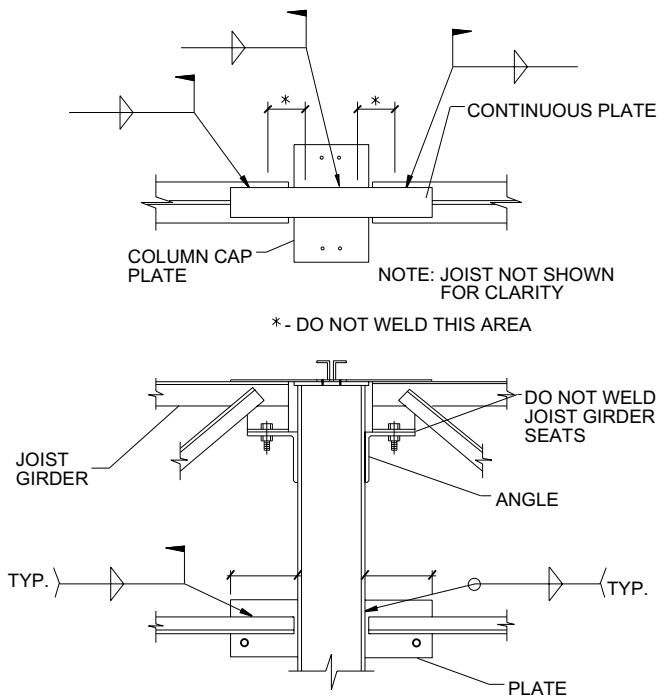


Fig. 4.6.1 Joist Girder with Flange Plate

strength. This extra 25 percent in strength plus the normal factor of safety in weld strength as compared to tensile yield strength will ensure plate yield prior to weld yield. An unwelded length is necessary along the length of the plate to allow unrestricted plastic flow in the plate. This unwelded length is usually taken as 1.5 times the plate width.

Column Design

An adjustment in the effective length factor for the column is required when wind connections are used. The effects of semi-rigid connections and wind connections have been addressed by several researchers.^{5,8,9} Disque suggests in Reference 8, that the effective length factor can be found by assuming that only one girder restrains the column from rotating and the girder be considered pinned at its far end. Thus, in computing the stiffness factor for use in the AISC nomograph, the girder stiffness is modified by using one-half of the moment inertia of the member framing into the column.¹⁵ The logic for this approach is quite direct. If one assumes that plastic hinges have formed in the

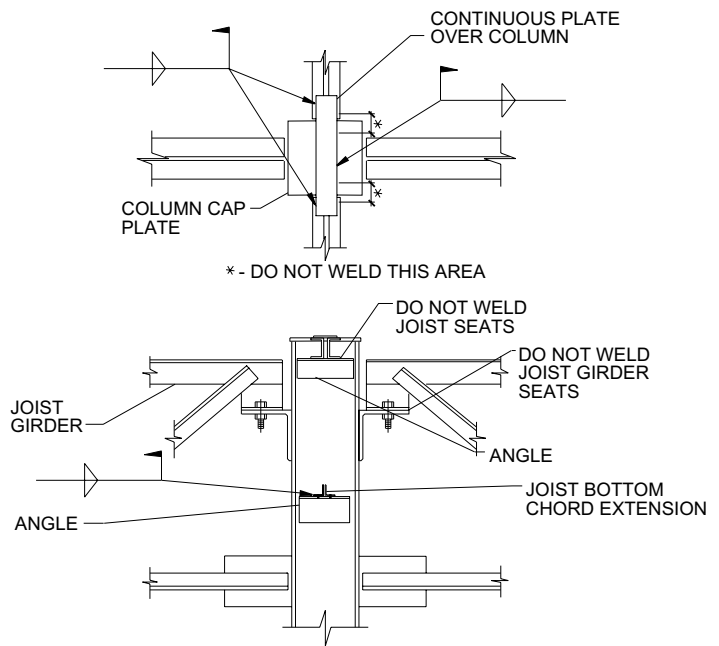


Fig. 4.6.2 Joist with Flange Plate

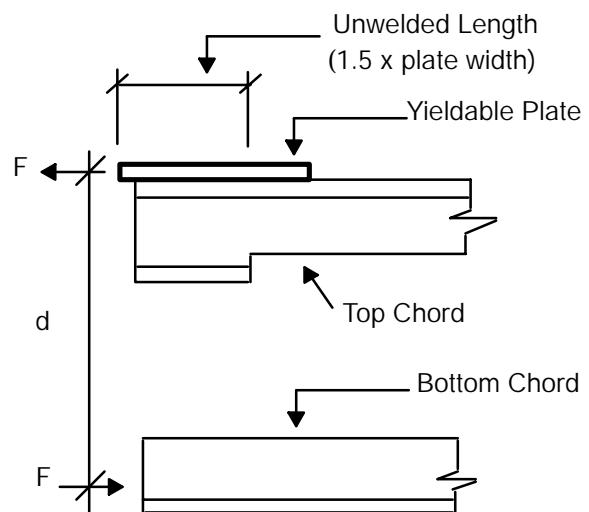


Fig. 4.6.3 Joist Girder Wind Connections

wind connections under gravity loads then the only restraint against column sway would be those beam-column connections that unload during lateral sway. For example, for the columns shown in Figure 4.6.4, for movement to the right, stability for column B is provided by the unloading of the connection at the left end of member E.

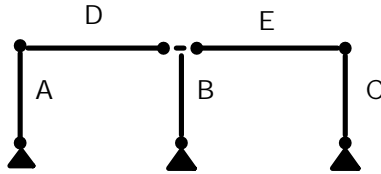


Fig. 4.6.4 Column Stability

The connections at the right end of member D and E provide no restraint since they would simply continue to rotate in their yielded direction. Stability for column A is provided by the connection at the left end of member D. Column C would be a leaner column and depend upon columns A and B for stability. For movements to the left the opposite situation would occur. Thus, story stability must be checked using the equation $\Sigma P \leq \Sigma P_{\text{allowable}}$.

Drift Considerations

Drifts for frames with wind connections are larger for identical frames with “fully” rigid moment connections. The yieldable top plate connection is quite stiff and little loss of stiffness occurs when the wind connection is unloading elastically; however, the connections which continue to rotate in the same direction as caused by gravity load offer little lateral load resistance. To account for this behavior, drift calculations should be based on a computer model that places hinges in the connections that continue to yield under the action of lateral loads. A good approximation of the drift can also be found by assuming that the drift for a frame with wind connections is twice that of a frame with full moment connections.

4.7 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 wall bracing.
2. X-Braced roof systems with wall bracing.
3. Diaphragms and rigid frames.

The systems can be mixed to provide the optimum structure, for example rigid frames in one direction and vertical bracing and diaphragms in the perpendicular direction.

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

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

As a general 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 designer's first choice as a system. The four parameters listed above can cause a different framing system to be used.

Building Geometry

As mentioned in the discussion on diaphragms and horizontal bracing when the length to width ratio of the structure between vertical braces exceeds approximately 4 to 1, the structural requirements placed on the diaphragm or horizontal bracing system become severe. In addition uplift forces become significant at bracing locations. For these structures the most economical approach is to create rigid frames with joist girders. In all likelihood, the Basic Connection will not be suitable, since relatively few interior columns would be available to participate in the rigid 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 long direction of the building, the first choice would be to transfer the lateral loads to the sidewalls using diaphragm action with vertical wall bracing.

Expansion Joints

When the structure is of such a size that expansion joints are required, and these expansion joints destroy the integrity of the roof diaphragm, a rigid frame solution is necessary. If the diaphragm shears can be transferred across a singular expansion joint and the resulting diaphragm has a span-to-width ratio of less than 4 to 1, then the diaphragm solution should still provide the most economical system.

Roofing System

When a standing seam roof is used, either a horizontal roof bracing system with vertical bracing or rigid frames must be used. Again, if the length-to-width ratio is greater

than 4 to 1, the rigid 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 rigid frame system will probably be less expensive than the horizontally braced system. The rigid frame solution will most likely have heavier columns than the X-braced system, but the erection cost of the X-bracing will be more expensive than the extra cost for the columns.

Future Expansion

Usually, future expansion considerations only influ-

ence the lateral bracing system since vertical bracing may not be permitted where the expansion will occur. If this is the situation, then a rigid frame may have to be used.

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. The building is to be a “braced” structure using X-braces at the perimeter walls in conjunction with a roof diaphragm. From code requirements, it is concluded that wind load controls. Use a wind load of 20 psf (pressure plus suction) for a fifty year mean reoccurrence interval.

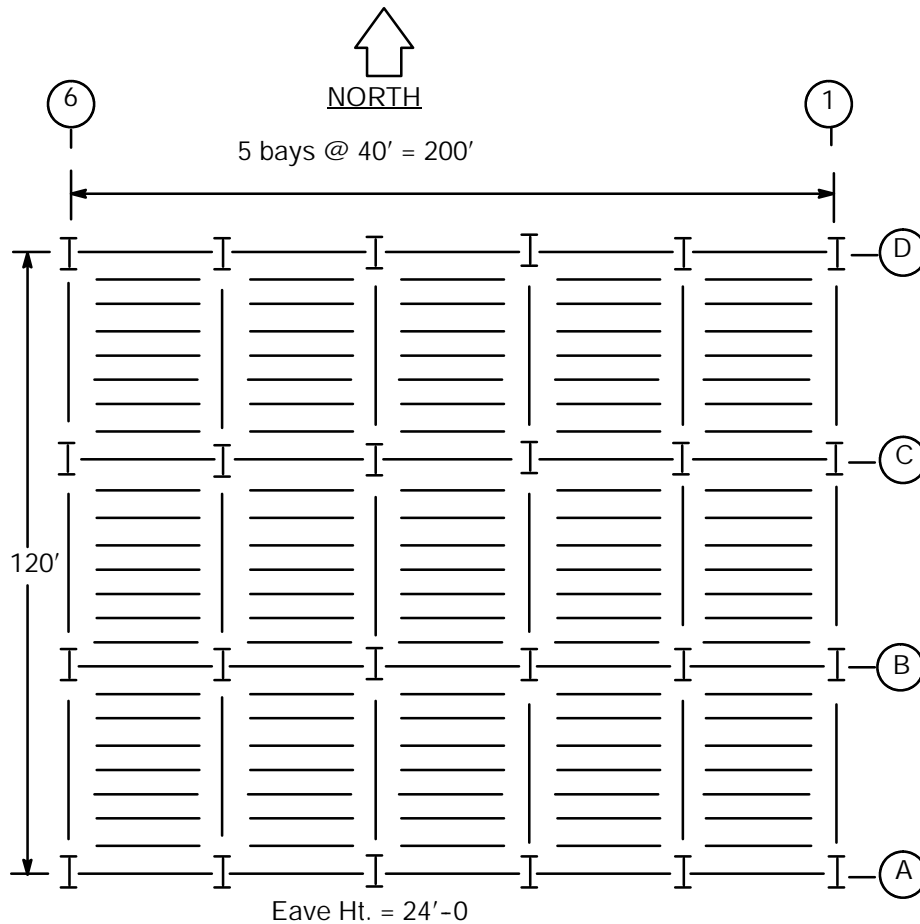


Fig. 4.8.1 Example 4.8.1

Live Load	= 20 psf	Joists at 5'-0, o/c , Typical joist 22K7
Collateral	= 5 psf	JG1 42G 8 N 4.6K (exterior typ.)
Dead Load	= 17 psf	JG2 42G 8N 9.2K (interior typ.)
Joist	= 3 psf	22 ga. Wide Rib Deck
Joist Girders	= 1 psf	
Total	= 46 psf	

Solution:

1. Diaphragm Design:

Determine the shear at each wall.

The wall shear equals the eave force per foot times one half of the length of the wind loaded wall.

Lines 1 & 6:

$$V_1 = V_6 = (0.020)(24/2)(200/2) = 24.0 \text{ kips}$$

Line A & D:

$$V_A = V_D = (0.020)(24/2)(120/2) = 14.4 \text{ kips}$$

Determine the maximum shear force per foot in the diaphragm.

Diaphragm shear equals the wall shear divided by the wall length.

$$v_1 = v_6 = (24.0)(1000)/120 = 200 \text{ plf}$$

$$v_A = v_D = (14.4)(1000)/200 = 72 \text{ plf}$$

Select a fastening pattern from Vulcraft's Steel Floor and Roof Deck Catalog. The following are possible selections:

For welded support fasteners and screwed sidelaps (welded sidelaps are not recommended for 22 ga. decks);

36/4 weld pattern with (1) #10 Tek sidelap screw.

$$v_{\text{allow}} = 218 \text{ plf}$$

or

36/5 weld pattern without sidelap screws.

$$v_{\text{allow}} = 228 \text{ plf}$$

For power driven fasteners;

36/4 Ramset 26SD drive pins with (1) #10 Tek sidelap screw

$$v_{\text{allow}} = 236 \text{ plf}$$

or

36/5 Ramset 26SD drive pins with no sidelap screws

$$v_{\text{allow}} = 241 \text{ plf}$$

It should be noted that one sidelap fastener is required at this joist spacing to meet Factory Mutual and SDI requirements.

Choose:

36/4 weld pattern with (1) #10 Tek sidelap screw.

2. Check the diaphragm chord forces at lines A and D:

The edge joists are assumed to resist the diaphragm chord force.

Determine the diaphragm chord force in the edge joists due to wind loading.

$$M_{A\&D} = wL^2/8 = (0.02 \times 24/2)200^2/8 = 1200 \text{ ft.-kips}$$

$$P_{\text{chord}} = M/\text{diaphragm depth} = 1200/120 = 10 \text{ kips}$$

Determine the top chord force in the joist due to gravity loading.

$$M_{D+L} = wL^2/8 = (0.045 \times 5/2)40^2/8 = 22.5 \text{ ft.-kips}$$

$$P_{D+L} = M/\text{depth} = 22.5 \times 12 / (22 - 2 \times 0.5) = 12.9 \text{ kips (0.5 assumed dist. to centroid)}$$

Combine gravity load + wind load.

$$P_{D+L+W} = 12.9 + 10 = 22.9 \text{ kips}$$

Determine the allowable joist chord force for the typical edge joist (22K7):

For the 22K7 the allowable gravity load = 231 plf

$$M_{\text{allow}} = 0.231 \times 40^2/8 = 46.2 \text{ ft.-kips}$$

$$P_{\text{allow}} = 46.2 \times 12/21 = 26.4 \text{ kips}$$

$$26.4 \text{ kips} > 22.9 \text{ kips} \quad \text{o.k.}$$

The 22K7 is adequate as the diaphragm chord at A and D provided eccentric bending is eliminated.

To eliminate bending stresses in the joist top chord use a tie plate between adjacent joists to transfer the chord force.

3. Size the tie plate and weld:

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

Try a 4" x 3/16" plate.

Assuming an unwelded (unsupported) plate length of 4", check the buckling capacity of the plate:

Plate properties:

$$A = 0.75 \text{ in}^2.$$

$$r_x = 0.054 \text{ in.}$$

$$L/r_x = 4/0.054 = 74$$

Per AISC Sect. E:

$$F_a = 16.01$$

$$f_a = P_{\text{diaph}}/A = 10/0.75 = 13.3 \text{ ksi}$$

$$f_a \leq F_a$$

$$13.3 \text{ ksi} < 16.01 \text{ ksi.} \quad \text{o.k.}$$

Using a 3/16 inch E70 fillet weld:

The allowable weld strength = 2.78 kips/in.
Weld length req'd = $10/(2.78) = 3.6$ inches.

Use 4 inches of 3/16 inch fillet on each end of each plate.

4. Design the diaphragm edge angle at lines 1 and 6:

Since deck sidelap fasteners are required, an edge angle is required at lines 1 and 6. This angle will be specified as continuous and used as the diaphragm chord at these edges.
Try an edge angle 2-1/2 x 2-1/2 x 3/16.

Determine the diaphragm chord force at lines 1 and 6:

$$M_{1\&6} = (0.02 \times 24/2) 120^2/8 = 432 \text{ ft.-kips}$$

$$P_{\text{chord}} = 432/200 = 2.2 \text{ kips}$$

Determine the axial stress in the 2.5x2.5x3/16 inch edge angle:

$$f_a = P/A = 2.2/0.902 = 2.4 \text{ ksi}$$

Determine the allowable stress for the edge angle per AISC, ASD Spec. Sect. E. To determine the slenderness ratio for the edge angle use r_y . The welding of the angle to the joists and to the deck is assumed to prevent buckling about r_z . The joists are spaced sixty inches apart.

$$L/r_y = 60/0.778 = 77$$

$$F_a = 15.7$$

$$2.4 < 15.7 \quad \text{o.k.}$$

5. Design the force transfer into the vertical bracing at lines A and D.

Locate the bracing in the middle of each wall to allow thermal movement in each direction from the brace.

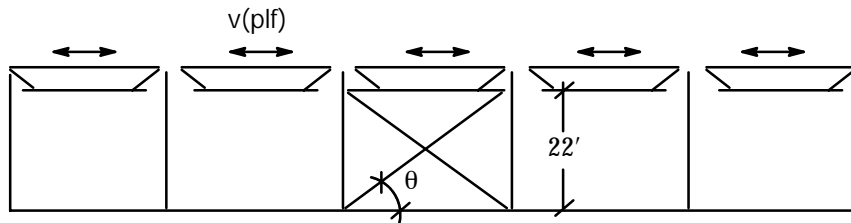


Fig. 4.8.2 Schematic Elevation at Wall

The maximum force that must be transferred between joists,

V_{diap} equals the length of two bays times the diaphragm force per foot.

$$V_{\text{diap}} = (80)(72)/1000 = 5.8 \text{ kips.}$$

This is less than the 10 kip chord force for which the tie plates were previously designed, therefore, the tie plates are adequate to transfer the 5.8 kip force to the braced bay.

Connect the vertical brace to the bottom chord of the joist. See Fig. 4.8.2.

The brace shear equals the shear at wall A or D = 14.4 kips.

Size the vertical bracing at A and D:

$$\text{Brace length} = (40^2 + 22^2)^{.5} = 45.7 \text{ ft.}$$

$$\text{Brace force} = (V)(\sec \theta)$$

$$\text{Brace force} = (14.4)(45.7/40) = 16.5 \text{ kips}$$

Determine the brace area required (A36 angle):

$$A_{\text{req'd}} = 16.5/(22) = 0.75 \text{ in}^2(\text{net}).$$

Determine the radius of gyration required. Use a maximum slenderness ratio of 300. For the in-plane slenderness ratio of single angles, 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 . See Reference 16.

$$r_z(\text{req'd}) = (45.7/2) (12)/300 = 0.914 \text{ in.}$$

$$r_y(\text{req'd}) = (0.75)(45.7)(12)/300 = 1.37 \text{ in.}$$

Use an \angle 5x5x5/16

$$A = 3.03 > 0.75$$

$$r_z = 0.994 > 0.914$$

$$r_x = r_y = 1.57 > 1.37$$

If the joists are not welded to the joist girder seats at the column tops, the joist top chord force can be transferred thru the joist web system to the bottom chord and then to the X-braces. If the joists are welded to the joist girder seats, a

rigid frame is created. The rigid frame action will create extra joist chord forces and seat rollover forces. For this example the joists will not be welded to the joist girder seats.

6. Design the force transfer into the bracing at Lines 1 and 6.

The edge shear ($v = 200$ plf) is transferred into the joist girders through the joist seats. This edge shear tends to cause a rollover in the joist seats.

The rollover force per joist equals the edge shear times the joist spacing $= (200)(5) = 1000$ lbs. $= 1$ kip.

From Section 7.6.1, the rollover capacity $= 1.92$ kips.

Therefore rollover capacity is adequate.

Along lines 1 and 6, the chord force must be transferred from joist girder to joist girder. Unless reinforcement is added, the force must be transmitted through the joist girder seat into the column cap then into the adjacent joist girder seat. Determine if the joist girder seat requires reinforcement.

Shear force in edge girders between A - B and C - D must be transferred into the girder between B - C.

$$V_{A-B} = V_{C-D} = (200)(40) = 8000 \text{ lbs.} = 8 \text{ kips.}$$

The Basic Connection is adequate to transfer this chord force between girders. Refer to Table 7.1.1.

Connect the vertical brace to the bottom chord of the joist girder.

Size the vertical brace along Lines 1 and 6. Use X-bracing between Lines B and C.

$$\text{Brace height} = 20.3 \text{ ft.}$$

$$\text{Brace length} = (20.3^2 + 40^2)^{.5} = 44.9 \text{ ft.}$$

$$\begin{aligned} \text{Brace force} &= (0.200)(40)(3)(44.9/40) \\ &= 26.94 \text{ kips.} \end{aligned}$$

$$\text{Area (req'd)} = 26.94/22 = 1.22 \text{ in.}^2 \text{ (net).}$$

$$r_z \text{ (req'd)} = (44.9/2)(12)/300 = 0.9$$

$$r_y \text{ (req'd)} = (0.75)(44.9)(12)/300 = 1.35$$

Use an $\angle 5 \times 5 \times 16$.

7. Evaluate the effects of rigid frame action caused by welding the joist girder bottom chord in the braced bay.

The welding will be done after dead loads are applied.

From a rigid frame analysis:

$$M_{LL} = 209 \text{ inch-kips} = 17.4 \text{ ft.-kips.}$$

Determine the joist girder chord force:

$$P_{LL} = M/d = 209/(42-2 \times 0.5) = 5.1 \text{ kips.}$$

Refer to Table 7.1.1 for the Basic Connection

$$P_{LL} = 5.1 \text{ kips} < P_{\text{allow}} = 8 \text{ kips}$$

$$P_{LL+W} = 13.1 \text{ kips} > P_{\text{allow}} = 8 \text{ kips}$$

The Basic Connection is inadequate.

It should be noted that the joist girders could be detailed to slide on top of the columns thus eliminating the continuity moments; however, connection ties would then be required to transfer the edge loading from girder to girder, or a larger edge girder could be used.

8. Check the maximum deflection of the roof diaphragm:

Determine the effective stiffness of the deck:

Refer to the Vulcraft Steel Floor and Roof Deck Catalog for the specified deck:

$$G' =$$

$$\begin{aligned} & \frac{K2}{3.78 + (0.3 \times DB/SPAN) + (3 \times K \times SPAN)} \\ K &= 0.50951 \quad SPAN = 5'-0'' \\ DB &= 1072 \\ K2 &= 870 \end{aligned}$$

$$\begin{aligned} G' &= \frac{870}{3.78 + (0.3 \times 1072/5) + (3 \times 0.50951 \times 5)} \\ &= 11.486 \text{ (kips/in.)/ft.} \end{aligned}$$

The equation for the calculation of shear deflection is:

$$\Delta_s = \frac{WL^2}{8DG'}$$

where

$$\begin{aligned} W &\text{ equals the eave force (kips/ft.)} \\ &= 0.24 \text{ kips/ft.} \end{aligned}$$

$$L \text{ equals the diaphragm length (ft.)} = 200 \text{ ft.}$$

$$D \text{ equals the diaphragm depth (ft.)} = 120 \text{ ft.}$$

$$\Delta_s = \frac{(0.24)(200)^2}{(8)(120)(11.486)} = 0.87 \text{ inches}$$

LATERAL LOAD SYSTEMS

The equation for the bending deflection is:

$$\Delta_b = \frac{5WL^4}{384EI}$$

$I = 2A(D/2)^2$ and is normally calculated using A equal to the perimeter member area (diaphragm flange). Conservatively the top chord area of the 22K7 equals one square inch.

Thus, $I = (2)(1)(60)^2(144) = 1,037,000 \text{ in.}^4$

$$\Delta_b = \frac{(5)(0.24)(200^4)(1728)}{(384)(29,000)(1,037,000)} = 0.29 \text{ in.}$$

$$\Delta_{\max} = \Delta_s + \Delta_b$$

$$\Delta_{\max} = 0.87 + 0.29 = 1.16 \text{ inches.}$$

The maximum sidesway for a ten year wind is approximately 75% of the sidesway for a 50 year wind.

$$\therefore \Delta_{\max} \text{ for a 10 year wind} = 0.75(1.16) = 0.87 \text{ inches.}$$

The building designer should evaluate if the deflection is o.k.

The following figures illustrate the specification of the joists and girders for the example problem.

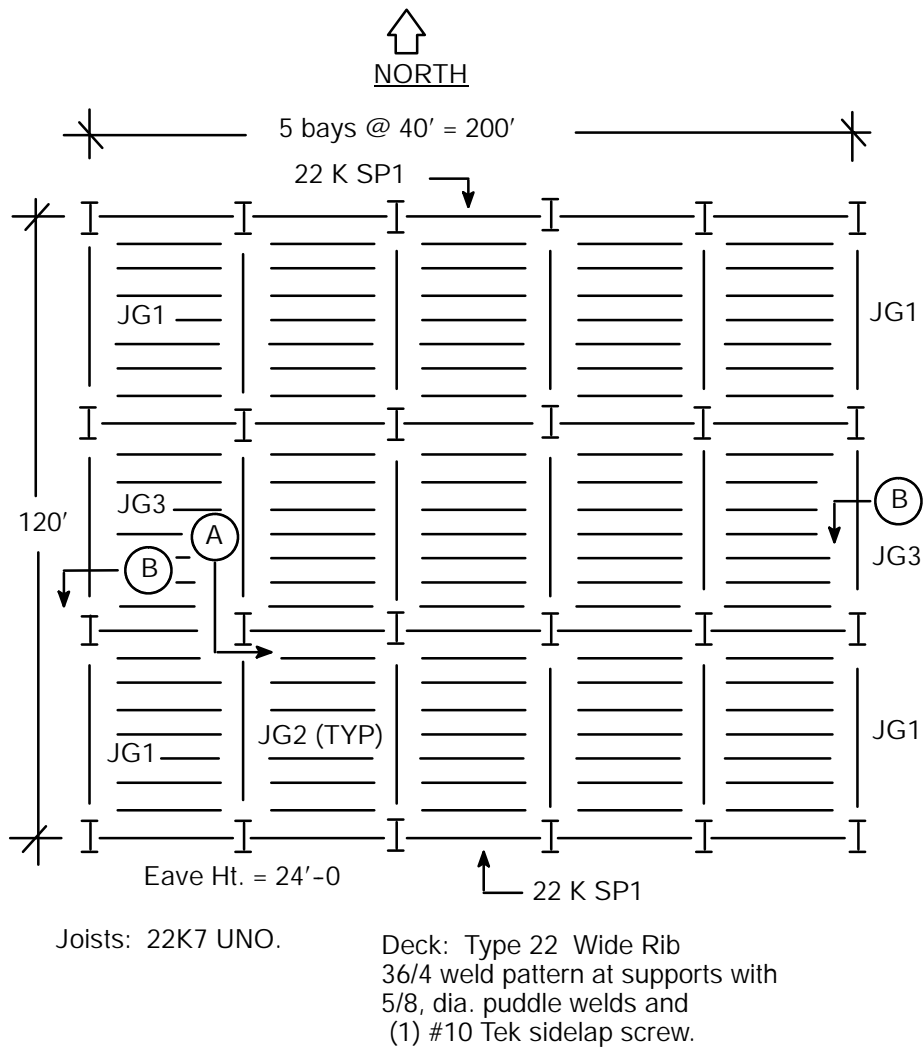


Fig. 4.8.3 Plan of Final Design

LATERAL LOAD SYSTEMS

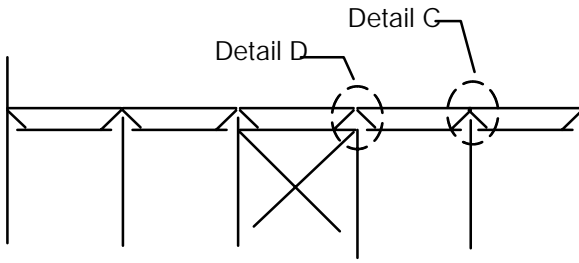


Fig. 4.8.4 North Elevation and South Elevation

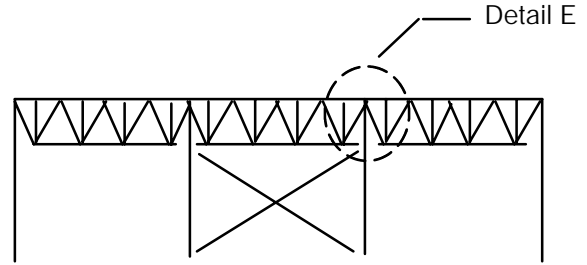
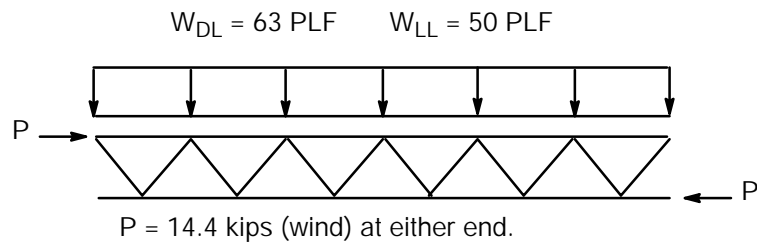

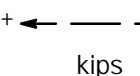


Fig. 4.8.5 East Elevation and West Elevation



Note: Design a symmetrical web system to transfer the force P from the top chord to the bottom chord.

Joist 22 K SP1

JOIST GIRDER SCHEDULE						
Mark	Size	Moment Live Load		Chord Forces Wind		Notes
		<div>+  +</div>		<div>+  +</div>		
		Left	Right	Top	Bottom	
JG1	42G8N4.6K	--	--	± 8	--	
JG2	42G8N9.2K	--	--	--	--	
JG3	42G8N4.6K	17.4	-17.4	-24	-24	See diagram above for Chord Force application.

Loads shown are unfactored loads (ASD)

Fig. 4.8.6 Joist Diagram and Girder Schedule

LATERAL LOAD SYSTEMS

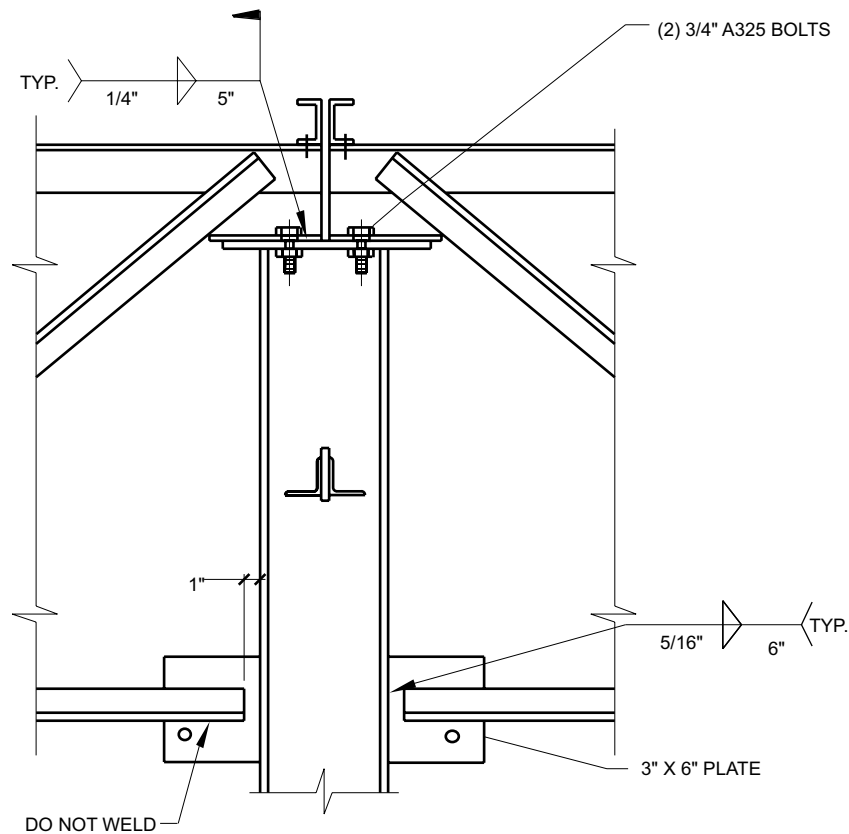


Fig. 4.8.7 Detail A

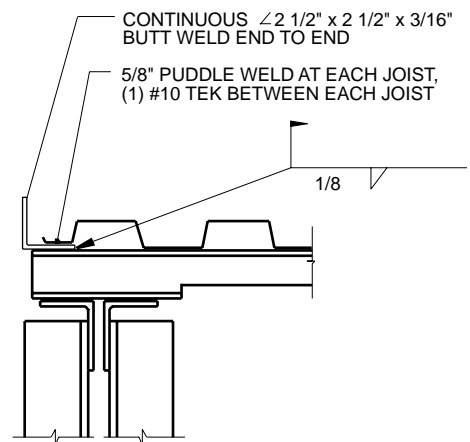


Fig. 4.8.8 Detail B

LATERAL LOAD SYSTEMS

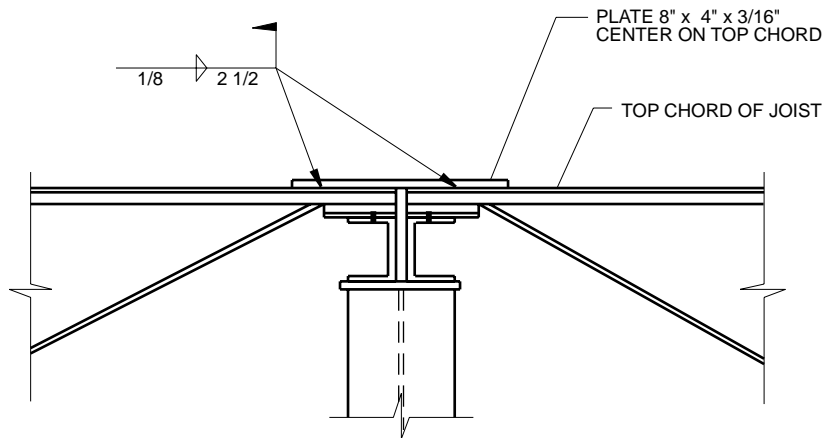


Fig. 4.8.9 Detail C

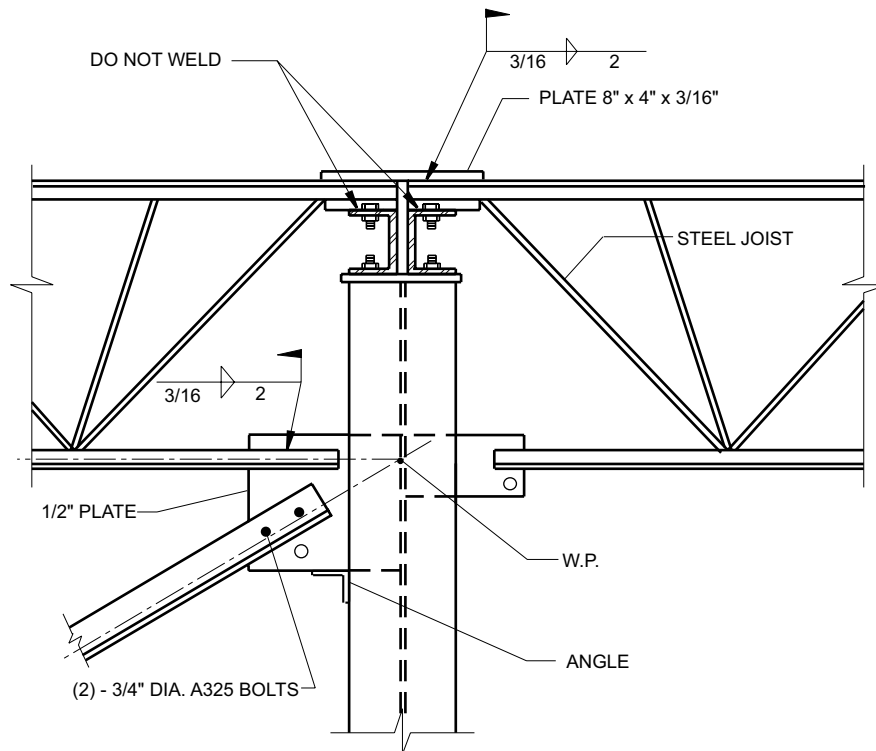


Fig. 4.8.10 Detail D

LATERAL LOAD SYSTEMS

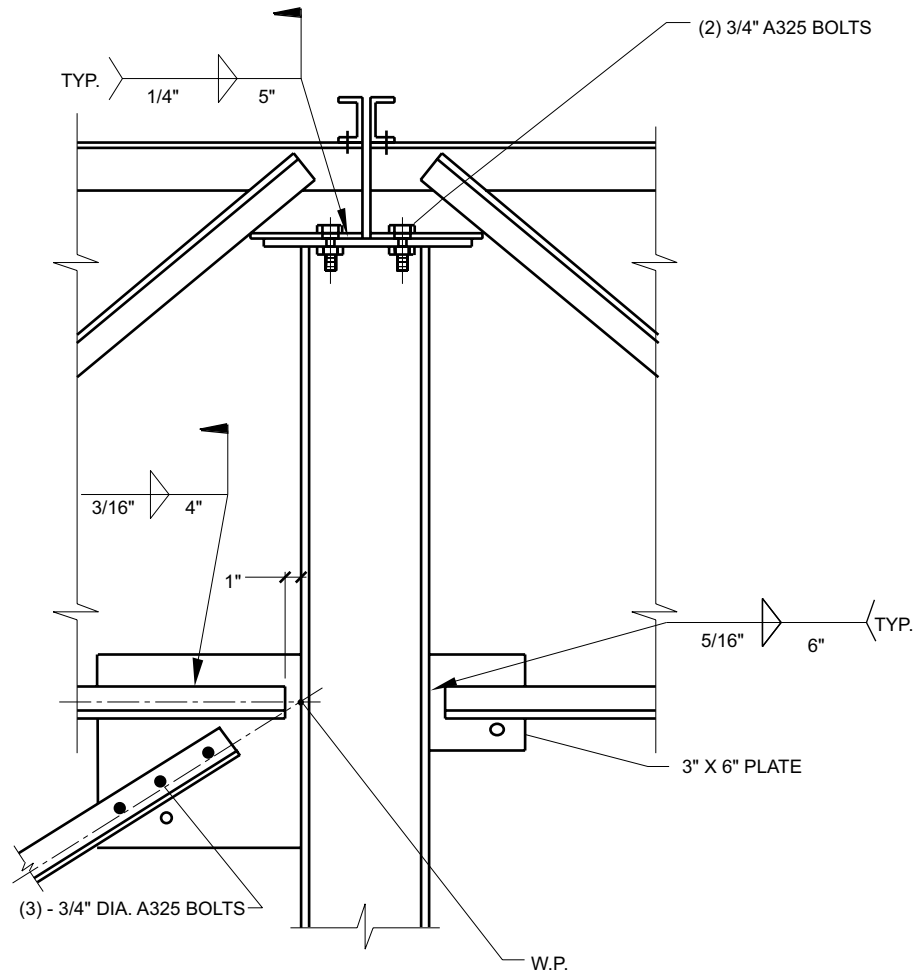
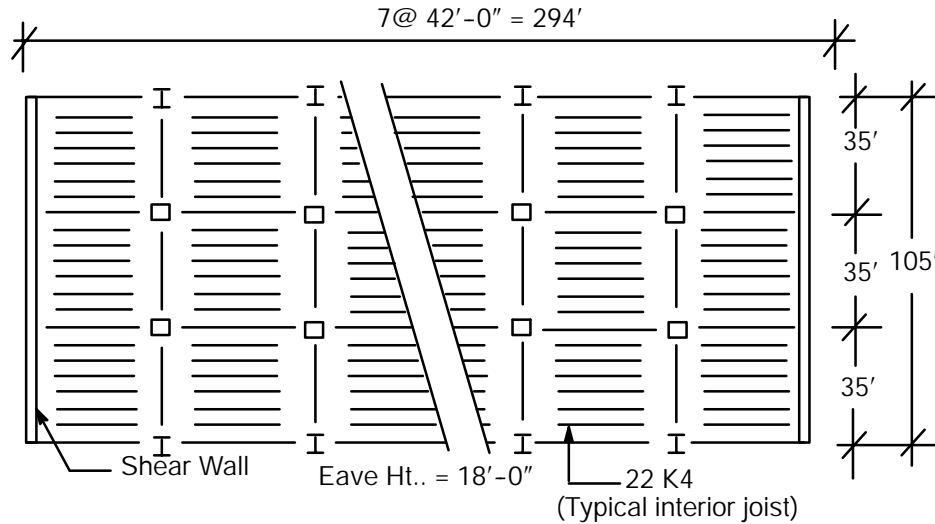


Fig. 4.8.11 Detail E

Example 4.8.2 Rigid Frame Building

Design a rigid frame lateral load resisting system in the di-

rection of the joist girders and joists for the building shown in Figure 4.8.12.



Dead Load = 5 psf (Standing Seam Roof)
 Joist Girder Weight = 1 psf
 Collateral = 4 psf
 LL = 20 psf (Live Load Reduction Allowed); (12 psf - Girders only)
 Wall Wind Load = 25 psf (Pressure + Suction)
 Roof Uplift = 18 psf (Suction)

Fig. 4.8.12 Example 4.8.2

Solution:

1. Design the joist girder rigid frame.

Determine the joist girder moment of inertia (36G7N).

$$\begin{aligned}
 P &= (DL + Coll. + LL_{red.})(5)(42) \\
 P &= (5 + 1 + 4 + 12)(5)(42)/1000 = 4.6 \text{ kips} \\
 I_G &= 0.027PNLd \\
 I_G &= (0.027)(4.6)(7)(35)(36 - 2 \times 0.5) \\
 &= 1065 \text{ in.}^4
 \end{aligned}$$

(Note: 0.5 is the estimated centroid distance for the chord angles.)

Provide a rigid connection between the perimeter column and the joist girder as shown in the Figure 4.8.13. The connection is to be made after all dead loads are applied. The Basic Connection is used for the joist girders at the interior columns.

Based on a rigid frame analysis, the following joist girder end moments were determined.

$$M_{wind} = \pm 940 \text{ in.-kips}; M_{live} = 210 \text{ in.-kips}$$

The moments must be magnified to account for $P\Delta$ effects and the "leaner" column effect.

From the analysis, the eave deflection at full wind load is 2.4 inches. This is equivalent to $H/120$ for a 10 year wind, using 75% of full wind.

Check story stability:

Only the rigidly connected columns contribute to the story stability.

Assume the governing code requires $DL + LL$ and $DL + LL + W$ be considered. (Note: Collateral load is treated as a dead load.)

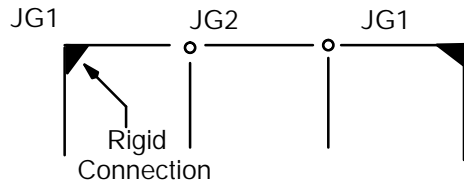
For $DL + LL$:

Total story dead load per bay (including collateral):

$$\Sigma P_{DL} = (10)(42)(105)/1000 = 44.1 \text{ kips.}$$

Total story live load per bay (with LL reduction):

$$\Sigma P_{LL} = (12)(42)(105)/1000 = 53.0 \text{ kips.}$$



JG1 and JG2 are 36" deep,
with 7 panel spaces.

Exterior Columns:
W14x38 ($F_y = 50$ ksi)

Properties:
 $A = 11.2 \text{ in}^2$
 $I_x = 385 \text{ in}^4$
 $r_x = 5.87 \text{ in}$

Interior Columns:
HSS 8x8x1/4 ($F_y = 46$ ksi)

Properties:
 $A = 7.10 \text{ in}^2$
 $I_x = 70.7 \text{ in}^4$
 $r_x = 3.15 \text{ in}$

Fig. 4.8.13 Rigid Frame

Thus, $\Sigma P_{DL+LL} = 97.1 \text{ kips}$

Determine $\Sigma P_{allowable}$:

Use the AISC nomograph. (AISC Fig. C-C2.2)

$$\begin{aligned} I_c &= 385 \text{ in}^4 \\ L_c &= 16.5 \text{ ft. (to girder mid-depth)} \\ I_g &= 1065 \text{ in}^4; I_{eff} = 1065/1.15 = 926 \text{ in}^4 \\ L_g &= 35 \text{ ft.} \end{aligned}$$

To account for the far end of the joist girder being pinned multiply $\Sigma I_g/L_g$ by 1/2.

$$G_A = \frac{\Sigma I_c/L_c}{(1/2)\Sigma I_g/L_g} = \frac{385/16.5}{(1/2)(926)/35}$$

$$G_A = 1.76; G_B = 10$$

From the nomograph; $K \approx 2.0$

$$KL/r_x = (2.0)(16.5)(12)/5.87 = 67.5$$

From AISC Eq. (E2-1); $F_a = 21.4 \text{ ksi}$.

$$\begin{aligned} P_{allowable} &= AF_a = (11.2)(21.4) \\ &= 239.7 \text{ kips/col.} \end{aligned}$$

$$\begin{aligned} \Sigma P_{allowable} &= 479.4 \text{ kips} \\ \Sigma P_{DL+LL} / \Sigma P_{allowable} &= 97.1/479.4 \\ &= 0.20 < 1.0 \end{aligned}$$

Story stability is adequate.

Determine moment magnifiers:

The live load moments are not magnified since the structure is symmetrical and no sway is associated with the live load moment.

The column and joist girder wind moments must be magnified.

Determine P_e :

$$\begin{aligned} P_e &= \frac{12\pi^2 EA}{23(KL/r_x)^2} \\ P_e &= \frac{(12)(\pi^2)(29000)(11.2)}{(23)(2.0 \times 16.5 \times 12/5.87)^2} = 367 \text{ kips/col.} \end{aligned}$$

For the wind moments:

$$\left(1 - \frac{\Sigma P}{\Sigma P_e}\right) = \left(1 - \frac{97.1}{734}\right) = 0.87$$

Thus the joist girder wind end moment must be specified as:

$$940/0.87 = 1080/12 = \pm 90 \text{ ft./kips.}$$

The live load end moment can be specified as:

$$210/12 = 17.5 \text{ ft./kips.}$$

2. Design the joist girder end connection.

Support the girder on a column bracket and use a top plate welded to the joist girder and to the column cap plate to transfer the force.

Top chord:

DL+LL+W:

$$M_{end} = 210 + 1033 = 1243 \text{ in.-kips. (DL End moment = 0)}$$

$P_{chord} = M/d = 1243/(36-0.5) \approx 35 \text{ kips}$ (0.5 in. assumed as the distance to the tension chord centroid). The distance to the top chord centroid is not deducted because the plate rests on the top chord.

$$A_{req'd} = P_{chord}/F_t$$

$$A_{req'd} = 35/(22) = 1.59 \text{ in.}^2$$

$$\text{Use a } 5" \times 3/8" \text{ plate; } A = 1.875 \text{ in.}^2$$

Check the plate for compression. The live load moments only create tension in the top plate, therefore only wind load moments are used for this check.

$$P_{chord} = -1033/35.5 = -29.1 \text{ kips (comp.)}$$

$$L_x = 5 \text{ in. (assumed unbraced length)}$$

$$\begin{aligned} I_x &= bt^3/12 = (5)(0.375)^3/12 \\ &= 0.022 \text{ in.}^4 \end{aligned}$$

$$r_x = (I_x/A)^{.5} = 0.11 \text{ in.}$$

$$L_x/r_x = 45.5$$

$$f_a = 29.1/1.56 = 18.7 \text{ ksi}$$

AISC, Sect. E $\rightarrow F_a = 18.74$ ksi

$$f_a < F_a \quad \text{o.k.}$$

Determine the top plate weld required:

3/16" fillet weld $v_{\text{allow}} = 2.78$ kips/inch

Required weld length = $32/(2.78) = 11.51$ inches.

Use a total of 12 inches of 3/16 weld at each end of the plate.

Determine the bottom chord connection (to stabilizer plate):

Try 6"x3/4" plate (min. size for detailing);

$$A = 4.5 \text{ in.}^2$$

$$f_a = 35/4.5 = 7.78 \text{ ksi}$$

$$F_a = (0.6)(F_y) = 22 \text{ ksi}$$

$$f_a < F_a \quad \text{o.k.}$$

Size the weld from the girder chord to the stabilizer plate. Per AISC Sect. J: 1/4 inch minimum weld is required.

$$v_{\text{allow}} = 3.71 \text{ kips/inch;}$$

$$\text{Weld length} = 35/(3.71) = 9.43 \text{ inches.}$$

Use 10 inches of 1/4" fillet weld total.

Check the column web for local web yielding and web crippling per AISC Sect. K:

$$W14x38 \quad (F_y = 50 \text{ ksi}).$$

AISC Equation (K1-2):

f_w = the column web stress.

$$f_w = R/(t_w(N+5k)) = 35/(0.31(6+5(1.06)))$$

$$= 10.0 \text{ ksi}$$

$$\text{Allowable web stress} = 0.66F_y = 0.66 \times 50$$

$$= 33 \text{ ksi}$$

$$f_w < 33 \text{ ksi} \quad \text{o.k.}$$

AISC Equation (K1-4):

R must be greater than the chord force.

$$R = 67.5t_w^2[1+3(N/d)(t_w/t_f)^{1.5}](F_{yw}t_f/t_w)^{.5}$$

$$R = 67.5(0.31)^2[1+3(6/14.10)(0.31/0.515)^{1.5}][50(0.515/0.31)]^{.5}$$

$$R = 94.4 \text{ kips}$$

$$R > 35 \text{ kips} \quad \text{o.k.}$$

3. Check the W14x38 columns:

Properties:

$$A = 11.2 \text{ in}^2 \quad F_y = 50 \text{ ksi}$$

$$S_x = 54.6 \text{ in}^3 \quad r_T = 1.77$$

$$r_x = 5.87 \text{ in.} \quad L_y = 16.1 \text{ in. (to joist)}$$

bottom chord)

$$r_y = 1.55 \text{ in.} \quad C_b = 1.75$$

$$KL/r_y = (1)(16.1)(12)/1.55$$

$$= 125 \rightarrow F_a = 9.56 \text{ ksi}$$

$$d/A_f = 4.04 \rightarrow F_b = 26.9 \text{ (controls) ksi}$$

$$L/r_T = (16.1)(12)/1.77$$

$$= 109 \rightarrow F_b = 22.4 \text{ ksi}$$

$$P_{DL+LL} = (10+12)(35/2)(42) = 16.2 \text{ kips}$$

$$f_a = P/A = 1.45 \text{ ksi}$$

$$P_{DL+LL+W} = (10+12-18)(35/2)(42)$$

$$= 2.94 \text{ kips}$$

$$f_a = P/A = 0.26 \text{ ksi}$$

$$M_{LL} = 210 \text{ in.-kips}$$

$$f_b = M/S_x = 3.85 \text{ ksi}$$

$$M_W = 1033 \text{ in.-kips}$$

$$f_b = M/S_x = 18.92 \text{ ksi}$$

$$M_{DL+LL+W} = 210+1033 = 1243 \text{ in.-kips;}$$

$$f_b = M/S_x = 1243/54.6 = 22.8 \text{ ksi}$$

By inspection DL+LL does not control.

For DL + LL + W:

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{\sum P}{\sum P_e}\right) F_b} = \frac{0.26}{9.56} + \frac{(0.85)(22.8)}{(0.87)(26.9)}$$

$$= 0.86 < 1.0$$

$$\frac{f_a}{0.6F_y} + \frac{f_b}{F_b} = \frac{0.26}{30} + \frac{22.8}{26.9} = 0.86 < 1.0$$

Use W14x38 ($F_y = 50$)

4. Design the rigid frame in the joist direction:

A schematic of the rigid frame used in the joist direction is shown in Figure 4.8.14.

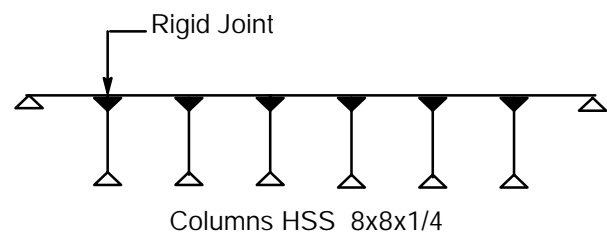


Fig. 4.8.14 Schematic of Rigid Frame

Using joists with $I = 200 \text{ in}^4$ a frame analysis indicated a drift of 2.20 inches under full wind load.

The typical joist for the roof is a 22K4.

The moment of inertia for a 22K4 can be found as follows:

From the joist load tables:

$$\begin{aligned} W_{LL} &= 79 \text{ plf for } L/360 \\ I_j &= 26.767(W_{LL})(L^3)(10^{-6}) \\ L &= 42-0.33 = 41.67 \text{ feet} \\ I_j &= (26.767)(79)(41.67)^3(10^{-6}) = 153 \text{ in}^4 \end{aligned}$$

To provide $I_j = 200 \text{ in}^4$ requires a joist with a live load capacity of $(79)(200)/(153) = 104 \text{ plf}$ (By proportioning).

Use a 22K7 at all column lines.

From the analysis the maximum wind end moment on the joist is $\pm 166 \text{ in-kips}$. The maximum live load end moment is 227 in-kips .

Check story stability.

Determine $\Sigma P_{\text{allowable}}$:

Use the AISC nomograph.

$$\begin{aligned} I_c &= 70.7 \text{ in}^4 \\ L_c &= 17.1 \text{ ft. (to joist mid-depth)} \\ I_j &= 200 \text{ in}^4 \\ L_j &= 42 \text{ ft.} \end{aligned}$$

For the (4) interior columns:

$$G_A = \frac{\Sigma I_c/L_c}{\Sigma I_g/L_g} = \frac{70.7/17.1}{(2)(200)/42} = 0.43$$

$$G_B = 10$$

From the nomograph: $K = 1.8$

$$KL/r_x = (1.8)(17.1)(12)/(3.15) = 117.3$$

From AISC Eq. (E2-1), $F_a = 10.67 \text{ ksi}$

$$P_{\text{allowable}} = AF_a = (7.10)(10.67) = 75.8 \text{ kips/col.}$$

For the (2) exterior columns:

$$G_A = \frac{70.7/17.1}{200/42 + (1/2)(200/42)} = 0.58$$

$$G_B = 10$$

From the nomograph: $K = 1.8$

Note that for this example the effect of the hinge at the far end of the joist is negligible.

$$\Sigma P_{\text{allowable}} = (6)(75.8) = 455 \text{ kips}$$

$$\Sigma P_{D+L} = (10+12)(294)(35)/1000 = 226$$

$$\Sigma P_{D+LL} < \Sigma P_{\text{allowable}} : 226 < 455$$

Story stability is adequate.

Moment magnifier for wind moments: $DL+LL+W$:

$$\begin{aligned} P_e &= \frac{12\pi^2 EA}{23(KL/r)^2} \\ &= \frac{(12)(\pi^2)(29000)(7.10)}{(23)(1.8 \times 17.1 \times 12/3.15)^2} = 77.1 \text{ kips/col.} \end{aligned}$$

$$\Sigma P_{D+LL+W} = (10+12)(294)(35)/1000 = 226$$

$$\Sigma P/\Sigma P_e = 226/(6 \times 77.1) = 0.49$$

Thus, the magnifier for second order $P\Delta$ effects equals $1/(1-0.49) = 1.96$.

The wind moments on the joists and the columns should be increased by 96%. The maximum wind moment equals $(1.96)(166) = 325 \text{ in-kips}$.

Both the live load moment and the wind moments must be specified to the joist manufacturer.

Determine the rollover force on the joist girder seats due to the joist moments.

The chord forces in the joists are as follows:

For $DL+LL$:

$$F = M/d = 227/(22-2.5-0.5) = 11.9 \text{ kips}$$

Note that since the connection of the joist is at the seat to column interface, the depth d is based on the joist depth minus the seat depth (2.5") minus the distance from the outside of the bottom chord to its centroid.

For $DL+LL+W$:

$$F = 552/19 = 29 \text{ kips}$$

For $DL+W$:

$$F = 325/19 = 17.1 \text{ kips}$$

Based on these chord forces, E member extensions are required on the joists. (See Section 7.1). The specifying engineer must indicate this requirement in the contract documents.

Using the Basic Connection, two joists will rest on two joist girder seats at each interior column. Thus if both joists have equal wind end moments, the rollover force will be split between the two seats. The greatest rollover force is due to the wind force of the 17.1 kips. The capacity of a joist girder seat with 7/16 inch seat angles (Detail G, Fig. 4.5.9) is 4.0 kips (ASD). Since

this is less than the 17.1 kips required, check if placing stiffeners in the seats will work. (Detail I, Fig. 4.5.11). The capacity of Detail I equals = 8.55 kips.

Thus, the interior joist girders must be placed on column brackets and the joists connected directly to the column tops.

Determine the joist bottom chord connection:

To prevent the HSS wall from being overstressed, the joist bottom chord should be welded to an angle which in turn is connected to the tube. The reader is referred to Example 7.4.4 for the details relative to this design.

5. Check the HSS 8x8x1/4 column:

Properties:

$$\begin{aligned} A &= 7.10 \text{ in}^2 \\ S &= 17.7 \text{ in}^3 \end{aligned}$$

$$\begin{aligned} r &= 3.15 \text{ in.} \\ KL/r_y &= (1)(18)(12)/3.15 = 69 \\ KL/r_x &= (1.8)(17.1)(12)/3.15 = 11 \text{ (Controls)} \\ F_e' &= 10.67 \text{ ksi} \\ F_a &= 10.67 \text{ ksi} \\ F_b &= 0.6(46) = 27.6 \text{ ksi} \end{aligned}$$

$$\begin{aligned} P_{DL+LL} &= P_{DL+LL+W} \\ &= (10+12)(42)(35)/1000 = 32.4 \text{ kips} \end{aligned}$$

The column wind moment equals the sum of the joist end moments.

$$\begin{aligned} M_W &= (166+166) = 332 \text{ in.-kips (from analysis).} \\ &= (325+325) = 650 \text{ in.-kips (magnified for } P\Delta\text{).} \end{aligned}$$

$$\begin{aligned} f_a &= P/A = 32.4/7.10 = 4.56 \text{ ksi} \\ f_b &= M/S_x = 650/17.7 = 36.7 \text{ ksi} \end{aligned}$$

The HSS 8x8x1/4 columns do not work.

A HSS 10x10x5/16 works.

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 effect the design of joist and joist girder systems. These include hanging loads, headers and openings, roof top units, joist reinforcement, spandrel systems, ponding, vibrations, 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. These crane systems are suspended from the joists and impart vertical, lateral and longitudinal loads onto the joist system. The vertical load to the joist 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, or skewing of the crane bridge. The longitudinal forces are due to the tractive force of the crane accelerating or decelerating or due to the crane bumping against the runway stops. Due to the dynamic nature of the loads, the design of crane support systems requires that consideration be given to fatigue and impact.

In underhung crane and monorail support situations, the joists are serving a dual function. They are supporting both a roof (or floor) and the crane system. The engineer must take proper precautions to ensure that these functions are compatible. For example, sidesway caused by longitudinal 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 deleterious effect on the joist to deck attachment.

Crane systems suspended from joists should be limited to the following characteristics:

Cranes that conform to the Crane Manufacturers Association of America, Inc. (CMAA) classifications A, B and C.³¹

Crane or monorail capacity of not more than five tons, and

pendant operated cranes only.

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 diaphragm bracing to resist the lateral and longitudinal crane thrusts in the plane of the roof for such crane systems.

Top running cranes may have their lateral thrusts resisted by rigid or braced frames. The design of rigid and braced frames and the proper specification of joists for these frames is dealt with in Chapter 4. The crane thrusts provide another load condition to be considered in the design of the frames, but no other special considerations need be considered.

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 capacity of the crane support system is affected by fatigue considerations. The CMAA service classifications (CMAA Specification #74, Revised 1987) have been established to describe the conditions of use for a crane in a particular situation. They 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 periods 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 2 to 5 lifts per hour, averaging 10 feet per lift.

2.4 CLASS C (MODERATE SERVICE)

This service covers cranes which may be used in machine shops or papermill 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."

If the engineer determines that fatigue considerations must be incorporated into the design of the joists, the engineer must select a joist with adequate capacity to meet the AISC fatigue provisions. The engineer must also inform the joist supplier that the joists are subject to fatigue loading and provide the appropriate allowable stress range for the critical general condition and situation. Two situations are of concern for crane loading on joists. These are: shear on the fillet welded diagonal to chord connections and tension in the base metal of the members adjacent to the welds. The AISC Specification requires that for Loading Condition 1 (this corresponds to CMAA classification A and B) the allowable shear stress range for fillet welds is limited to 15 ksi and the allowable stress range for base metal (thickness, $t < 1"$) adjacent to these welds be limited to 22 ksi. For Loading Condition 2 (CMAA classification C), the allowable stress range for fillet welds and the adjacent base metal ($t < 1"$) is 12 ksi and 13 ksi respectively. The AISC Specification requires that the configuration of welded end connections of dynamically loaded axial members be balanced about the center of gravity of the member.

If fatigue is not a consideration, the maximum allowable shear stress on E70 fillet welds is 21 ksi and the maximum allowable tension on a 50 ksi yield material chord or diagonal is 30 ksi. The AISC fatigue provisions do not limit the maximum allowable tensile stress in the member or the maximum shear stress in the weld. Only the stress range due to the fluctuating load is limited by the AISC fatigue provisions. This distinction allows the engineer to select a joist that has adequate capacity to support the total load and then verify that the stresses due to the crane loading do not exceed the fatigue allowable stresses.

Hangers and Bracing

Economical underhung crane runway beams can usually be designed to span 15 to 20 feet. Runway beams or monorails may be constructed from standard W or S shapes, or special patented shapes. Special joists should be designated at the support hangers of the runway beams. The manufacturer could be asked to mark the special joists with the use of a different colored primer. This would avoid 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 else concentrated load reinforcement must be provided or the manufacturer 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 should have a lateral brace to prevent the crane beam from swaying at the hanger location. A typical hanger and brace for this situation is illustrated in Figure 5.2.1. 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, their loads and locations must be supplied to the joist manufacturer.

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.

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 sup-

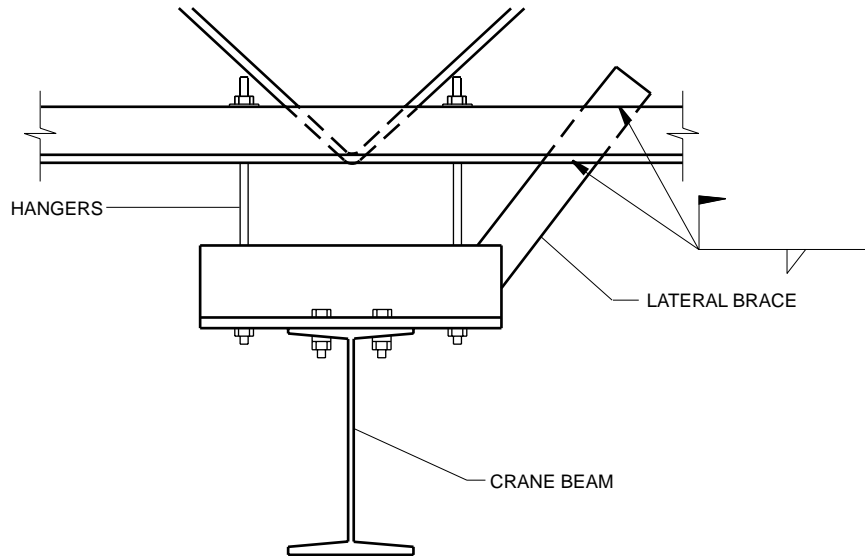


Fig. 5.2.1 Crane Runway Hanger

port system should be coordinated between the engineer 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. The typical hanger detail will require modification to also transfer the longitudinal load into the joist.

Clamp type hangers may be used to attach hangers to the bottom chord of joists. However, the engineer must design the clamps to avoid bending the outstanding legs of the joist chord. Clamps and hangers are not part of the components designed and supplied by the joist manufacturer.

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 5.2.3 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 Design of Cold-Formed Steel Structural Members.

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 series joists should be considered for this situation. In Example 5.2.1, a KCS series joist is selected to support a one ton underhung crane.

Example 5.2.1 KCS Series Joist/Crane Support

Choose a KCS series 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 ft.
- Joist spacing = 5 ft. o/c
- Dead load = 20 psf
- Live load = 30 psf
- Crane bridge length = 20 ft.
- Hangers at 10 ft. from each joist end.
- Wheel load = 2.5 kips/wheel
- Wheel spacing = 6'-0" (2 wheel/end truck)
- Crane bridge wt. = 2.8 kips
- Trolley wt. = 1.0 kips
- Crane is for standby use only. CMAA Class A (Less than 20,000 cycles.)
- Crane beam wt. = 30 plf.

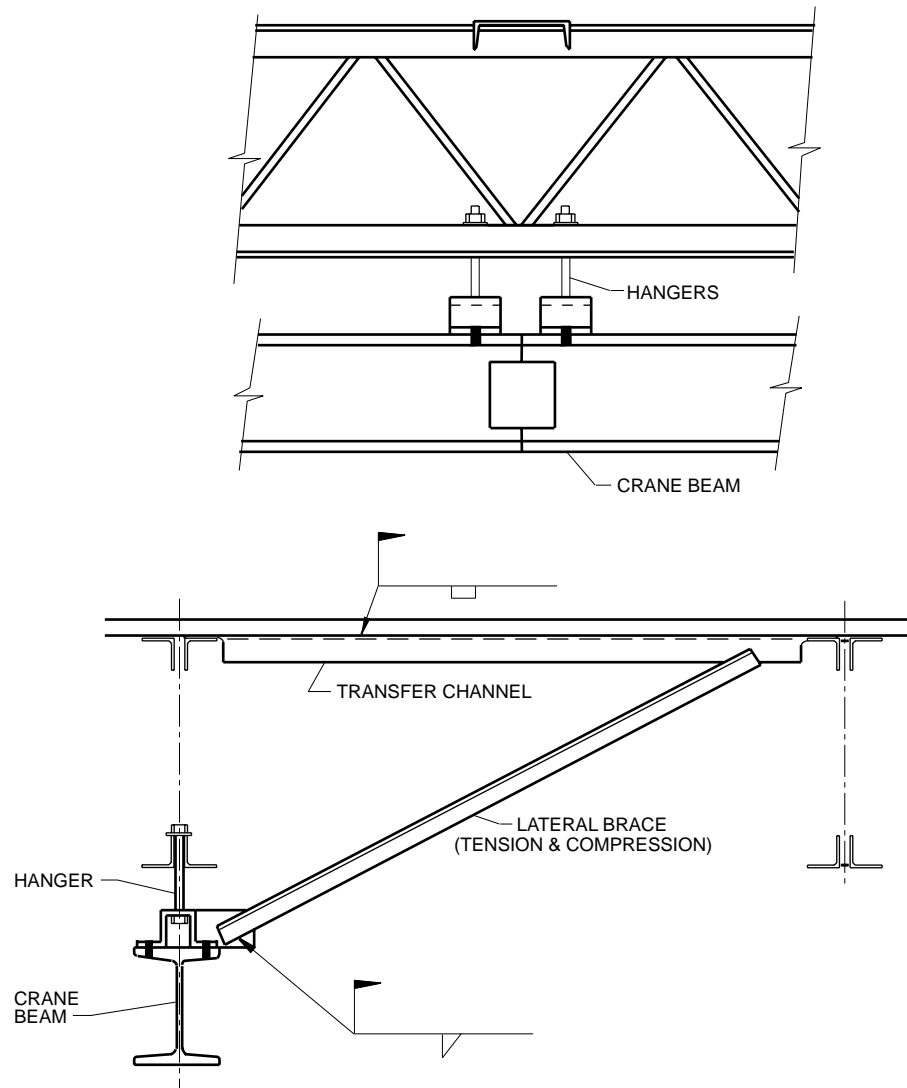


Fig. 5.2.2 Crane Runway Hanger

Solution: (ASD)

1. Roof load on the joist = $(20+30)(5) = 250 \text{ plf} = 0.25 \text{ klf}$.
2. Determine the maximum hanger reaction:
 $R_{\max} = 2.5(1+14/20) = 4.25 \text{ kips}$.
 Increase for impact and beam load:
 $R_{\max} = (4.25)(1.1) + (0.03)(20) = 5.3 \text{ kips}$
3. 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.
 $\text{Min. wheel load} = (2+2.8+1-2.5-2.5)/2 = 0.4 \text{ kips}$.

Determine the minimum hanger reaction:

$$R_{\min} = 0.4(1+14/20) = 0.68 \text{ kips}$$

Increase for impact and beam load.

$$R_{\min} = (0.68)(1.1) + (0.03)(20) = 1.3 \text{ kips}$$

The average reaction:

$$R_{\text{ave}} = (5.3+1.3)/2 = 3.3 \text{ kips}$$

4. Construct the load diagrams (Figs. 5.2.4 and 5.2.5) and solve for maximum shear and moment.

Based on the maximum hanger reaction:

$$R_L = (0.25)(40/2) + (1.3)(30/40) + (5.3)(10/40) = 7.3 \text{ kips}$$

$$R_R = (0.25)(40/2) + (5.3)(30/40) + (1.3)(10/40) = 9.3 \text{ kips}$$

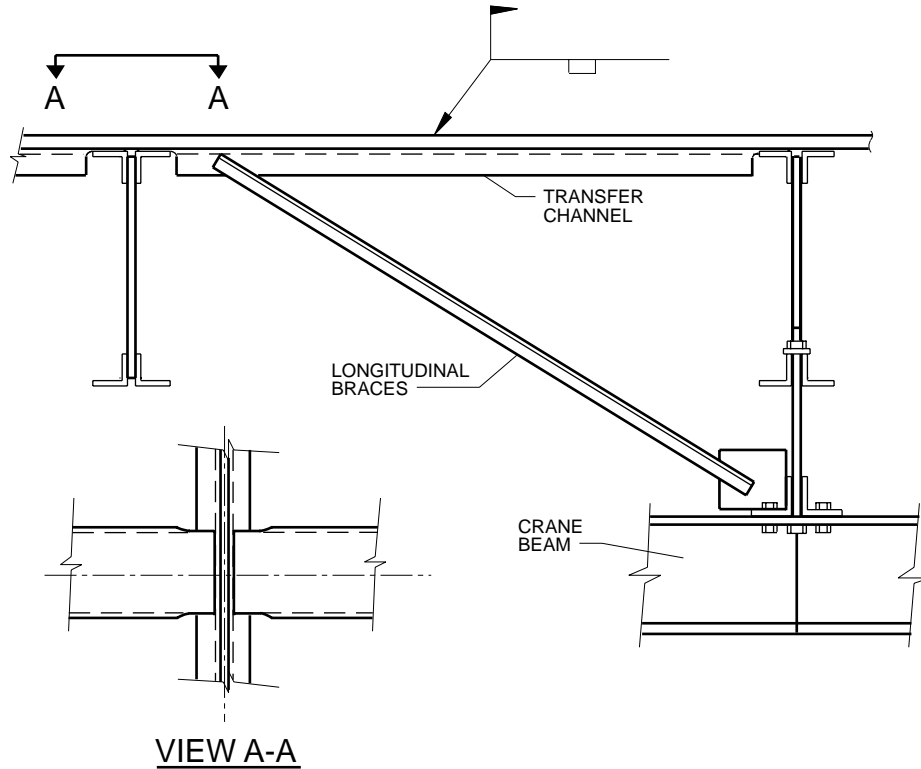


Fig. 5.2.3 Longitudinal Crane Brace

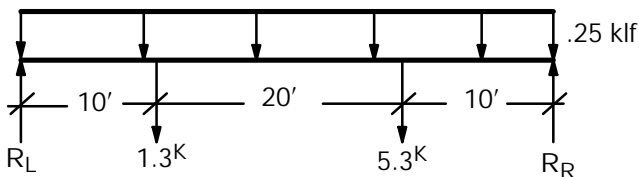


Fig. 5.2.4 Load Diagram

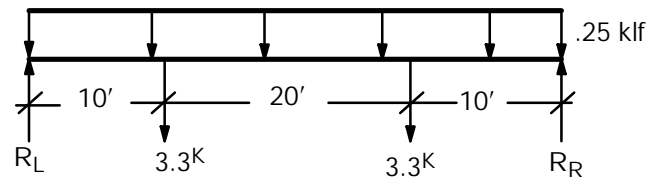


Fig. 5.2.5 Load Diagram

Using statics the point of zero shear is located 24 feet from the left support.

$$\begin{aligned}
 M_{\max} &= (7.3)(24) - (1.3)(14) - (0.25)(24)^2/2 \\
 M_{\max} &= 85 \text{ ft.-kips} \\
 &= 1020 \text{ in.-kips}
 \end{aligned}$$

Based on the average hanger reaction:

$$\begin{aligned}
 R_L = R_R &= (0.25)(40/2) + 3.3 = 8.3 \text{ kips} \\
 M_{\max} &= (8.25)(20) - 3.3(10) - (0.25)(20)^2/2 = 82 \text{ ft.-kips} \\
 &= 984 \text{ in.-kips.}
 \end{aligned}$$

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 series 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 capacity = 9200 lbs.

Moment capacity = 1576 inch-kips

The KCS series load tables can be used to select a joist with a sufficient 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 with regard to the crane support requirements. Generally a one percent grade is acceptable.

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 5.2.6a illustrates a beam to girder web connection.

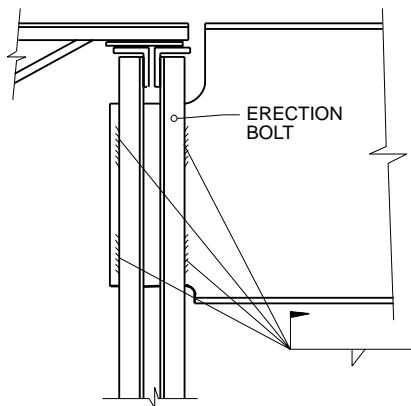


Fig. 5.2.6a Beam to Girder Connection

In place of the welded connection shown in Figure 5.2.6a the beam could be bolted to the vertical web member angles. 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.

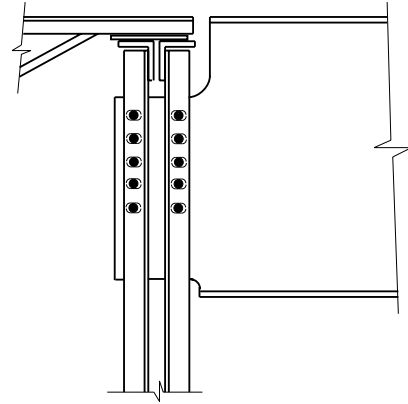


Fig. 5.2.6b Beam to Girder Connection

The beam to girder web connection should be designed to deliver the beam reaction to the center of the joist girder and to minimize the amount of beam end rotation induced into the girder. If practical, the end seat type of 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 as long as the unreinforced web section of the member can transfer the shear load. If this capacity 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" deep seat for the given beam and loading.

Given:

W16x31

Reaction, $R = 9$ kips.

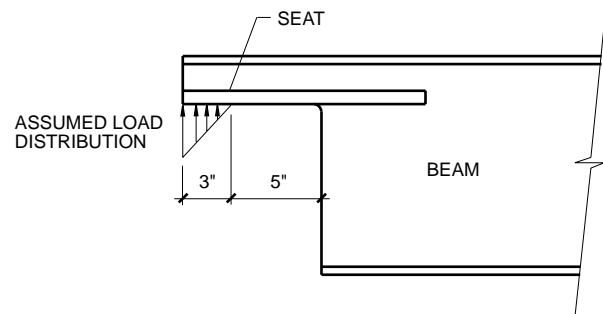


Fig. 5.2.7 Example 5.1.2

Solution:

1. Check the shear capacity of 2-1/2" deep section:

The maximum shear capacity = $V_R = F_v t_w h$

$$F_v = 0.4F_y = 14.4 \text{ ksi}$$

$$t_w = 0.275 \text{ inches; } h = 2.5 \text{ inches}$$

$$V_R = (14.4)(0.275)(2.5) = 9.9 \text{ kips}$$

$$9 \text{ kips} < 9.9 \text{ kips} \quad \text{o.k.}$$

2. Reinforce the section for bending:

$$M = RL$$

Based on the triangular stress distribution shown in Fig. 5.2.7 the reaction is located one inch from the end of the seat, thus $L = 7$ inches.

$$M = 9(7) = 63 \text{ in.-kips.}$$

Try adding a 4" wide plate to the T section. (See Fig. 5.2.8.)

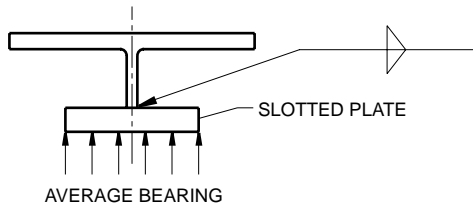


Fig. 5.2.8

Determine the required plate thickness. The thickness is based on cantilever bending of the plate.

$$\text{The average bearing stress} = (9)/[(4)(3)] = 0.75 \text{ ksi.}$$

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 1.7 inches. The required thickness is determined by solving the equation $M_x = S_x F_b$ for the plate thickness.

Where:

$$M_x = (0.75)(1.7)^2/2 = 1.08 \text{ in.-kips}$$

$$S_x = bt^2/6 = (1)(t_{\text{reqd}})^2/6$$

$$F_b = 0.75F_y = 27 \text{ ksi}$$

$$\therefore t_{\text{reqd}} = \sqrt{\frac{(1.08)(6)}{27}} = 0.49 \text{ inches}$$

Use a 1/2" plate.

The section properties for the composite cantilever section are:

$$A = 4.93 \text{ in.}^2$$

$$I_x = 4.67 \text{ in.}^4$$

$$S_{x \text{ top}} = 4.12 \text{ in.}^3$$

$$S_{x \text{ bottom}} = 3.42 \text{ in.}^3$$

$$y_{\text{bottom}} = 1.36 \text{ in.}$$

Check the bending stress in the cantilever.

$$f_b = \frac{M}{S_{\text{bottom}}} = \frac{63}{3.42} = 18.4 \text{ ksi}$$

$$F_b = 0.6F_y = 22 \text{ ksi}$$

$$f_b < F_b \quad \therefore \text{o.k.}$$

3. 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 = Moment of inertia

Q = The first moment of area of the added material.

$$v = \frac{(9)(1.36 - 0.25)(0.5)(4)}{4.67} = 4.3 \text{ kips/in.}$$

Using a 3/16" fillet weld near side and far side (ns/fs).

Weld stress = $v \div \text{weld area}$

$$f_v = 4.3/(0.707)(0.1875)(2) = 16.22 \text{ ksi}$$

$$f_v \leq 21 \text{ ksi} \quad \text{o.k.}$$

Evaluate the weld required to anchor the plate:

Plate force:

$$P = MQ/I$$

$$P = \frac{(63)(1.36 - 0.25)(0.5)(4)}{4.67} = 29.9 \text{ kips.}$$

Length of 3/16 fillet weld ns/fs required:

Allowable weld force per inch

$$= (0.707)(70)(0.3)(0.1875) = 2.8 \text{ kips/in. for one 3/16 in. fillet weld.}$$

$$L = 29.9/(2)(2.8) = 5.3 \text{ inches.}$$

\therefore Extend the plate 6" beyond the cope and weld with 3/16 fillet weld ns/fs. See Fig. 5.2.9 for final configuration.

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 with the conveyor supplier. 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:

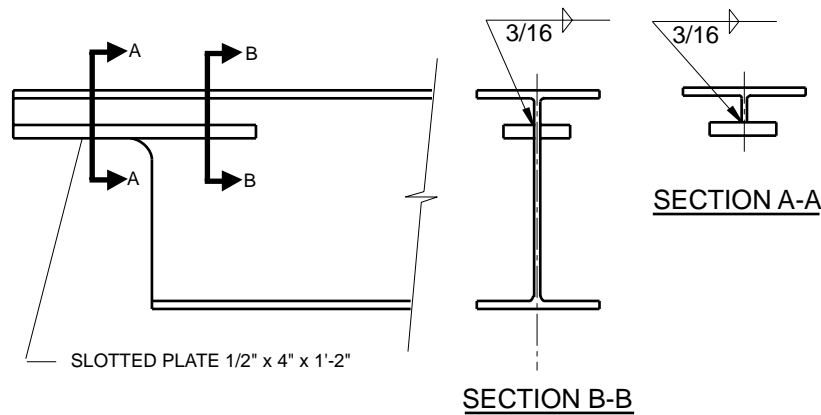


Fig. 5.2.9 Beam Seat

Continuous Belt Conveyors

Trolley Conveyors, and

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 engineer 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 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 with regard to 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 are dependent upon the drive system used, the type of support system used at the conveyor level and the configura-

tion 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 engineer 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 similar to the details required for underhung cranes. The details presented in the preceding chapter 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 engineer accommodate the load from the sprinkler systems and provide for the hanger attachment for the sprinkler sys-

tems. The support of process piping, small ducts, and cable trays requires similar considerations. The suppliers for these products should be consulted with regard to support requirements and loads. The support of sprinkler systems is rather a generic problem and standards are available to aid the engineer in designing support for these systems.

Load capacity for sprinkler systems is usually provided by the specifying engineer by using a uniform collateral load of sufficient magnitude to account for the loads induced by the piping system. This collateral load is added to the other loadings, and a joist of sufficient capacity 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 (NFPA) provides guidelines for the support of sprinkler systems in their publication NFPA 13 Standard for the Installation of Sprinkler Systems.³² 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, 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 5.2.1 lists typical weights and hanger reactions for sprinkler pipes.

Typical Sprinkler System Weights			
Pipe Diameter (inches)	Pipe & Water (pounds/ft.)	Hanger Load 5 ft. spacing (pounds)	Hanger Load 12 ft. spacing (pounds)
2	5	26	61
3	11	54	130
4	16	82	196
5	23	117	280
6	32	158	378
8	50	251	603
10	75	373	895
12	99	493	1184

Table 5.2.1 Typical Sprinkler System Weights

Hanger locations are not typically specified on the structural drawings. However, the engineer could specify hanger spacings at shorter intervals than required by NFPA requirements to reduce individual hanger reactions to the roof system. The specifying engineer 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 with regard to hanger locations and the permissible types of hangers.

Sprinkler systems in areas subject to earthquakes require sway bracing to resist lateral, longitudinal and vertical movement resulting from seismic loading. The sway bracing 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, Factory Mutual or other insurance criteria should be consulted if appropriate.

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.

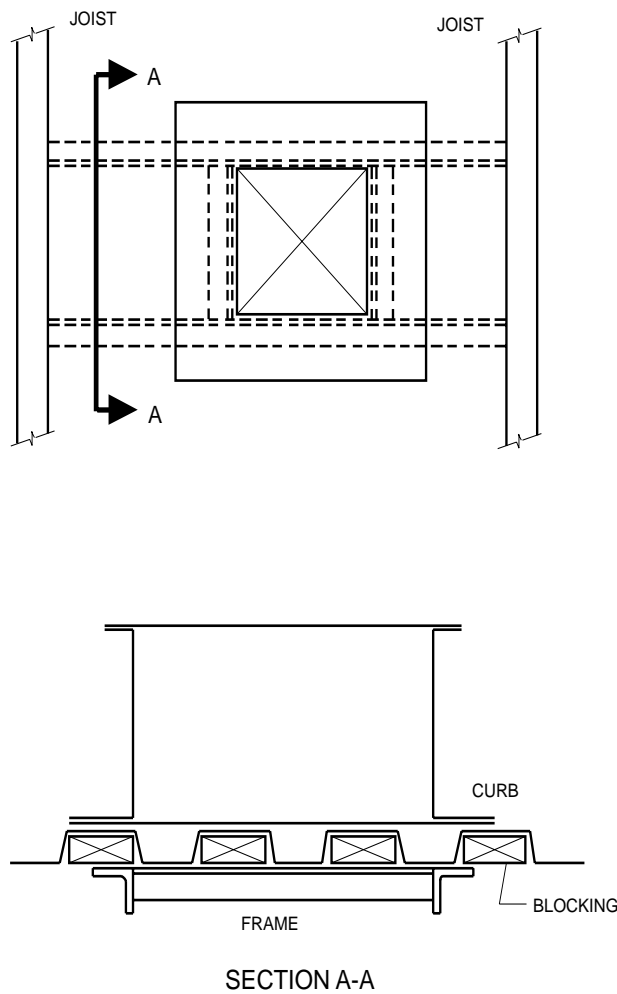


Fig. 5.3.1 Typical Roof Opening

The frames are usually constructed from hot rolled angles that have been welded into the required shapes. 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 considered to be 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 panel points or off panel points. If located at a panel point the effect on the joist 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 on 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, the manufacturer should be given the option of increasing the top chord size or adding 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 insure 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 as long as the bridging is properly anchored. Additional comments regarding bridging are included in the Section 5.10.

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

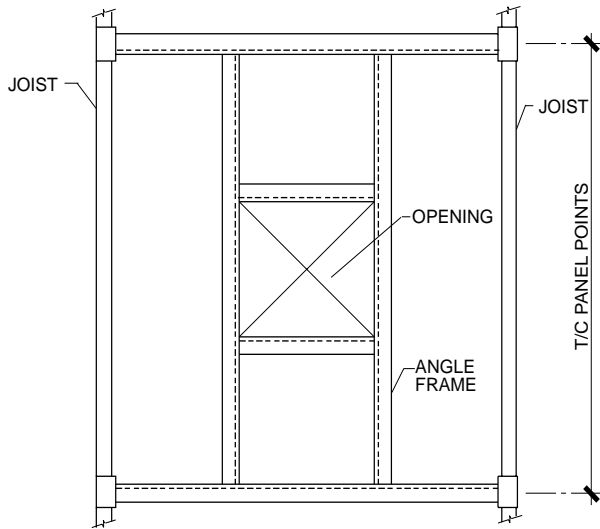


Fig. 5.3.2 Double Frame

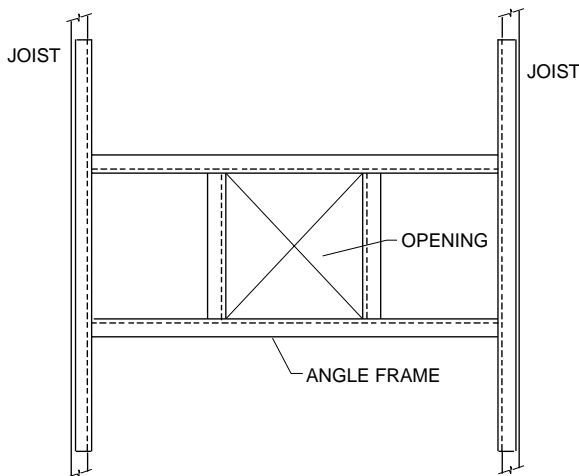


Fig. 5.3.3 Double Frame

roof top units 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. The frame would be 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 couple of 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 engineer must be particularly concerned with the load imposed by the unit on a specific joist. 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 engineer 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 varies considerably from one unit supplier to another, and the specifying engineer 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 engineer should be aware that it is not uncommon 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 engineer 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 engineer's decision about how to best provide capacity for the roof top units will depend on the size, number and similarity of the units. The engineer may provide capacity for large roof top units by specification of a special joist that can support the specific units 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 large number of relatively large units are randomly dispersed on a roof the engineer may prefer to use KCS series joists in lieu of specifying individual special joists. This may prove most effective if all of the special joists are just slightly different in loading. Replacing a large number of similar special joists with KCS series joists will avoid confusion, minimize the potential for errors, and maximize the flexibility of the system.

If a project is being fast tracked or if the specifying engineer is unable to procure definitive unit load and placement information, the engineer may choose to resort to the zoning method to provide capacity to support roof top units. In the zoning method, the engineer 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 possible loads placed within the zones.

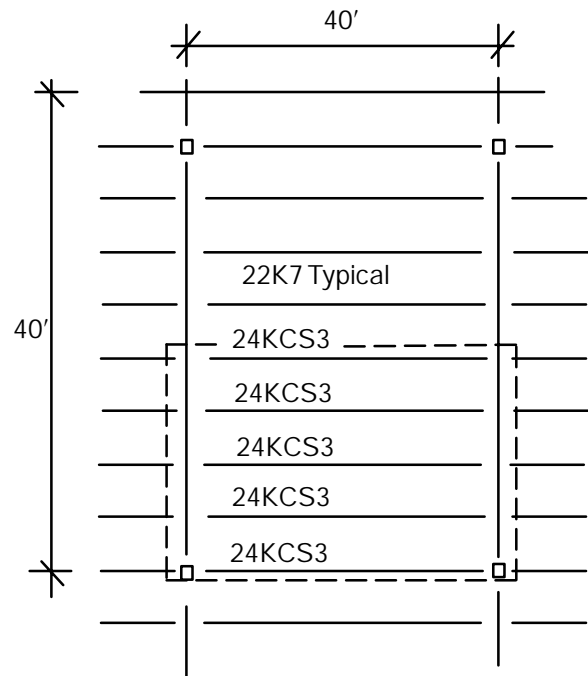
The zones should be located to provide the maximum area, while affecting the fewest number of joists and joist girders possible. Locating the zones near to columns will minimize the amount of flexural resistance required in the system. Zones should be located at relative high points of the roof to avoid 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 series 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 < 300 lbs.) may be designed to support these units at any location on the roof. The specifying engineer would determine the worst case loading of unit reaction to a joist, and use the procedure outlined in Chapter 6 to choose a standard joist to resist the unit load. These joists would be used throughout the roof. This procedure essentially provides a uniform collateral capacity throughout the roof. The engineer using this procedure will quickly be able to determine if joist size selection has been appreciably affected. If the cost of providing the uniform capacity throughout the roof is overly expensive, the alternatives of zoning the units or providing special joists at each unit should be investigated.

To support mechanical loads on a joist system, the specifying engineer must fill the gap between the joist manufacturer 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 capacity for small randomly placed units



Typical Roof Load
30 psf Live,
15 psf Dead

- - - - indicates perimeter of the 20' x 40' Roof Top Unit Zone. Locate Units Anywhere Within Zone. Maximum Allowable Reaction to a Given Joist is 1500 lbs.

Fig. 5.4.1 Roof Top Unit Zone

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 structural engineer. The joist designations and load diagrams will allow the joist manufacturer to coordinate with the engineer. The joist manufacturer 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 capacity 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 engineer 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 from the manufacturer. If this is not possible, a special 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, the joist manufacturer will add the diagonal in the shop if instructed to do so, and if the exact location of the diagonal is specified. The provision for concentrated loads is presented in Section 6.3.

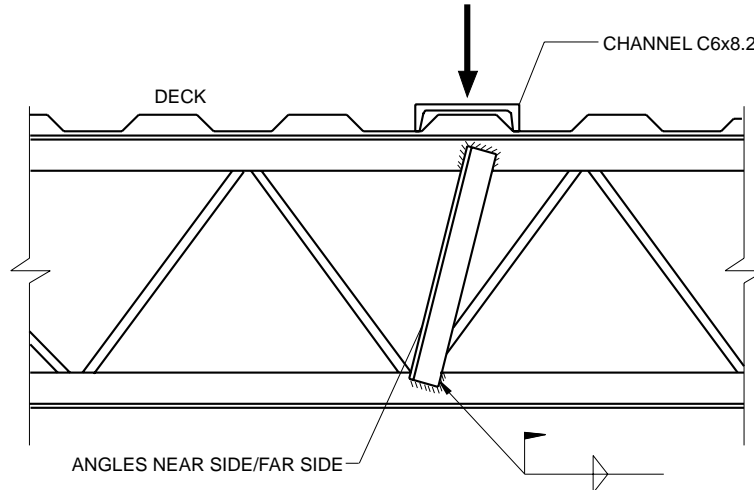


Fig. 5.4.2 Joist Reinforcement

The use of an inverted channel to carry the unit load between the joists is also illustrated in Figure 5.4.2.

The engineer 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 1999 BOCA model code and the 1997 UBC model code and the 2000 IBC (by referencing ASCE 7-98) all require that snow accumulation be considered for roof projections greater than or equal to fifteen feet in length.

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 midspan. Equation 5.4.2 defines the natural frequency of a joist loaded primarily by a uniform distributed load.

$$f = 188 / \sqrt{\Delta} \quad \text{Eq. 5.4.1}$$

$$f = 213 / \sqrt{\Delta} \quad \text{Eq. 5.4.2}$$

where;

f = the natural frequency of the joist in cycles per minute.

Δ = The joist deflection at midspan, 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 engineer 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. An attach-

ment to the building structure should be provided to avoid tearing the deck or roofing. It is usually desirable to attach the guy wires to a vertical standard several inches above the

level of the roof to avoid interference with the roofing materials. A possible guy wire connection detail is illustrated in Figure 5.4.3.

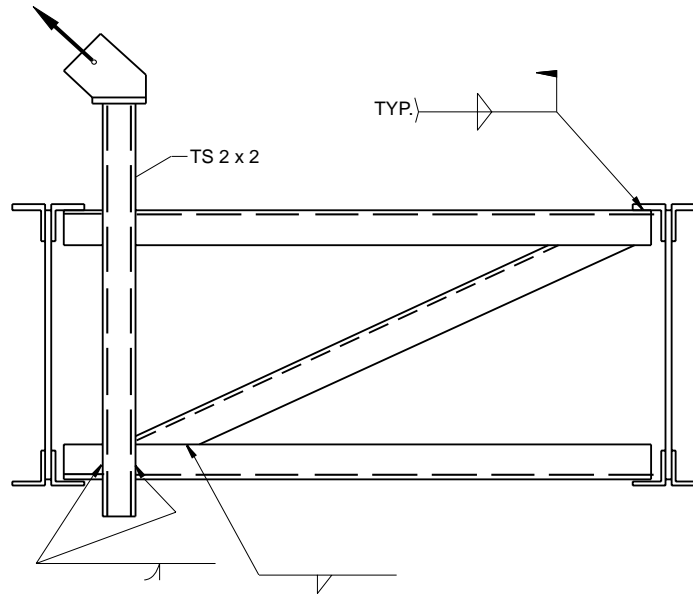


Fig. 5.4.3 Guy Wire Attachment

All of the force vectors and eccentricities of the guy wire attachment must be resolved into the support structure and the joists sized accordingly.

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 capacity 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 engineer can ask for assistance from Vulcraft for the reinforcement design.

The capacity 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 (J, H, K, LH, DLH) the web members be designed for a minimum percentage of the joist end reaction. The specifications also have requirements for the minimum capacity 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 a sixty year digest that is particularly useful for determining the capacity of older joists. The sixty year digest contains the specifications and load tables for all of the series of joists published between 1928 and 1988. 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, and the field welder must have room to weld the pieces in position. The project site visit also allows the evaluation of the present actual loading condition on the joists.

If the engineer 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. The manufacturer 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. The manufacturer could be contacted to determine if he has 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 engineer 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 capacity 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 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 Reference 23. One particular 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.

The relative stiffnesses of the joists and the distribution beam is defined by the characteristic parameter beta as defined in Equation 5.5.1.

$$\beta = \sqrt[4]{(K/S)/(4EI)} \quad \text{Eq. 5.5.1}$$

Where: K = The stiffness of the joist, kips/inch
S = The spacing of the joists
E = The modulus of elasticity for the beam
I = The moment of inertia of the beam

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 considered to 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

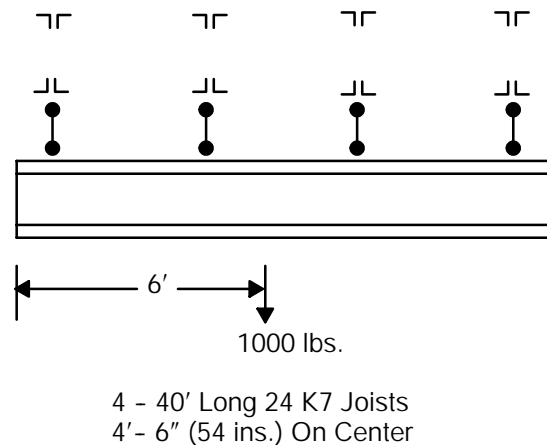


Fig. 5.5.1 Example 5.5.1

Size the beam shown in Fig. 5.5.1 to act rigidly and determine the reactions to the joists. Assume the load is located at mid-span on the joists.

Solution:

1. Determine the stiffness of the joists:

$$I_J = 26.767(W_{LL})(L^3)(10^{-6})$$

For a 24K7:

$$W_{LL} = 148 \text{ plf.}$$

$$I_J = 26.767 (148)(39.667)^3 10^{-6} = 247.25 \text{ in.}^4$$

Divide I_J by 1.15 to account for shear deflection.

$$I_{J \text{ eff}} = 247.25/1.15 = 215 \text{ in.}^4$$

$$K = P/\Delta = P/(PL^3/48EI) = 48EI/L^3$$

$$K = (48 \times 29,000 \times 215)/(39.667 \times 12)^3 = 2.78 \text{ k/in.}$$

2. Based on Eq. 5.5.1, determine the beam size necessary to distribute the load to the four joists:

Try a W16x26;

$$I_x = 301$$

$$\beta = \sqrt[4]{\frac{2.78/54}{4 \times 29,000 \times 301}} = 0.0062$$

Check if spacing $S < \pi/4\beta$:

$$S = 54" < \pi/4\beta = 127"$$

Check the length of beam $< 1/\beta$:

$$L = 13'-6" = 162.0 \text{ inches}$$

$$1/\beta = 1/0.0062 = 161.3 \text{ inches}$$

$$162 \approx 161.3$$

Therefore, the beam may be assumed to act as a rigid body in delivering load to the joists.

3. 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 Fig. 5.5.2.

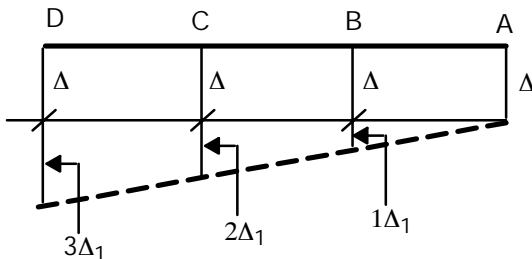


Fig. 5.5.2 Deflected Shape of the Beam

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.

$$\Sigma F_y = 0$$

$$K(\Delta + 3\Delta_1) + K(\Delta + 2\Delta_1) + K(\Delta + \Delta_1) + K\Delta - P = 0$$

$$4K\Delta + 6K\Delta_1 - P = 0$$

$$\Delta_1 = \frac{P - 4K\Delta}{6K}$$

Sum the moments about point A:

$$\Sigma M_A = 0$$

$$K(\Delta + \Delta_1)4.5 + K(\Delta + 2\Delta_1)9 + K(\Delta + 3\Delta_1)13.5 - P(7.5) = 0$$

Reducing:

$$27K\Delta + 63K\Delta_1 - P(7.5) = 0$$

Substituting for Δ_1 and solving:

$$27K\Delta + 63K(P/6K - 4K\Delta/6K) - 7.5P = 0$$

$$27K\Delta + 10.5P - 42K\Delta - 7.5P = 0$$

$$\Delta = -3.0P/(-15K) = P/15K = 1/[(5)(2.78)] = 0.0719 \text{ in.}$$

$$\Delta_1 = \frac{P(4)(2.78)(0.0719)}{6(2.78)}$$

Solving for the reactions:

$$R_A = 2.78(0.0719) = 0.20 \text{ kips.}$$

$$R_B = 2.78(0.0719 + 0.012) = 0.233 \text{ kips.}$$

$$R_C = 2.78(0.0719) + 2 \times 0.012 = 0.267 \text{ kips.}$$

$$R_D = 2.78(0.0719 + 3 \times 0.012) = 0.300 \text{ kips.}$$

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.

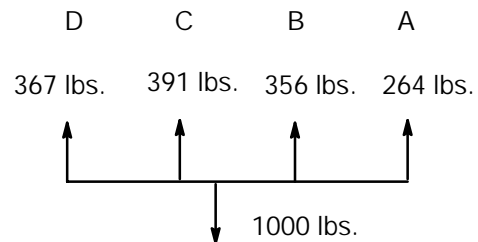


Fig. 5.5.3 Joist Reactions

4. The joist capacity 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 engineer 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 engineer may model the beam and joist assembly using a plane frame analysis program.

Adding New Joists

Once it has been determined that the existing system is inadequate, the engineer 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 a number of 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 capacity 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 capacity of the joist. The capacity of web diagonals determines the shear capacity of the joists. The capacity of the end seat determines the allowable end reaction of the joists.

The moment capacity of a given joist can be determined from the standard load tables. For H joists, the moment capacity is tabulated directly. For K, LH, and DLH joists, the moment capacity 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. The joist manufacturer 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 capacity 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. Sufficient 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, then the members should be shored while the member is being reinforced. The AWS D1.1: 2000 Structural Welding Code

states that “the engineer shall determine the extent to which a member will be permitted to carry loads while heating, welding, or thermal cutting is performed.” If the engineer 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 may 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 specified. The reinforcing members should be in line from workpoint to workpoint. 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 principles involved in the reinforcement of joists. Note that the reinforced joist has considerably more capacity than is required for the new loading condition. Given the unknowns associated with the reinforcement of joists, some conservatism seems justified. The added capacity 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.

Example 5.5.2 Joist Reinforcement

Given:

Reinforce a joist to support the uniform load and a new hanging load of 2000 lbs. at 10' from the left end.

From existing plans the joist is a 20K7.

Uniform applied load = 275 plf.

Length = 33 ft.

The top chord angle dimensions were field measured as 1-3/4 x 1-3/4x0.156 inches.

For the two angles:

$$A \approx 1.04 \text{ in.}^2$$

$$I_x \approx 0.306 \text{ in.}^4$$

The distance between panel points is 24 inches according to field measurements.

Solution:

1. Check the chord capacity:

Determine the end reactions:

$$R_L = 0.275 \times 33/2 + 2 \times 23/33 = 5.93 \text{ kips}$$

$$R_R = 0.275 \times 33/2 + 2 \times 10/33 = 5.14 \text{ kips}$$

Determine the maximum moment:

$$M_{\max} = R_R(x) - w(x^2/2)$$

$$\text{where } x = 5.14/0.275 = 18.70 \text{ ft.}$$

$$\begin{aligned} M_{\max} &= (5.14)(18.70) - (0.275)(18.7)^2/2 \\ &= 48.1 \text{ ft.-kips.} \\ &= 577 \text{ in.-kips.} \end{aligned}$$

Determine the allowable moment:

From the SJI load tables $W_{\text{allow}} = 309 \text{ plf.}$

$$M_R = 0.309 \times 33^2/8 = 42.1 \text{ ft.-kips.} = 505 \text{ in.-kips.}$$

$$M_x > M_R$$

\therefore Chord reinforcement is required.

It should be noted that if the joist designation is not known then the calculation of the chord capacity is more complex. The chord capacity 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.

2. Determine the amount of flexural reinforcement required:

Note: The joist will be shored prior to reinforcement, so prestress need not be considered.

The required additional chord force equals $M_{\max} - M_{\text{allow}}$ divided by the effective depth of the joist. The additional area can then be found by dividing the additional chord force by the allowable stress in the chord.

Additional chord force = $(577 - 505)/19 = 3.79$ kips.
Try adding (2) - 3/4" diameter rods to each chord ($F_y = 50$ ksi).

$$A = 0.442 \times 2 = 0.884 \text{ in}^2$$

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:

$$\text{Chord force} = M/d = 577/(20 - 0.5 - 0.5) = 30.37 \text{ kips}$$

$$\text{Chord stress } f_a = 30.37/(1.04 + 0.884) = 15.78 \text{ ksi}$$

Determine the allowable stress for the reinforced top chord:

$$I_x \text{ for the angles} = 0.306 \text{ in}^4 \text{ (given)}$$

$$A = 1.04 + 0.884 = 1.924 \text{ in}^2$$

$$r_x = (0.306/1.924)^{1/2} \approx 0.4 \text{ in.}$$

$$L/r_x = 24/0.4 = 60$$

$$F_a = 22.72 \text{ ksi.}$$

$$f_a < F_a \quad \text{o.k.}$$

3. Determine where $M_{\max} = M_{\text{allow}}$:

Locate the distance (x) from the right end:

$$42.1 = 5.14x - 0.275x^2/2$$

$$x^2 - 37.38x + 306.1 = 0$$

$$x = 12.12 \text{ ft.}$$

Locate the distance (x) from the left end:

$$42.1 = 5.93x - 0.275x^2/2 \quad (\text{for } x < 10')$$

$$x^2 - 43.13x + 306.1 = 0$$

$$x = 8.95 \text{ ft.}$$

The chord reinforcement must be fully developed at these locations.

4. 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 3/4 inch rod the allowable weld force equals 2.46 kips/in.

Weld at ends:

$$\text{Max rod force} = F_t A = 30 \text{ ksi} \times 0.442 = 13.3 \text{ kips.}$$

$$\text{Length of flare bevel weld req'd} = 13.3/2.46$$

$$= 5.4 \text{ inches.}$$

∴ Provide 6" of flare bevel weld at the ends of each rod.

Weld along the length of the member.

$$I_{\text{joist}} = 26.767(W_{LL})(L^3)(10^{-6})$$

$$I_{\text{joist}} = 26.767(181)(33 - 0.33)^3(10^{-6}) = 169 \text{ in}^4$$

$$I_{\text{eff}} = 169/1.15 = 147 \text{ in}^4$$

Required shear flow = v .

$$v = VQ/I$$

$$v = (5.9)(0.442 \times 9.5)/147 = 0.17 \text{ kip/inch/rod}$$

The shear flow is introduced into the chord at each panel point. Using the panel point spacing of 24 inches, determine the length of flare bevel weld required at each panel point.

$$\text{Length of weld req'd} = (0.17)(24)/2.46 = 1.65 \text{ inches.}$$

∴ Provide 2 inches of flare bevel weld at each panel point. Also provide 2 inches of weld at the midpoint of each panel point to control buckling of the rods.

For the compression chord, check buckling of the rod between welds: Radius of gyration for 3/4 in. round rod = 0.1875

$$L/r = 12/0.1875 = 64; F_a = 22.02 \text{ ksi.}$$

$$f_a \leq F_a \quad \text{o.k.}$$

5. Check the web capacity:

$$V_R = wL/2$$

$$V_R = 0.309 \times 33/2 = 5.1 \text{ kips}$$

$$\text{Minimum shear/SJI} = V_R/4$$

$$V_R/4 = 1.27 \text{ kips}$$

Construct the allowable and actual shear diagram.

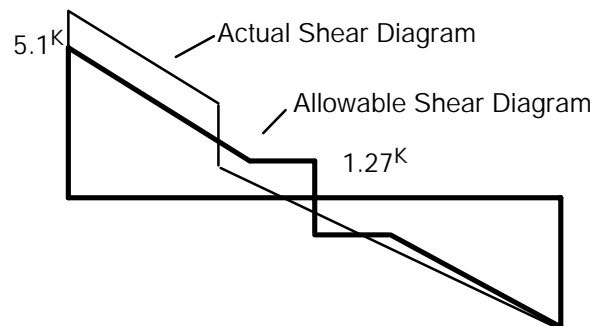


Fig. 5.5.5 Shear Diagrams

From inspection of the shear diagram, it can be seen that the diagonals from the left end to the load require reinforcement.

6. Determine web reinforcement:

Field measurements have provided the panel point locations as shown in Figure 5.5.6.

Conservatively add diagonal angles along the web

members to carry the entire shear force to each side of the chords.
Using A36 reinforcing angles.

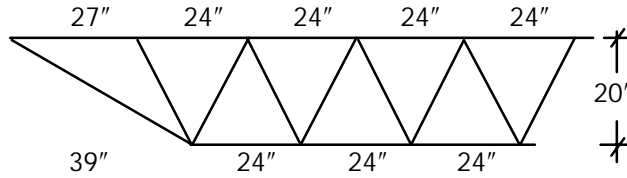


Fig. 5.5.6 Joist Measurements

stress equals $2.8/(0.707 \times 0.1875 \times 5) = 4.2 \text{ ksi} < 21 \text{ ksi}$. 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 5.5.7, 5.5.8 and 5.5.9.

From statics, the forces in the diagonals may be determined.

Tension in the end bar; $T = 12.8 \text{ kips}$.

Maximum compression rod; $C = 5.96 \text{ kips}$.

Tension diagonal:

$$A_{\text{req'd}} = 12.8 / 22 = 0.58 \text{ in.}^2$$

Use 2 L 1-1/4 x 1-1/4 x 3/16, $A = 0.868 \text{ in.}^2$

Length of 3/16" weld required = $12.8 / 2.78$

= 4.6 inches

Use 3" of 3/16" fillet on each angle.

Compression diagonal:

Try 2 L 1-1/4 x 1-1/4 x 3/16

$A = 0.868$

$r_x = 0.377 \text{ in.}^2$ (for two angles, battens are required between the angles)

$$L/r_x = 38 / 0.377 = 101 \rightarrow F_a = 12.85 \text{ ksi,}$$

$$f_a = 5.96 / 0.868 = 6.87 \text{ ksi} \quad \text{o.k.}$$

7. 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 5.5.9)

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, $H = 11.5 / 2 = 5.75 \text{ kips}$ and a vertical force $V = 5.6 / 2 = 2.8 \text{ kips}$.

Assume that the horizontal force component is resisted by the weld between the plate and the top chord. The stress in the 3/16 inch x 5 inch long weld equals $5.75 / (0.707 \times 0.1875 \times 5) = 8.68 \text{ ksi} < 21 \text{ ksi}$.

Assume that the vertical force component is resisted by the weld between the plate and the seat angle. The weld

SPECIAL TOPICS

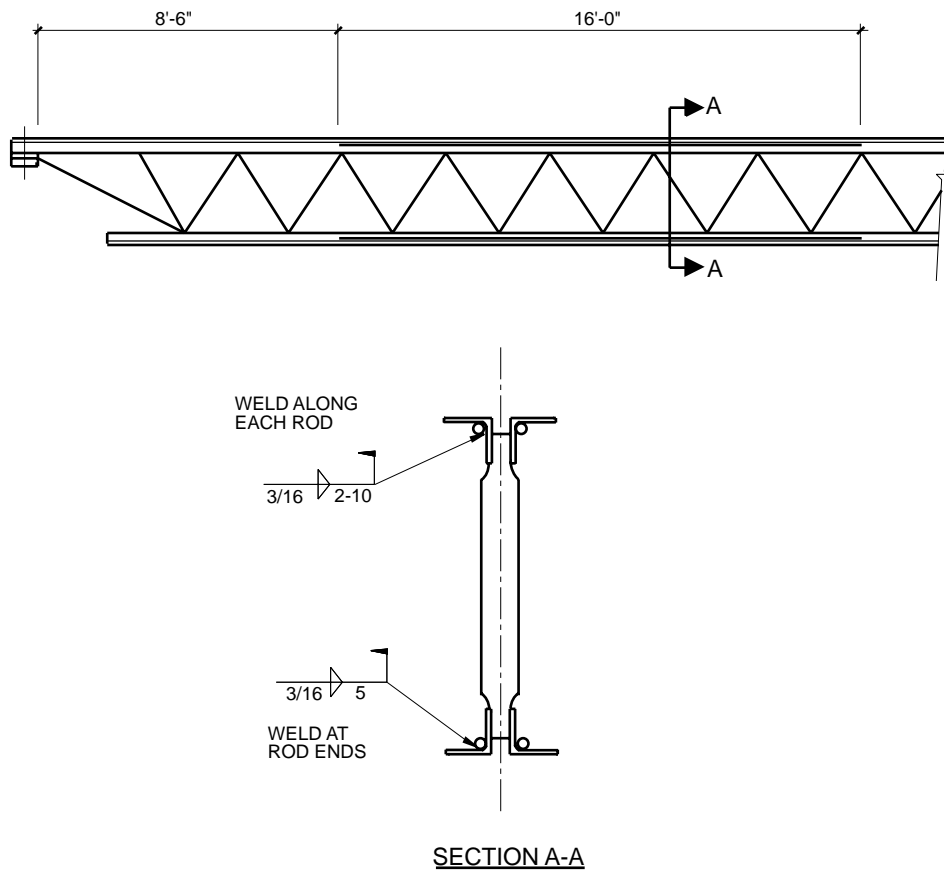


Fig. 5.5.7 Joist Chord Reinforcement

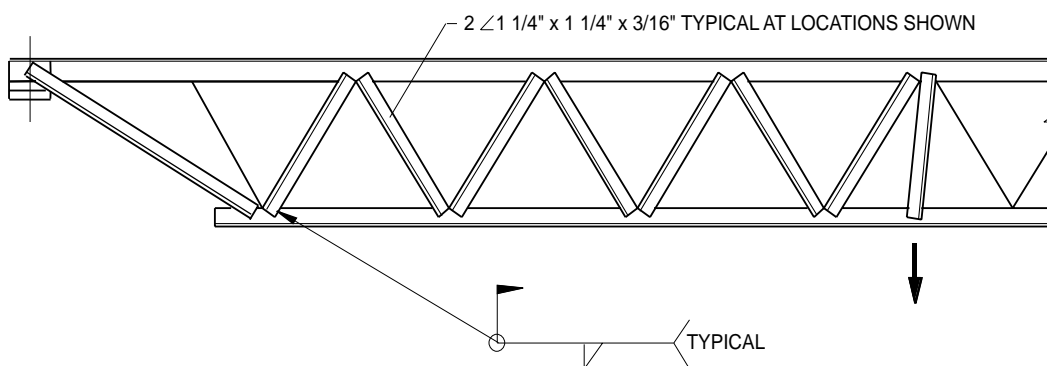


Fig. 5.5.8 Joist Diagonal Reinforcement

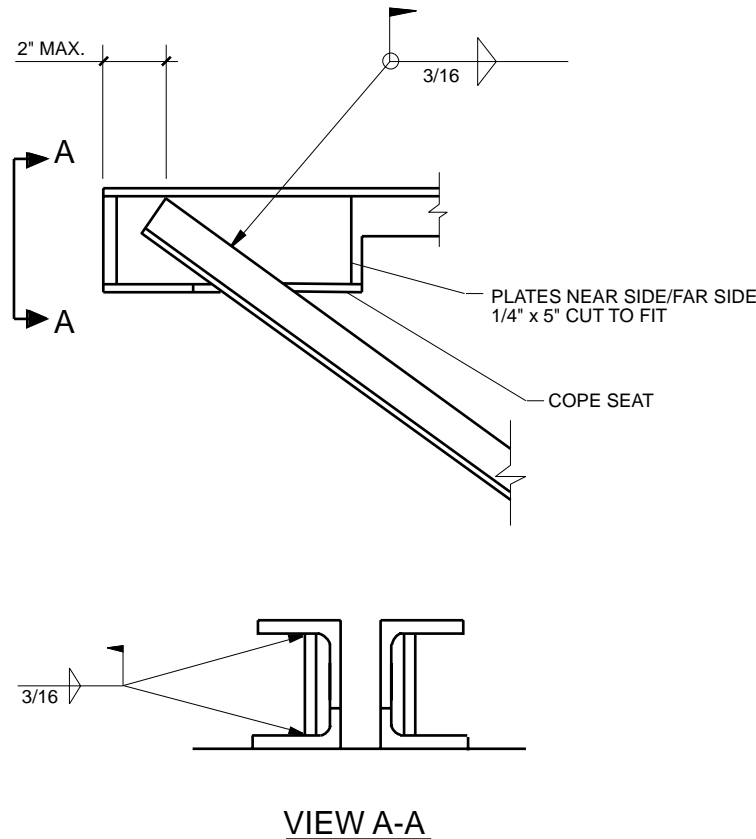


Fig. 5.5.9 Joist End Seat Reinforcement

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. The vast majority of 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 or not the occupants are annoyed by floor vibrations. This criterion is, by definition rather nebulous, and the design of a floor support system that meets this requirement must be based on the sound judgment of a qualified engineer using researched and documented design techniques.

In general, floor vibration considerations can be grouped into two categories. These are vibrations due to rhythmic or repeating excitation, and vibrations due to transient vibrations.

The human perception of transient floor vibrations relates to the frequency, amplitude, and duration of the vibra-

tion 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 given floor system such as ducts, ceilings, partitions, furnishings and even people.

The Steel Joist Institute has sponsored considerable research with regard to the perceptibility of transient floor vibrations to human occupants on steel joist floor systems. This research was conducted at the University of Kansas and is summarized in SJI Technical Digest No. 5, "Vibra-

tion of Steel Joist and Concrete Slab Floors”. The digest discusses an analysis procedure for the design of steel joist floor systems. Based on the research, the natural frequency of a given joist in a steel joist and concrete deck system is based on composite action of the joists and supported concrete deck. The motion imparted by a human footfall is related to a heel drop impact. The deck serves to spread this motion perpendicular to the joists so that an effective number of composite joists may be considered in determining the amplitude. After the amplitude and frequency have been determined, these parameters are related to human perception of vibrations due to transitory motion. The current research with regard to design of steel framed floor systems subjected to transient vibrations is contained in the AISC Design Guide No. 11 “Floor Vibrations Due to Human Activity”. The recommendations of the AISC Design Guide are generally consistent with the theory given in the previous SJI Technical Report. The following refinements are provided in the AISC Design Guide.

The current research with regard to design of steel framed floor systems subjected to transient vibrations is contained in the AISC Design Guide No. 11 “Floor Vibrations Due to Human Activity”. The recommendations of the AISC Design Guide are generally consistent with the theory given in the previous SJI Technical report. The following refinements are provided in the AISC Design Guide.

A refined method of determining the natural frequency of the floor system provided.

Recommendations are provided for the amount of damping that could reasonably be anticipated for different building types.

Recommendations are provided for an acceptability criterion based on a maximum acceptable acceleration for different occupancies.

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 on 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 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 also improves the behavior of the floor system. The thicker 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 an 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 “cook-book” 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 engineer 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 engineer making the required decisions with regard to span direction, member size, spacing and floor thickness.

In addition to human perceptibility considerations, it is possible that floor vibrations may effect 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 the AISC Design Guide. 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 detail 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 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 capacity, 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.

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, viz. S1 through S12 which are extended top chords and R1 through R12 which are extended ends. 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 by two. 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 2-1/2" will be required. It is the requirement of the Joist Specification and the SJI Code of Practice that the loads on extended ends be given on the construction documents by the specifying engineer. 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.

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" which gives design thicknesses for various combinations of slab depth and overhang. This table does not provide live load capacity, 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 moment 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 insure 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. Manufacturer's standard outriggers are available and, of course, custom designed outriggers can be used. The use of outriggers necessitates dropping the joist on the exterior column line so the outriggers can cantilever over them 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. Manufacturer'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.

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 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 in Reference 14 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 5/8".
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 3/8".
4. Deflection due to live loads: span over 360 to a maximum of 1/4" to 1/2" 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 co-ordinate 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 "Recommended Code of Standard Practice for Steel Joists and Joist Girders" states in paragraph 5.5 that "Due Consideration by the specifying engineer or architect shall be given to live loads due to: 1. Ponded rain water" In addition each of the joist/joist girder specifications gives the following requirement:

"Unless a roof surface is provided with sufficient slope towards points of free drainage, or adequate individual drains to prevent the accumulation of rain water, the roof system shall be investigated to assure stability un-

der ponding conditions in accordance with Section K2 of the AISC Specifications (ASD). The ponding investigation shall be performed by the specifying engineer or architect.” For further reference, refer to Steel Joist Institute Technical Digest No 3., “Structural Design of Steel Joist Roofs to Resist Ponding.”

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, and
2. The source of load (water) is uncontrollable, i.e. nature.

When water can accumulate on a structural system due to empondment 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.

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 insure that convergency 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 AISC Specification Commentary (1989) in Section K2 gives limits on framing stiffness which provide a stable roof system. They are:

$$(\text{Eq. K2-1}) \quad C_p + 0.9C_s \leq .25$$

$$(\text{Eq. K2-2}) \quad I_d \geq 25(S)^4 10^{-6}$$

where

$$C_p = (32L_s L_p^4)/(10^7 I_p)$$

$$C_s = (32S L_s^4)/(10^7 I_s)$$

$$L_p = \text{length of primary members, ft.}$$

$$L_s = \text{length of secondary members, ft.}$$

$$S = \text{spacing of secondary members, ft.}$$

$$I_p = \text{moment of inertia of primary members, in}^4$$

$$I_s = \text{moment of inertia of secondary members, in}^4$$

$$I_d = \text{moment of inertia of the steel deck, in}^4 \text{ per foot}$$

The Specification also states that “Total bending stress due to dead loads, gravity live loads (if any) and ponding shall not exceed 0.80 F_y for primary and secondary members. Stresses due to wind or seismic forces need not be included in a ponding analysis.” The Commentary to the Specification goes on to elaborate on this total bending stress requirement. It should be noted that the checking of bending stresses is not required if the stiffness controls of equations K2-1 and K2-2 are met. Equation K2-2 is met in most buildings without the need to increase the stiffness of the deck. Equation K2-1 in many cases would require stiffer elements than would be required by loading and the SJI criteria that live load deflection be limited to a maximum of span over 240.

In the majority of cases, roofs which do not meet equation K2-2 can be shown to conform to the bending stress limit of 0.80 F_y. The procedure given in “Commentary” is based on:

1. A calculation of the deflection due to the accumulation of water in the deflected shape of the primary and secondary members at the initiation of ponding. These deflected shapes are taken to be half sine waves which is sufficiently accurate for this calculation.
2. A factor of safety of 1.25 for stresses due to ponding, which results in an allowable stress of 0.8F_y.
3. Behavior of the members is in the elastic range so that deflection is directly proportional to stress.
4. Stress due to ponding is limited to 0.80F_y minus the stress in the members at the initiation of ponding.

Thus, the method uses four variables:

U_p, the stress index for the primary member.

U_s, the stress index for the secondary member.

C_p, the stiffness index for the primary member.

C_s, the stiffness index for the secondary member.

C_p and C_s are as given in the specification in Section K-2.

U_p and U_s are given as
 $(0.8 F_y - f_o)/f_o$
 where f_o is the bending stress in the member (primary or secondary) at the initiation of ponding.

The Commentary presents two figures C-K2.1 and C-K2.2. Figure C-K2.1 is used to find a maximum C_p when U_p and C_s are given. Figure C-K2.2 is used to find a maximum C_s when U_s and C_p are given. The procedure in the Commentary is thus a checking procedure since trial sections must be chosen to establish C_p , C_s , U_p and U_s . Figures C-K2.1 and C-K2.2 are graphs which represent combinations of stress and stiffness which control the increment of load (stress) and deflection at the initiation of ponding. If one studies the relationships in these figures, the following can be noted. Required stiffness is inversely related to initial stress. If the stress index associated with values of C_p and C_s which meet equation K-2.1 is plotted, one can see that the stress index is very low, indicating that f_o is very near $0.6F_y$. This is logical since the system is so rigid that the ponded accumulation is negligible. As one moves beyond C_p and C_s , which meet equation K-2.1, it can be seen that the term $(0.8F_y - f_o)$ must increase to provide for the reduction in stiffness, e.g. the increase in C_p and/or C_s . Thus it can be seen that the accurate calculation of f_o is the essential element in using this procedure. The Commentary states that f_o is "the computed bending stress in the member due to the supported loading, neglecting the ponded effect...". The matching text from the specification would be "due to dead loads [and] gravity live loads (if any) ...". The calculations for the increment of ponded water are a function of the initial deflection and stiffness of the primary and secondary members. The initial deflection and the initial stress are the result of the "initial loads", and are thus logically those present at the "initiation of ponding". This means that the "initial loads" may be and will probably be different from the design loads.

The initial loads would include all appropriate dead and collateral loads, e.g.

1. Weight of structural system.
2. Weight of roofing and insulation system.
3. Weight of interior finishes.
4. Weight of mechanical and electrical systems.
5. Weight of roof top mechanical systems.

The initial loads would also include some or all of the live load. The requirement of the specification and the commentary point to the fact that the live load must actually be present at the initiation of ponding. Thus the appropriate portion of design live load is not necessarily 100% of the design live load. The amount of live load

which is to be used is up to the judgment of the engineer. The most significant factor in northern regions of the country will be a prediction as to the amount of snow which will be present at the initiation of ponding. A significant factor in all regions will be a judgment as to the amount of water on the roof at the initiation of ponding. Also consideration must be given to the combination of snow and water where applicable. A reading of the Specification and Commentary demonstrates that the loading at the initiation of ponding would not include the water which produces the stresses due to ponding, but it would include water trapped on the roof because the roof had not been "provided with sufficient slope towards points of free drainage or adequate individual drains to prevent the accumulation of rain water...". Also, it should be noted that ASCE 7-98 (Section 8.3) states that roofs with at least 1/4" per foot need not be investigated for ponding if water cannot be empounded. Thus, the live load at the initiation of ponding could include water trapped by plugged internal roof drains. The depth of water would be that from the top of roofing to the height of the free roof edge or the invert elevation of roof relief scuppers. Building codes such as the International Building Code make the provision for entrapped water a load case.

The 2000 IBC Code references ASCE 7-98 (Section 8.3). ASCE 7-98 requires that "each portion of a roof shall be designed to sustain the load of all rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow." Previous model codes included similar requirements.

The use of the weight of trapped or empounded water is recommended in SJI Technical Digest No. 3, "Structural Design of Steel Joist Roofs to Resist Ponding Loads". This reference also gives an approach for accounting for the potential for snow and water in combination. It recommends that "where ice and snow are the principal source of roof live load" 50% of the design live load be used up to 30 psf live load, and 100% of the design live load when the design live load is 40 psf and greater. Presumably the percentage could be interpreted as varying linearly for loads between 30 and 40 psf. When these values are used to account of rain and snow, it is not necessary to add in the weight of potential trapped water described above unless the weight of empounded water would be greater than the reduced design live load. The 2000 IBC requires that roofs with a slope of less than 1/2 in 12 be designed for rain on snow in accordance with ASCE 7-98 (Section 1608.3.4).

It should be noted that the foregoing described two-way roof framing systems. There is a separate case where joists bear directly on walls. This case eliminates the primary member deflection and the AISC Commentary pro-

cedure can be used by reference to figures C-K2.1 and C-K2.2 for which C_s is calculated using the deck properties and C_p is calculated using the joist properties. Also the SJI Technical Digest No. 3 gives a procedure for accounting for a reduction in the accumulated water weight due to camber. Logic suggests that concept could also be applied to the two-way system. Neither AISC nor SJI recognizes the deflected geometry of a continuous primary framing system. All of the deflection and load calculations of both procedures are based on the half-sine wave shape of the deflected element. This shape is conservative with a continuous primary member, because it overestimates the volume in the deflected compound curve.

It can be seen from the foregoing discussion that:

1. Ponding is an important concern in roof design.
2. Using the stiffness criteria of the code can produce unnecessarily conservative designs.
3. Use of the design approach presented in the AISC Commentary is recommended.
4. Determination of the appropriate loading in the calculation of initial stress is absolutely critical for the method to produce an accurate result. The reader is encouraged to examine the AISC Specification and Commentary and SJI Technical Digest No. 3 in detail.

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".³³ The purpose of fire tests is to establish the relative performance of different assemblies under identical laboratory test conditions. The majority of 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 particular project that all of 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 or 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 structural engineer 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 catalog 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 a number of special situations and problems that confront the designer of joist and joist girder buildings. This section offers a few brief comments regarding each.

Camber

The Steel Joist Institute Specification tabulates the camber for K, LH and DLH joists. In addition, camber is given in the Vulcraft catalog for the SLH joists and the KCS series. Camber is optional with the manufacturer for K series joists. If camber is required on K joists, it should be specified in the project specifications. Vulcraft provides standard SJI camber on K series joists. Vulcraft does not typically camber joists where the top chord is pitched two inches or more per foot, because deflections are minimal for such joists.

Erection and detailing problems can occur with LH, DLH and super long span 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 endwall, 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 endwall, the deck can often be pushed down flat on an endwall 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.

In addition the design engineer must pay attention to different length joists that are parallel to one another. These joists will have different 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 endwall, 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 endwall 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. 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 absolutely necessary for some reason to do so.

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 sub-purlin system can be used to support the standing seam roof. The sub-purlin is generally a light gage hat section spaced 5'-0" on center. Any economical joist spacing may be used, but the sub-purlin system must be designed to span the distance between the joists. The reactions from the sub-purlins and their locations on the joists must be specified by the engineer of record on the structural drawings to the manufacturer. If the panel points on the joists cannot be spaced so that the sub-purlin reactions are applied at panel points, the top chord of the joists must be designed for the concentrated loads delivered by the sub-purlins. The designer should be careful in using this system if UL uplift requirements have been specified. The sub-purlin 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. If a significant tilt exists, i.e. greater than 2 on 12, consideration should be given to the down hill component of loads. This load component can be resisted either by designing the roof diaphragm system to resist the loads or by providing a special design of the bridging system to trans-

fer the loads from the joist chords to the bridging, and then to some other part of the structure. On double sloped buildings, the bridging can be designed to be continuous across the ridge of the structure in order to provide a "self-balancing" system for the downslope load components. This mechanism works well unless unsymmetrical loading is required on the structure. The specifying engineer is responsible for the special bridging or diaphragm design.

Extended Ends

Two types of extended ends are available with K series joists. The first is the top chord extension of "S" type. This is the simplest type and the most economical. The second type is when seat angles are extended, which is designated an "R" type. When the "R" type is used, a larger moment capacity exists because of the I-beam shape of the extension. With the "R" type, the reinforcement must be extended back into the joist so that the cantilever moment can be resisted by a force couple acting at the seat and the first interior web member. The current edition of the SJI Specification and Vulcraft's catalog provide uniform load tables and the moment of inertia and section modulus for both the "S" and "R" extensions. These tables allow the engineer to determine the extension requirements. Prior to publishing this data, a load diagram was required to define those cases in which a concentrated load as well as a uniform load was applied to the extension. The designer may still show a load diagram for extensions if he desires. If the extension type is not specified, or if a load diagram is not provided, the manufacturer will provide an extension that meets the uniform load requirements for the specified joist.

Folding Partitions

The designer 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 partition load more than one inch, the partition will become inoperable. Thus, a deflection of less than one inch must be specified for these joists. 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, a high shear 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 the manufacturer so that the joists can be properly designed. This is another situation where the Vulcraft KCS series joist can be specified.

Seat Depth Changes

On occasion, engineers may forget that there is a difference in seat heights between the various types of joists, i.e. K and LH, 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 seats so that the top of the chords are 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 the manufacturer 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 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. The joist manufacturer should be aware of the intent so that he 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") 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 the joist manufacturer 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 designer until the members are

designed by the manufacturer. The designer may call for a 5/16" fillet weld on the edge of a joist girder chord and the joist girder chord supplied is only 3/16 inches thick, thus the weld cannot be made. The designer should attempt to use 3/16" fillet welds whenever possible to eliminate this potential problem. The designer can contact the manufacturer early in the design process if exact sizes need to be known, so that special weld requirements can be determined. The manufacturer can then provide oversized members to meet specified weld requirements. If the specified welds are not actually required, this can add significantly to the price of the project. This is an area where good communication between the engineer/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 off-set from the existing joists. Again, the manufacturer must be informed of the designers intent so that the joist girder 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 at a later date to allow room for the new joists to be placed on the perimeter member. The manufacturer must also be aware of this situation so that the joist seat can be designed for the 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 designers 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. The authors have found that the cost of placing holes in the top chord of the joist girder is an expensive item. Also the size of the top chord of the

girder must often be increased to compensate for the holes. 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.

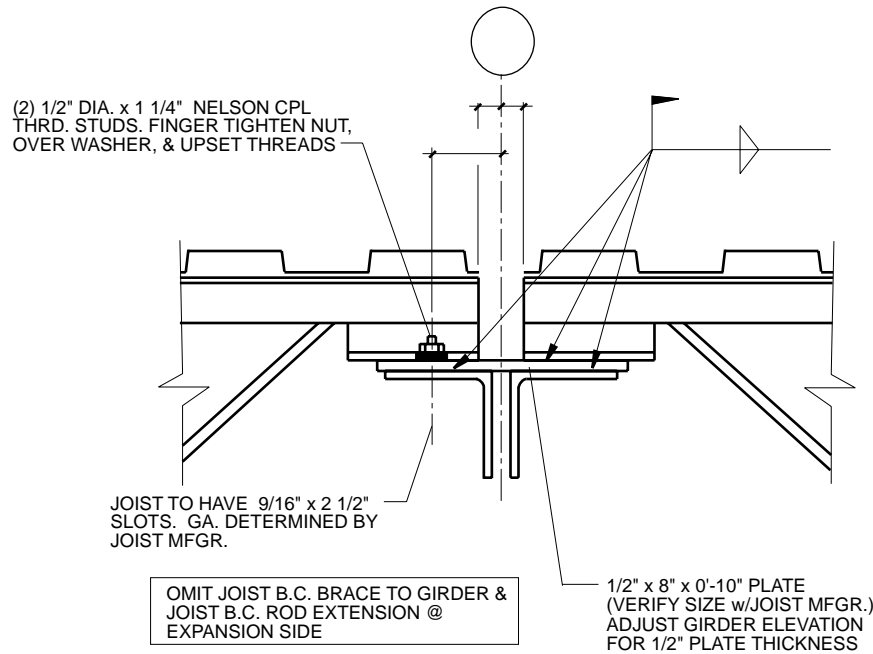


Fig. 5.10.1 Expansion Joint

Special Profile Joists

Special consideration must be given when non-standard type joists are used. Several examples of non-standard type profiles are illustrated in Figure 5.10.2. The local Vulcraft representative should be consulted prior to specifying these joists. These joists are custom designed for each application and the feasibility of the desired profile must be verified for each situation.

Particular care must be taken in the specification and support structure design for scissors and arched chord joists. When these members deflect vertically under gravity loads, the end supports of the member, if unrestrained, translate outward a significant amount. See Figure 5.10.3.

If this translation is restrained, horizontal thrusts are imparted to the support structure. The specifying engineer must either specify a sliding base at one end of the joist with a deflection criterion, or the support structure must be designed to resist the horizontal thrusts. The specifying engineer may contact a Vulcraft office for assistance in determining the horizontal reactions.

The construction documents should clearly indicate the type of support (pinned or sliding) used.

Sloping Joists

Currently SJI specifications do not address joists that are to be used at a slope rate greater than 1/2 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 problems with sloped joist designation includes:

1. The use of horizontal projection as the span.
2. The inconsistency in how loads are being applied to sloped joists.
3. The affect of the load component parallel to the chords of the joists.

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 axis.

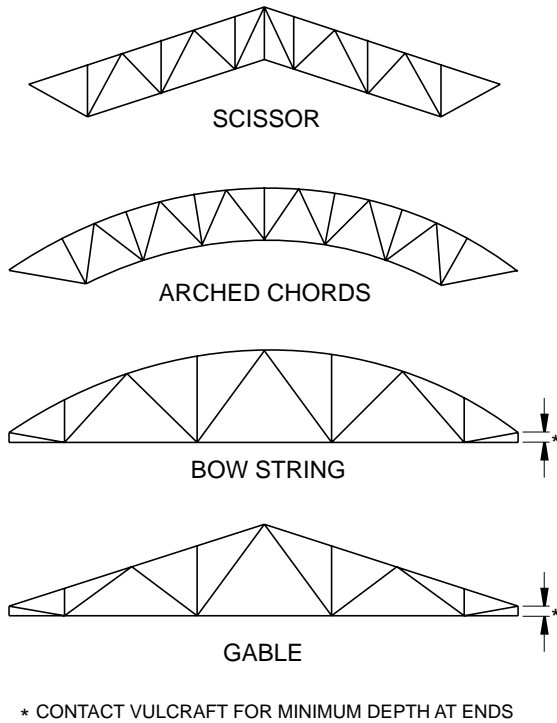


Fig. 5.10.2 Non Standard Joist Types

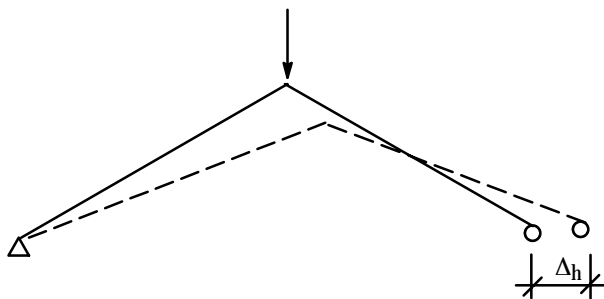
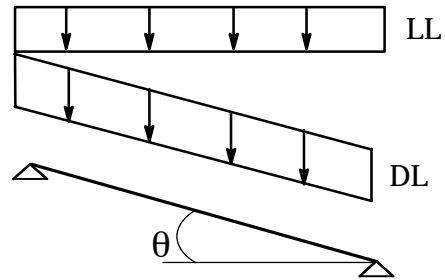
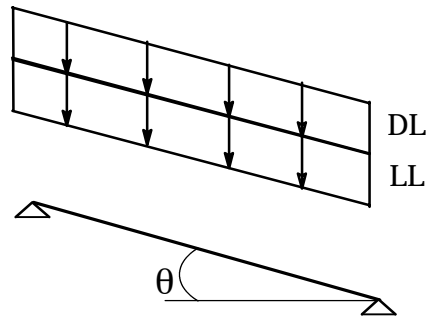


Fig. 5.10.3 Loaded Scissor Joist

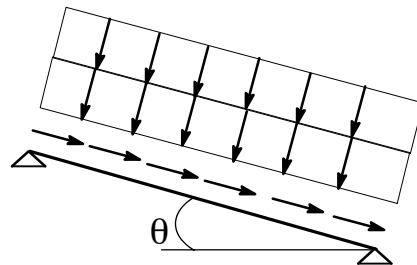
The live load is applied over the plan length of the member and the dead load is applied over the slope length.



To orient both loadings to the same axis, multiply the live load by the $\cos \theta$.



To determine the normal component of each, multiply again by the $\cos \theta$.



Using the normal and parallel components of the loading the proper joist can be specified.

This method produces several benefits in that it:

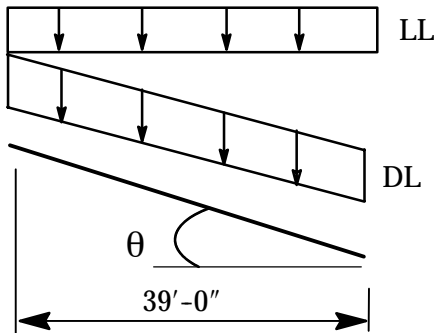
1. Eliminates the need for additional load tables.
2. Ensures the joist will be designed for the moment capacity for which it was specified.
3. Considers the actual joist length during selection, preventing overspan conditions.
4. Provides a standard procedure compatible with current SJI load tables.

5. Adds the parallel component into the top chord axial force of the joists.

Example 5.10.1 Sloping Joists

Determine the joist to be specified for the following:

Roof slope = 6:12
 LL = 14 psf
 DL = 22 psf
 Plan dimension of bay,
 $L_p = 39'-0''$
 Typical joist spacing = 5'-0" o/c



$\theta = \tan^{-1} (6/12) = 26.6^\circ$
 $LL \times \cos^2 \theta \times \text{joist space} = 56 \text{ plf}$
 $DL \times \cos \theta \times \text{joist space} = 99 \text{ plf}$
 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 joist load table using TL = 155 plf and LL = 56 psf and joist length = 43'-7".

Specify a 22K5 (for 44'-0" span, allowable uniform total load = 157 psf, live load that produces deflection of $L/360 = 76 \text{ plf}$).

In addition, the manufacturer will need to design this joist for the affects of the load parallel to the joist. This load would be:

$$[(LL \times \cos \theta) + DL] \sin \theta = 77 \text{ plf}$$

This load will be applied as an additional top chord axial force in the joist by the manufacturer.

Joist Splices

Long span joists are spliced when required for shipping and handling. Per SJI Technical Digest No. 9, it is the erector's responsibility to "match mates". Technical Digest No. 9 goes on to state: "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! 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.

CHAPTER 6

SPECIFICATION OF COMPONENTS

6.1 INTRODUCTION

The purpose of this chapter is to discuss the proper specification of joists and girders for imposed loads, and to discuss the proper designation of joists, joist girders and bridging on the design drawings. A summary of the extent of information required on the structural drawings is contained in Chapter 8. Since steel deck, joists, and joist girders are currently designed using Allowable Stress Design, the specifying engineer, when using Load and Resistance Factor Design, must transmit all loads and forces to the manufacturer unfactored.

The engineer should consider the information required as consisting of two portions.

The first portion involves the specification of the required dimensions and the structural capacity of the joist or joist girder required to withstand the applied loads.

The second portion relates to the design of the joist and joist girder connection details. The connection types are usually indicated on the structural drawings. It is the responsibility of the specifying engineer to indicate the type of attachment for every joist and joist girder in the structure. The engineer is cautioned to take particular care in the design of the joist and joist girder connections. As illustrated in Chapters 4 and 7, the configuration of the details transferring axial forces into and out of joist or joist girder chords has considerable impact on the design of the joists and joist girders.

6.2 JOISTS SUBJECTED TO UNIFORM GRAVITY LOADS

Joist Selection

For joists subjected to a uniform gravity load, the joist designation can be selected directly from the SJI load tables. To determine the load per lineal foot applied to the joist, the engineer multiplies the live load (or snow load) and the total load times the tributary width supported by the joist. It should be noted that 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 table with sufficient capacity to resist the applied uniform load and to meet the project deflection criteria. The SJI tables contain the allowable total uniform load and the allowable live load that an individual joist can support for a given span. The tabulated allowable live load

indicates the load that causes a deflection of $L/360$ for the joist. If a greater live load deflection is acceptable, the deflection check may be made by ratio. Vulcraft's catalog contains an economical joist guide that is helpful to the designer in making a least weight joist selection. This table does not account for bridging. Under certain conditions a heavier joist with less bridging, may result in less cost.

In the load tables, the top (compression) chord of the joist is taken as continuously braced against out of plane buckling. If this is not the case, the chord bracing which is available should be indicated on the drawings.

Custom Joist Designs

Custom joists are often used when the required load per lineal foot of joist exceeds the loads listed in the SJI tables, or when the engineer wishes to match the joist capacity to the load requirements as closely as possible. If the engineer wishes to specify a custom joist to resist the uniform load, the following designation format should be used:

X SPEC TL/LL

where:

- X = The depth of the joist in inches.
- SPEC = The appropriate SJI joist series, i.e. K, LH, DLH
- TL = The total uniform load applied to the joist in lbs./ft.
- LL = The uniform live load applied to the joist in lbs./ft.

An example of this would be 28K600/400. The manufacturer will design the joist for the loads indicated. If the specifier wants to have the joist designed for a live load deflection of span over 240, he must so specify or he can adjust the LL in the designation, e.g. 28K600/267.

The engineer must verify that the specified joist can be manufactured with the standard seat depth. Provided in Tables 6.1 and 6.2 are values of maximum moment with associated joist depths for which 2.5 or 5.0 inch seat depths can be used. The specifying engineer needs only to determine the maximum end moment or centerline moment for a given joist, and to compare the calculated moment to the moment listed in the respective table. If the calculated mo-

SPECIFICATION OF COMPONENTS

ment is less than the moment listed for the given joist, then the associated seat depth can be furnished, otherwise coped seats would be required.

For example, if the 28K 600/400 custom joist shown

above had a clear span of 40 feet the centerline moment equals $WL^2/8 = (0.6)(40)^2/8 = 120$ ft.-kips. This is less than the 130 ft.-kip moment listed in Table 6.1; thus a 2-1/2 inch seat can be used.

K Series Steel Joists	
Joist Depth (inches)	Limiting maximum joist moment with 2.5 inch deep seats (ft.-kips)
18	81
20	91
22	100
24	110
26	120
28	130
30	140

**Table 6.1 Limiting Maximum K-Series Joist Moments
with 2.5 inch Deep Seats**

LH Steel Joist	
Joist Depth (inches)	Limiting maximum joist moment with 5 inch deep seats (ft.-kips)
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 6.2 Limiting Maximum LH Joist Moments
with 5 inch Deep Seats**

The seat depth may often be determined by comparison with joists listed in the standard joist tables or by contacting the manufacturer. The reader is referred to the sec-

tion on special joists below for additional information on seat depths.

6.3 JOISTS RESISTING CONCENTRATED LOADS

When considering concentrated loads in a joist floor or roof system, the engineer must choose the appropriate means to resist that concentrated load. The options are:

1. Use a special joist designed for the uniform load and the concentrated load.
2. Use a heavier standard joist to resist the uniform load and the concentrated load, i.e. a joist that covers the shear and moment diagram for the loads imposed.
3. Use a KCS series joist.
4. Substitute a wide flange beam to resist the uniform and concentrated loads.

Options 1, 2 and 3 may consist of a single joist or a double joist. The selection of a single joist is generally the most economical choice. If double joists are used, the specifying engineer must insure that the loads are transferred into each joist.

Special Joists

In general, the most economical option is to use a special joist to resist the concentrated load. 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 of the joist. 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 would result from this moment and this end shear are calculated. If a standard K joist of the desired depth can be selected to resist the greater of these uniform loads, the manufacturer will be able to supply the special joist with the 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 7.5 inch seat depth. Examination of the KCS series 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 also will have to be accounted for in the height of the support steel and possibly the determination of the eave height.

The specification of a special joist for a concentrated load involves three steps:

1. Locate the special joist on the plan and provide a load diagram for that joist.

2. Verify that the joist can be manufactured with a seat depth compatible with the surrounding joists.
3. Account for chord bending when concentrated loads cannot be located at panel points.

A suggested procedure for specification of the loads on a special joist is contained in the SJI Code of Standard Practice in Section 5.5. The Code of Standard Practice suggests that the engineer choose a standard joist and provide a load diagram. The Code of Standard Practice also contains an example illustrating the proposed procedure. An example of two joist load diagrams is given in Figure 6.3.1.

Standard Joists

In many situations, concentrated loads may be resisted with a standard joist. To select the appropriate joist the engineer must choose a joist that has sufficient shear and moment capacity to resist the loads. The engineer must calculate the moment and shears due to the combined loads on the joist. The equivalent uniform load that would result in the maximum moment can be calculated. The equivalent uniform load that would result in an allowable shear diagram that completely covers the actual shear diagram must also be calculated. The larger of these two equivalent uniform loads should 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 special joist or a KCS series joist should be used. It should be noted that specifying a standard joist to resist concentrated loads is generally less economical than specifying a special joist. Also, a load diagram should always be provided when the joist is subjected to a partial length uniform load or non-uniform load of a magnitude greater than the published allowable uniform load for the joists. This information must be transmitted to the manufacturer so that the joist chord can be checked for bending between the panel points. The selection of a standard joist to resist a concentrated load is illustrated in the following example.

Example 6.3.1 Joist with a Concentrated Load

Select a joist to carry a uniform load of 200 plf plus a concentrated load of 600 lbs located 6 feet from one end.

Solution:

1. Solve for reactions:

$$R_L = 600 \times 30/36 + 200 \times 36/2 = 4100 \text{ lbs.}$$

$$R_R = 600 \times 6/36 + 200 \times 36/2 = 3700 \text{ lbs.}$$

SPECIFICATION OF COMPONENTS

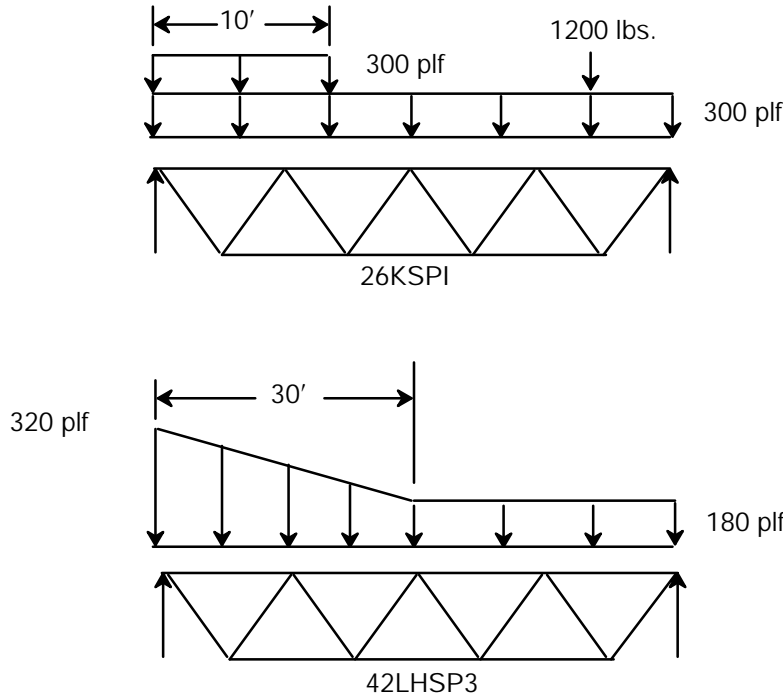


Fig. 6.3.1 Joist Load Diagrams

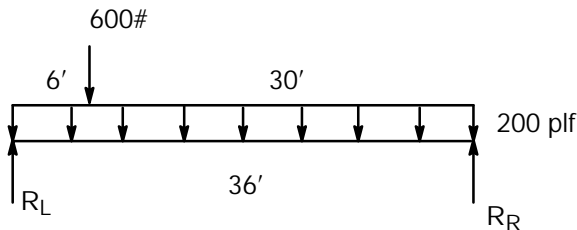


Fig. 6.3.2 Example 6.3.1

2. Solve for the maximum moment:

Zero shear is located at $3700/200 = 18.5'$ from the right end.

(Note location of point of zero shear. Possible shear reversal is insignificant.)

$$M = 18.5(3700) - 200(18.5)^2/2$$

$$M = 34,225 \text{ ft.-lbs.}$$

3. Solve for the end shear that will completely cover the actual shear diagram. See Fig. 6.3.3.

4. Solve for the equivalent uniform loads based on the maximum moment and joist end shear:

Shear :

$$W_{eq} = V_{end}/18$$

$$W_{eq} = 4350/18 = 242 \text{ plf}$$

Moment:

$$W_{eq} = 8M/L^2$$

$$W_{eq} = (8 \times 34,225)/36^2 = 211 \text{ plf}$$

∴ Choose 22K6

$$W_{allow} = 257 \text{ plf}$$

Note: Concentrated load reinforcement may be required at the concentrated load location.

KCS Series Joists

A versatile alternative to requiring special joists and selecting standard joists for resisting concentrated loads is the use of KCS series joists. The KCS series joist is a Vulcraft standard design. It has a constant shear capacity and a constant moment capacity throughout its length. All of the KCS series joist diagonals, except the end diagonals, are designed for 100% stress reversal. The end diagonals are designed for tension only, since stress reversal will never occur under gravity loading. The load tables for KCS series joists list the shear and moment capacity of each KCS series joist. The selection of a KCS series joist is analogous to selecting a wide flange beam. The designer calculates the maximum moment and shear imposed, and selects the ap-

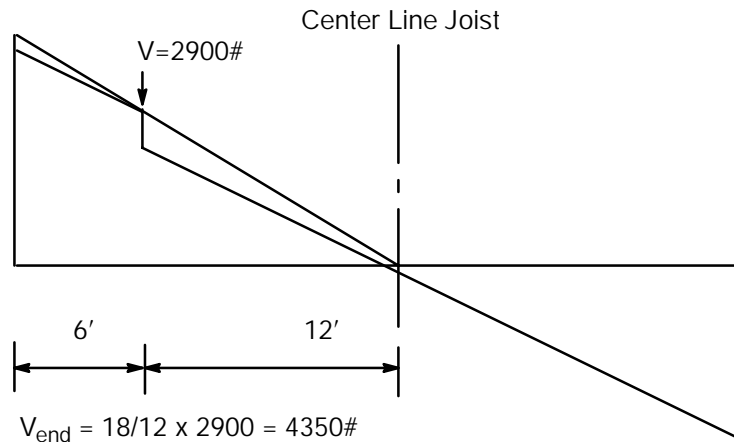


Fig. 6.3.3 Shear Diagram

appropriate joist. If the concentrated load does not fall at a panel point, the designer must account for chord bending.

Concentrated Load Reinforcement

Where concentrated loads cannot be located at panel points, chord bending is induced in addition to the other forces in the joist. Chord bending can either be resisted by the chord itself or be eliminated by the addition of a web member at the concentrated load.

When the magnitude and location of the load can be specified, the chord can be checked by the manufacturer who has the option of increasing the chord size or adding a

web member. Depending on the requirements of the contract, such web members may be installed in the shop, or in the field by the joist erector.

When the magnitude of load is known but the location is not, the chord design is more complicated since several locations must be checked to determine the critical condition. Again, either the chord can be increased in size or a web member added. Since the locations are unknown, the web members, if required, would, of necessity, be field installed by the joist erector once the locations are determined. An illustration of concentrated load reinforcement using added web members is shown in Figure 6.3.4.

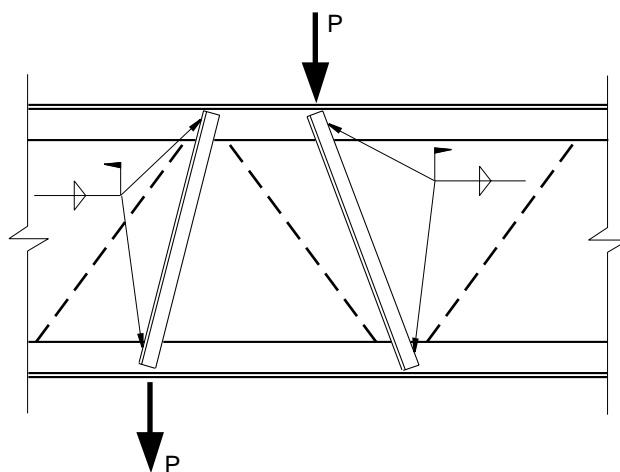


Fig. 6.3.4 Concentrated Load Reinforcement

Beams

In some situations the use of a wide flange beam in place of joists is warranted. If the load is due to crane or conveyor loading, the use of a beam will mitigate the problems associated with fatigue. In some cases the load cannot be conveniently attached to the joists, and the use of a beam substitute may solve difficult detailing problems. The use of beam framing around large openings can facilitate the attachment of headers or stair framing.

If the engineer decides to use a beam in place of a joist, the beam should have an end seat designed with the same depth as the 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. If this is not possible, a beam with a thicker web should be selected or the beam should be attached to a vertical member on the joist girder. An example of the design of 2.5 inch deep end seat for a beam is presented in Section 5.2.

6.4 END MOMENTS AND CHORD FORCES IN JOISTS

Joist End Moments

When joists are used as part of a rigid frame, the engineer must provide the joist end moments to the manufacturer. This may be accomplished through the use of a joist load diagram or a schedule of joist moments. The schedule or diagram should include the magnitude and direction of the moments for the various load cases considered. In addition, the specifying engineer should specify that the bottom chord braces be designed and furnished by the joist manufacturer. 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 the end moments on joists is illustrated in Figure 6.4.1.

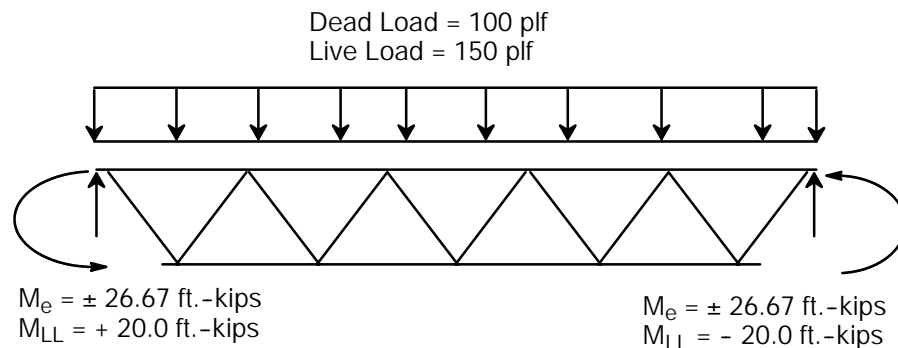


Fig. 6.4.1 Joist Load Diagram

Joist Schedules

The use of a schedule to specify the end moments on joists is illustrated in Figure 6.4. The schedules can be adjusted to have as many headings as required. For example, a heading for required moment of inertia could be added if required by the frame analysis.

Chord Forces

Bracing systems and moment frames may impart axial loads into joist chords. These forces should be specified to the manufacturer either on a load diagram or in a schedule. As mentioned above, the specifying engineer should also specify that bottom chord braces be designed and furnished by the joist manufacturer. Vulcraft will check the effect of the chord forces and adjust the chord design accordingly. A procedure for determining the capacity of a joist chord to resist applied chord forces is illustrated in Example 4.2.1. A special case of transferring chord 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 of the joist. This occurs when a wall brace is attached to the bottom chord of a joist and the roof bracing is in the horizontal plane of the top chord. The design and specification of joists for this condition is discussed in Chapter 4. An example load diagram is also provided in Figure 4.2.15.

As mentioned in Section 6.2, the specifying engineer must also verify if the joist requires a special depth end seat. Tables 6.1 and 6.2 can be used by multiplying the chord force times the effective joist depth to obtain the joist moment. This moment can then be compared to the limiting moments shown in the tables.

6.5 JOISTS AND JOIST GIRDERS SUBJECTED TO UPLIFT LOADING

Joist and joist girders in roof systems will be subjected to net uplift loads if the code imposed wind uplift exceeds the permanent dead load. This uplift loading will effect the design of the members and the bridging. Under gravity loads, the top chord of the joist 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 bridging design must always be adjusted to account for the uplift condition. Uplift also causes a stress reversal in the joist diagonals. This condition must be checked by the manufacturer.

SJI specifications require that the joist manufacturer be informed of the net uplift occurring on the joists and joist girders. This may be accomplished with a note on the drawings such as “Design and furnish joists and bridging for a net uplift of 15 psf.” Many building codes require that components and cladding be designed to resist increased wind loading at corners and edges. In this situation, the best method for informing the manufacturer of the net uplift on the joists is to provide a net uplift diagram. This is illustrated in Figure 6.5.1.

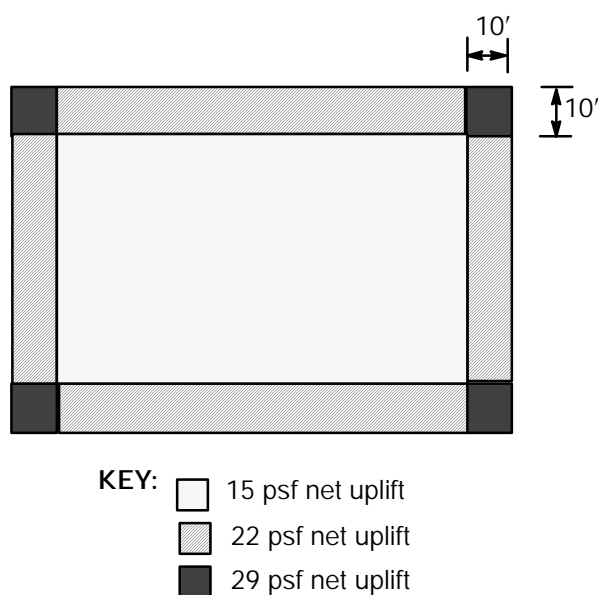


Fig. 6.5.1 Joist Uplift Load Diagram

As mentioned, the engineer should also specify the uplift load on the joist girders. The manufacturer will design bottom chord braces for the girder as required by the uplift load. Joist girders may be considered as primary members when determining the uplift loads. Because of this, the uplift load on joist girders would usually be less than the uplift load on joists, and therefore the engineer may elect to specify a lower net uplift load on the joist girders.

6.6 JOIST GIRDERS SUBJECTED TO GRAVITY LOADS

For simple span joist girders subjected to equal uniformly spaced point loads, noting the joist girder designation on the plan provides an adequate specification of the

member. An example of a standard joist girder designation is 42G12N9K. 42 indicates the midspan depth of the joist girder. G indicates the joist girder series. 12N indicates the number of joist spaces. 9K indicates the magnitude of each panel point load in kips. The specifier should include the weight of the joist girder in the panel point load. Vulcraft will use the most economical web configuration based on the depth of the joist girder and the spacing of the joists. Figure 6.6.1 illustrates 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.

Vulcraft also offers a VG series joist girder. The VG type has the largest amount 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 6.6.2. The VG type is slightly more expensive than a G type.

If the spacing and magnitude of loading varies, the engineer should use a load diagram to illustrate the loading applied to the joist girder. Loading applied to the bottom chord of the member should also be indicated on the load diagram. An illustration of a joist girder subjected to an uneven load distribution is illustrated in Figure 6.6.3. Vulcraft will determine the optimum web configuration for the joist girder.

If the girders are used as part of a rigid frame system or bracing system, the end moments or chord forces should be illustrated in a load diagram or with the use of a schedule. Bottom chord braces should be specified to be designed and furnished by the joist girder manufacturer. An example of a joist and joist girder schedule is provided in Figure 6.6.4. Vulcraft's design procedure for joist girders subjected to end moments is analogous to the design procedure for joists subjected to end moments. The member is first designed as a simple span member, and then the chords and diagonals for the joist girder are checked (and resized as required) for the effect of the end moments.

The schedule depicted in Figure 6.6.5 is convenient when designing multiple buildings, or when a relatively small number of changes are required to change the schedule from one project to another.

6.7 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.

SPECIFICATION OF COMPONENTS

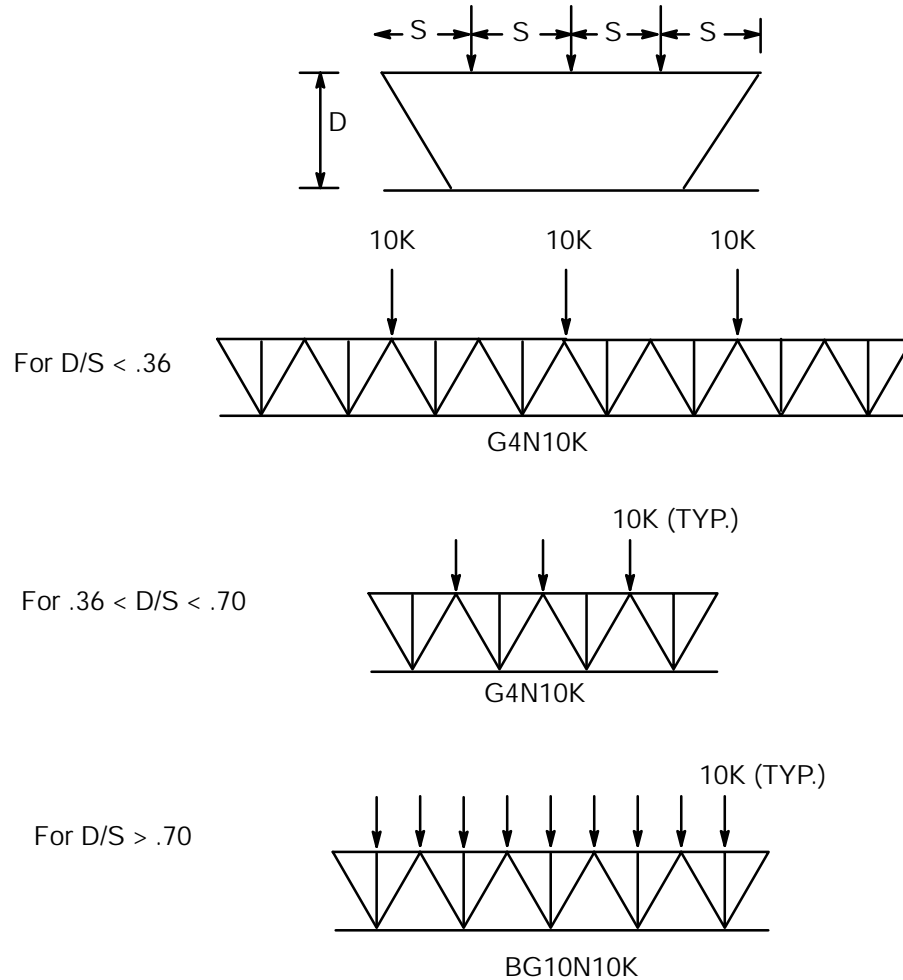


Fig. 6.6.1 Joist Girder Web Configurations

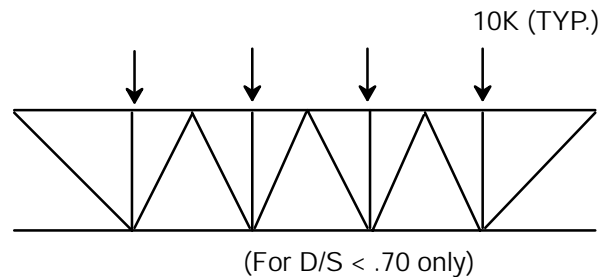


Fig. 6.6.2 VG5N10K Joist Girder

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 consider-

ation. 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 rods or 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

SPECIFICATION OF COMPONENTS

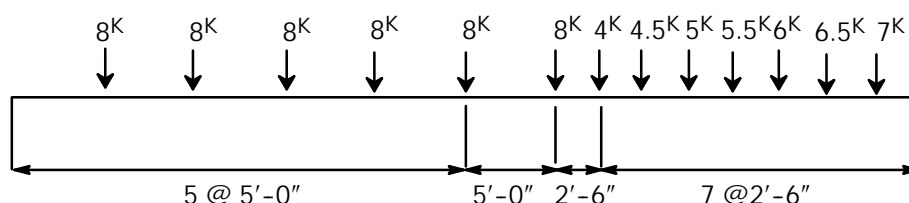





Fig. 6.6.3 Joist Girder Load Diagram

Mark	Designation	Load Combination	Uplift plf	Moments		Chord Force		Remarks
				<div><div>+</div><div></div><div>+</div></div> <div>kip-ft.</div>		<div><div>+</div><div></div><div>+</div></div> <div>kips</div> <div></div>		
				Left	Right	Top	Bottom	

Note: If forces shown are factored (LRFD) or unfactored (ASD).

Fig. 6.6.4 Joist and Joist Girder Schedule

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 SLH and KCS series joists are contained in the Vulcraft catalog. The size, type and number of rows of bridging can be illustrated on the drawings, or alternatively a notation on the drawings can be used to specify the bridging requirements to the manufacturer.

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 between the last two joists. This condition often exists at expansion joints and when joists are used in lieu of beams at end walls.

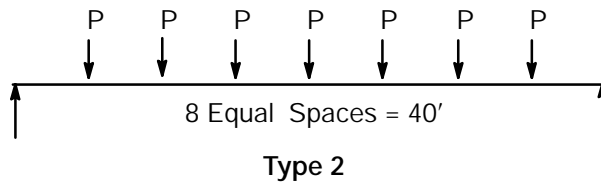
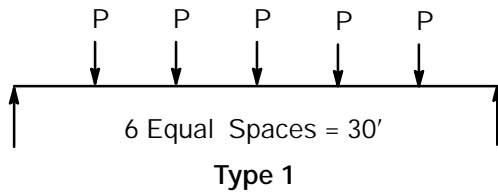
Standard bridging is required to laterally stabilize the top chord of the joists until the permanent deck is attached. Construction loads must not be applied to the joists until the bridging is attached to the joists and anchored at its ends.

Floor and roof decks usually have adequate stiffness to provide lateral stability to top chord of joists subjected to

design loads. The most common exception is standing seam roof systems. The engineer should assume that the standing seam roof has no diaphragm capability, and specify that sufficient 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 bridging design (size and spacing) must be adjusted to provide sufficient lateral bracing to the joist for the design loads.

Wind loading on joists will effect the design of the bridging. Under net uplift, the bridging is required to provide lateral stability to the bottom (compression) chord of the joists. 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 6.5 regarding the specification of uplift forces.

JOIST GIRDER SCHEDULE			
Mark	Type	P (kips)	Notes
JG1	1	8.0	1, 2
JG2	2	6.0	1, 3
JG3	2	12.0	1, 3



EXAMPLE NOTES:

1. Maximum depth 42 inches
2. VG profile required
3. A minimum moment of inertia of 20,000 in⁴ is required.

Fig. 6.6.5 Sample Joist Girder Schedule

Lastly, the use of standard bridging as a brace to resist lateral loads due to monorails or other equipment should be evaluated carefully, because the forces involved are usually greater than the stability forces for which bridging is typically designed.

6.8 SAMPLE SPECIFICATIONS

This section presents four sample specifications covering the following:

- 1) Steel Joists and Joist Girders
- 2) Steel Roof Deck
- 3) Steel Form Deck
- 4) Composite Steel Deck

These specifications were developed by the authors using the Construction Specifications Institute three part format and the basic information presented in the respective sections in CSI Spectext. The reader is encouraged to review these specifications as they contain numerous items which simplify the basic specifications for these materials. They should not be used however without modifications for the project at hand.

STEEL JOISTS AND JOIST GIRDERS

Section 05210

PART 1 - GENERAL

1.01 SECTION INCLUDES

- A. Applicable requirements of Condition of Contract and General Requirements apply to Work specified in this Section.
- B. Work included:
 - 1. Provide open web steel joists, long span steel joists, and bridging.
 - 2. Provide steel joist girders.
 - 3. Provide all accessories per SJI requirements.
- C. Related work specified elsewhere:
 - 1. Structural Steel: Section 05100
 - 2. Steel Roof Deck: Section 05310
 - 3. Steel Form Deck: Section 05320
 - 4. Composite Steel Deck: Section 05330

1.02 QUALITY ASSURANCE

- A. Steel joist and joist girder manufacturer: Member of SJI.
- B. Steel joists and joist girders: SJI approved.
- C. Conform to SJI Standard Specifications, Load Tables and Weight Tables.

1.03 REFERENCES

- A. ASTM A325 - High Strength Bolts for Structural Steel Joints.
- B. AWS D1.1 - Structural Welding Code.
- C. SJI - "Standard Specification for Open Web Steel Joists K-Series", and "Standard Specification for Longspan Steel Joists LH-Series, and Deep Longspan Steel Joists DLH-Series" and "Standard Specifications for Joist Girders."
- D. SJI - "Recommended Code of Standard Practice for Steel Joists and Joist Girders."
- E. AISC - Manual of Steel Construction.
- F. SJI Technical Digest #9 - "Handling and Erection of Steel Joists and Joist Girders."

1.04 SUBMITTALS

- A. Submit shop drawings under provisions of Section 01300.
- B. Indicate standard designations, sizes, spacing and locations of joists, bridging, connections, attachments and top and bottom chord extensions.
- C. Design for special joists and joist girders:
 - 1. Special joists and joist girders shall be designed for the load designations specified on the structural drawings. Designs shall properly account for the distribution of concentrated loads, live loads and for the effect of openings. Designs are to meet requirements of SJI.
 - 2. Joists and joist girders shall meet the following deflection criteria per SJI. Maximum live load deflection shall not exceed: $L/240$ for roofs; $L/360$ for floors where L = span length, center to center of bearing.
 - 3. Designs shall include the net wind uplift loads indicated on the drawings.
 - 4. Provide joist girder bottom chord bracing to meet SJI slenderness ratio criteria. Bracing shall not develop continuity in the joist system unless continuity has been provided for in the joists.

1.05 STORAGE

- A. Store materials off ground on wood sleepers.
- B. Storage and handling of steel joists to conform to SJI Technical Digest #9.

PART 2 - PRODUCTS

2.01 MATERIALS

- A. Steel Joists and Joist Girders: Meet SJI Standard Specifications. Cold-formed joist chord members are unacceptable.
- B. Bolts, Nuts and Washers: ASTM - A325.
- C. Primer: Manufacturer's standard primer.

2.02 FABRICATION

- A. Fabricate steel joists and joist girders in accordance with the approved shop drawings and SJI Standard Specification.
- B. Provide top and bottom joist chord extensions where indicated.

- C. Bearing:
 - 1. Provide sloped bearing ends where joist or joist girders slope exceeds 1/4" in 12".
 - 2. Provide bearing lengths per SJI requirements unless greater bearing lengths are shown on the Drawings.
- D. Remove loose scale, rust and other foreign materials from fabricated joists, joist girders and accessories and apply one coat of primer paint per SJI specifications.
- 2. Erected vertical alignment shall not exceed D/48, where D is the joist depth in inches.
- F. Do not permit erection of decking until joists are braced and bridged.
- G. Do not field cut or alter joists and joist girders without written approval of Engineer.
- H. After erection, prime welds, abrasions and surfaces not primed. Use primer consistent with shop coat.

END OF SECTION

PART 3 - EXECUTION

3.01 INSPECTION

- A. Verify completed construction will permit and support steel joist and joist girder erection.
- B. In the event of discrepancy, immediately notify the Engineer.
- C. Do not proceed or install in areas of discrepancy until all such discrepancies have been fully resolved.

3.02 PREPARATION

- A. Coordinate placement of anchorages in concrete and masonry construction for securing bearing plates and angles.

3.03 ERECTION

- A. Erect steel joists and joist girders in accordance with the approved shop drawings and SJI Standard Specifications and SJI Technical Digest #9.
- B. Bear joists and joist girders on supports in accordance with SJI.
- C. During erection, provide temporary bracing for induced loads and stresses.
- D. Field weld joist seats to preset bearing plates and angles after alignment and positioning and after installation of bridging.
 - 1. Welded Joints: Comply with AWS 1.1.
 - 2. Welding Electrodes and Fillets: AISC Specification.
- E. Positioning:
 - 1. Erected horizontal sweep shall not exceed L/360.

STEEL ROOF DECK

Section 05310

PART 1 - GENERAL

1.01 SECTION INCLUDES

- A. Applicable requirements of Condition of Contract and General Requirements apply to Work specified in this Section.
- B. Work included:
 - 1. Provide steel roof deck.
- C. Related work specified elsewhere:
 - 1. Structural Steel: Section 05100
 - 2. Steel Joists: Section 05200
 - 3. Metal Fabrications: Section 05500 (Bearing plates and angles).

1.02 QUALITY ASSURANCE

- A. Metal deck shall be designed in accordance with the latest edition of the Steel Deck Institute's (SDI) - "Specifications and Commentary for Steel Roof Deck."
- B. The roof deck shall be designed, manufactured and installed to meet Factory Mutual Class I construction.

1.03 REFERENCE STANDARDS

- A. AISI - "Specification for the Design of Cold-Formed Steel Structural Members."
- B. ASTM A36 - Structural Steel.
- C. ASTM A611 - Structural Steel, Sheet, Carbon, Cold-Rolled
- D. ASTM A653 - Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot Dip Process.

- E. AWS D1.1 – Structural Welding Code.
- F. AWS D1.3 – Specification for Welding Sheet Steel in Structures.
- G. SDI – “Design Manual for Composite Decks, Form Decks and Roof Decks.”
- H. AISC – Manual of Steel Construction.
- I. SDI – Diaphragm Design Manual.

1.04 SUBMITTALS

- A. Submit Shop Drawings for review of general conformance to design concept in accordance with Section 01300. Erection Drawings shall show type of deck, shop finish, accessories, method of attaching, edge details, deck openings and reinforcement, and sequence of installation.

1.05 STORAGE

- A. Store materials off ground with one end elevated on wood sleepers to provide drainage. Protect deck from elements with a waterproof covering and ventilate to avoid condensation.

PART 2 – PRODUCTS

2.01 ACCEPTABLE MANUFACTURERS

- A. Vulcraft – A Division of Nucor Corporation.

2.02 MATERIALS

- A. Sheet steel shall conform to ASTM A611 Grade C, D or E (for prime painted decks) and ASTM A653, Structural Quality (for galvanized decks) and have a minimum yield strength of 33,000 psi.
- B. Bearing Plates and/or Angles shall be ASTM A36 steel.
- C. Welding Methods and Materials shall conform to AWS D1.1 and AWS D1.3.
- D. Steel Closure Strips, Ridge and Valley Plates, and Related Accessories shall be a minimum of 22 gage sheet steel of required profiles and sizes.
- E. Finish:
Galvanizing shall conform to the requirements of

ASTM 525 coating Class G60.
Shop Primer shall be acrylic medium gray.
Touch-up primer shall be compatible with manufacturer’s primer.

- F. Mechanical fasteners shall be Teks as manufactured by Buildex, St. Charles Road, Elgin, Illinois, 60120. Selection of Teks fasteners not specified herein shall be in accordance with the manufacturer’s recommendations.

- G. Acoustical Insulation shall be glass fiber type with profile to suit decking and be supplied by the deck manufacturer.

2.03 FABRICATION

- A. Steel deck shall have formed ribs of the type, finish, dimension and gage as shown on Drawings.
- B. Fabricate deck in lengths to have three continuous spans or more whenever possible. Fabricate sheets to lap a minimum of 2” over supports at ends. Lap joints required where roof pitch changes due to the deck support elevations.
- C. Design steel decking in accordance with SDI “Design Manual for Composite Decks, Form Decks, and Roof Decks.” The maximum working stress shall not exceed 20,000 psi. The maximum working stress shall in no case exceed the maximum yield strength of the steel divided by 1.65 but may be increased by 33% for temporary concentrated loads provided the deck thus required is not less than that required for the specific uniform load. The deflection of the Deck under design live load shall not exceed 1/240 of the span. Minimum thickness of material supplied shall be within 5% of the design thickness.
- D. Section properties used in determining stress and deflection shall be calculated in accordance with the latest edition of the Steel Deck Institute’s “Design Manual for Floor Decks and Roof Decks”.
- E. Fabricate roof sump pan of 14 gage sheet steel, flat bottom, sloped sides, recessed 1-1/2 inches below roof deck surface, bearing flange 3 inches wide, watertight.
- F. Provide 6” closure strip where changes in deck direction occur. Closure shall be same gauge as deck.

PART 3 - EXECUTION

3.01 INSTALLATION

- A. On steel support members provide 1-1/2" minimum bearing. Align and level on supports.
- B. Fasten steel deck units to structural supports using Hex washer head Tek's or arc spot welds according to manufacturers' specifications and erection layouts and as specified herein. Decks thinner than .0280 inches shall be welded using 16 ga. welding washers with a 3/8" diameter hole. Side lap connections shall be screwed or button punched depending on deck profile.
- C. Attach ridge and valley plates and steel cant strips directly to the steel deck where shown on the Drawings to provide a finished surface for the application of insulation and roofing.
- D. Cutting of openings through the deck less than 16 square feet in area, and all skew cutting shall be performed in the field.
- E. Arc spot welds (puddle welds) to supports shall have a diameter of 5/8" minimum, or an elongated weld of 3/8" minimum width and 3/4" minimum length. Weld metal shall penetrate all layers of deck material at end laps and have adequate fusion to the supporting members. Welding shall be done in accordance with the American Welding Society Standard "Specification for Welding Sheet Steel in Structures", AWS D1.3.
- F. Fastening of deck to supports and side laps.
 - 1. Deck ends at building perimeter: 12" o/c (36/4 min.)
 - 2. Deck end laps: 12" o/c (36/4 min.)

- 3. Deck sides at building perimeter and deck side laps: Deck units with spans greater than five feet shall be fastened at midspan or at 36" intervals whichever is smaller.

- 4. See drawings for requirements beyond these minimum requirements.

- G. At ends of decks or where changes of deck direction occur, fasten at each flute. Furnish and install adequate closures and fasten to both sides at 18" o.c.
- H. Accessories shall be fastened to supports or deck with mechanical fasteners at not over 6" o.c. and at all corners and ends.
- I. Position roof sump pans with flange bearing on top surface of deck. Screw at each deck flute.

3.02 CLEAN UP AND FINAL ADJUSTMENTS

- A. Touch up surface coating damage and abrasions using a paint compatible with primer paint and/or specially formulated for use with galvanized steel.
- B. Installation holes shall be sealed with a closure plate 2 gauges thicker than deck and mechanically fastened to deck. Steel deck with holes visible from below will be rejected. Deck units that are bent, warped, or damaged in any way which would impair the strength and appearance of the deck shall be removed from site.
- C. Steel decking work and accessories, when complete, shall be solid, smooth, and uniform in appearance.
- D. Remove any unused steel deck, edge trimmings, screws, weld washers, butt ends of welding electrodes and other debris from completed installation.

END OF SECTION

STEEL FORM DECK
Section 05320

PART 1 - GENERAL

1.01 SECTION INCLUDES

- A. Applicable requirements of Conditions of Contract and General Requirements apply to Work specified in this Section.
- B. Work included:
 - 1. Provide steel form deck and accessories for forming of concrete floors.
- C. Related work specified elsewhere:
 - 1. Cast-In-Place-Concrete: Section 03300
 - 2. Concrete Reinforcement: Section 03200
 - 3. Structural Steel: Section 05100
 - 4. Steel Joists: Section 05210
 - 5. Metal Fabrications: Section 05500 (Bearing plates and angles).
 - 6. Electrical: Division 16 (telephone, floor outlets, raceway and covers).

1.02 QUALITY ASSURANCE

- A. Steel deck shall be designed in accordance with the latest edition of the Steel Deck Institute's (SDI) "Specifications and Commentary for Non-Composite Steel Form Deck."

1.03 REFERENCE STANDARDS

- A. AISI - "Specification for the Design of Cold-Formed Steel Structural Members."
- B. ASTM A36 - Structural Steel.
- C. ASTM A611 - Structural Steel, Sheet, Carbon, Cold-Rolled
- D. ASTM A653 - Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot Dip Process.
- E. AWS D1.1 - Structural Welding Code.
- F. AWS D1.3 - Specification for Welding Sheet Steel in Structures.
- G. SDI - "Design Manual for Composite Decks, Form Decks, Roof Decks."

- H. AISC - "Manual of Steel Construction."

- I. SDI - Diaphragm Design Manual

1.04 SUBMITTALS

- A. Submit Shop Drawings for review of general conformance to design concept in accordance with Section 01300. Shop Drawings shall show type of deck, shop finish, accessories, method of attaching, edge details, deck openings and reinforcement, and sequence of installation.

1.05 STORAGE

- A. Store materials off ground with one end elevated on wood sleepers to provide drainage. Protect deck from elements with a waterproof covering and ventilate to avoid condensation.

PART 2 - PRODUCTS

2.01 ACCEPTABLE MANUFACTURERS

- A. Vulcraft - A Division of Nucor Corporation.

2.02 MATERIALS

- A. Sheet steel shall conform to ASTM A653, Structural Quality (for galvanized decks).
 - 1. Fluted Decks and Ribbed Steel Forms 1-1/2 inches or higher shall be formed from sheet steel having a minimum yield strength of 33,000 psi (Grade 33).
 - 2. Ribbed Steel Forms less than 1-1/2 inches high shall be formed from sheet steel having a minimum yield strength of 80,000 psi (Grade 60).
- B. Bearing Plate and/or Angles shall be ASTM A36 steel.
- C. Welding Methods and Materials shall conform to AWS D1.1 and AWS D1.3.
- D. Metal Closure Strips, Wet Concrete Stops, Cover Plates and Related Accessories shall be a minimum of 22 gage sheet steel of required profiles and sizes.
- E. Finish:
 - Galvanizing shall conform to the requirements of ASTM 525 coating class G60.
 - Shop Primer shall be acrylic medium gray.
 - Touch-up primer shall be compatible with manufacturer's primer.

- F. Mechanical fasteners shall be Teks as manufactured by Buildex, St. Charles Road, Elgin, Illinois, 60120. Selection of Teks fasteners not specified herein shall be in accordance with the manufacturer's recommendations.

2.03 FABRICATION

- A. Steel Form Deck (metal centering) shall have formed ribs of the type, finish, dimension and gage shown on Drawings.
- B. Deck shall be capable of supporting loads indicated on the Drawings. Structural capacity of deck sections shall be established from section properties computed in strict accordance with the latest edition of the Steel Deck Institute "Design Manual for Composite Decks, Form Decks and Roof Decks". The maximum working stress shall not exceed the yield strength divided by 1.65. Minimum thickness of material supplied shall be within 5% of design thickness.
- C. Fabricate deck in lengths to have three continuous spans or more whenever possible. Fabricate Steel Deck and Ribbed Metal Forms to lap a minimum of 2" over supports at ends.
- D. Bearing lengths shall be established in accordance with the latest edition of the Steel Deck Institute's "Design Manual for Composite Decks, Form Decks and Roof Decks and shall be consistent with the deck capacity established in paragraph 2.03 B.

PART 3 - EXECUTION

3.01 INSTALLATION

- A. Deck and accessories shall be installed in accordance with the manufacturer's shop and erection drawings. Minimum bearing shall not be less than 1-1/2" inches.
- B. Fasten steel deck units to structural supports using Hex washer head Teks or arc spot welds according to manufacturer's specifications and erection layouts and as specified herein. Decks thinner than .0280 inches shall be welded using 16 ga. welding washers

with a 3/8" diameter hole. Side lap connections shall be screwed.

- C. Arc spot welds (puddle welds) to support shall have a diameter (weld nugget) of 3/8" minimum. Weld metal shall penetrate all layers of deck material at end laps and have adequate fusion to the supporting members. Welding shall be done in accordance with the American Welding Society Standard "Specification for Welding Sheet Steel in Structures", AWS D1.3.
- D. Fastening of deck to supports and side laps.
 - 1. Deck ends at building perimeter: 12" o/c (36/4 min.)
 - 2. Deck end laps: 12" o/c (36/4 min.)
 - 3. Deck sides at building perimeter and deck side laps: Deck units with spans greater than five feet shall be fastened at midspan or at 36" intervals whichever is smaller.
 - 4. See drawings for requirements beyond these minimum requirements.
- E. At ends of decks or where changes of deck direction occur, fasten at 12" o.c. Furnish and install adequate closures and fasten to both sides at 12" o.c.
- F. Cutting openings through the deck less than 16 square feet in area, and all skew cutting shall be performed in the field.
- G. Install sheet steel strip closures at all floor edge up-turned to thickness of slab to contain wet concrete. Provide closures of sufficient strength to remain in place without distortion.
- H. Install sheet closures and angle flashings to close openings between deck and walls, column, and openings.
- I. Accessories shall be fastened to supports or deck with mechanical fasteners at not over 18" o.c. and at all corners and ends.
- J. Concentrated loads and impact loads during erection and construction shall be avoided. Before the slab is poured, form deck shall be planked in all traffic areas to prevent damage to units.

3.02 CLEAN UP AND FINAL ADJUSTMENTS

- A. Touch up surface coating damage and abrasions using a paint specially formulated for use with galvanized steel. For painted deck use paint compatible with manufacturer's primer.
- B. Installation holes shall be sealed with a closure plate 2 gages thicker than deck and mechanically fastened to deck. Steel deck with holes visible from below will be rejected. Deck units that are bent, warped, or damaged in any way which would impair the strength and appearance of the deck shall be removed from site.
- C. Steel decking work and accessories, when complete, shall be solid, smooth, and uniform in appearance.
- D. Remove any unused steel deck, edge trimmings, screws, weld washers, butt ends of welding electrodes, and other debris from completed installation.

END OF SECTION

COMPOSITE STEEL DECK

Section 05330

PART 1 - GENERAL

1.01 SECTION INCLUDES

- A. Applicable requirements of Conditions of Contract and General Requirements apply to Work specified in this Section.
- B. Work included:
 - 1. Provide composite steel deck and accessories for forming of concrete floors.
- C. Related work specified elsewhere:
 - 1. Cast-In-Place Concrete: Section 03300
 - 2. Concrete Reinforcement: Section 03200
 - 3. Concrete Formwork: Section 03100
 - 4. Structural Steel: Section 05100
 - 5. Steel Joists: Section 05200
 - 6. Metal Fabrications: Section 05500 (Bearing plates and angles).

- 7. Electrical: Division 16 (telephone, floor outlets, raceway and covers).

1.02 QUALITY ASSURANCE

- A. Composite steel deck shall be designed in accordance with the latest edition of the Steel Deck Institute's (SDI) "Specifications and Commentary for Composite Steel Floor Deck."

1.03 REFERENCE STANDARDS

- A. AISI - "Specification for the Design of Cold-Formed Steel Structural Members."
- B. ASTM A36 - Structural Steel.
- C. ASTM A611 - Structural Steel, Sheet, Carbon, Cold-Rolled
- D. ASTM A653 - Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot Dip Process.
- E. AWS D1.1 - Structural Welding Code.
- F. AWS D1.3 - Specification for Welding Sheet Steel in Structures.
- G. SDI - "Design Manual for Composite Decks, Form Decks, Roof Decks."
- H. AISC - "Manual of Steel Construction."
- I. ASCE - "Specifications for the Design and Construction of Composite Slabs and Commentary on Specifications for the Design and Construction of Composite Slabs."
- J. SDI - Diaphragm Design Manual.

1.04 SUBMITTALS

- A. Submit Shop Drawings for review of general conformance to design concept in accordance with Section 01300. Shop Drawings shall show type of deck, shop finish, accessories, method of attaching, edge details, deck openings and reinforcement, and sequence of installation.

1.05 STORAGE

- A. Store materials off ground with one end elevated on wood sleepers to provide drainage. Protect deck from elements with a waterproof covering and ventilate to avoid condensation.

PART 2 - PRODUCTS

2.01 ACCEPTABLE MANUFACTURERS

- A. Vulcraft A Division of Nucor Corporation.

2.02 MATERIALS

- A. Composite steel floor deck shall be fabricated from steel sheet conforming to ASTM A611 Grades C or D, or A653, Structural Quality (or equal) having a minimum yield strength of 33 ksi.
- B. Bearing Plate and/or Angles shall be ASTM A36 steel.
- C. Welding Methods and Materials shall conform to AWS D1.1 and AWS D1.3.
- D. Steel Closure Strips, Wet Concrete Stops, Multiple Cell Headers, Cover Plates and Related Accessories shall be a minimum of 22 gage sheet steel of required profiles and sizes.
- E. Finish:
 - 1. The finish, as shown on the plans, is to be:
 - a) Galvanizing shall conform to the requirements of ASTM A653 coating class G60 or G90 or,
 - b) Phosphatized and painted over cleaned steel with the exposed side only painted. The side in contact with the concrete is to be phosphatized only. Paint is to be a baked-on acrylic primer.
 - c) If fireproofing is to be applied the paint shall be compatible with fireproofing materials.
 - d) Touch-up primer shall be compatible with manufacturer's primer.
- F. Mechanical fasteners shall be Teks as manufactured by Buildex, St. Charles Road, Elgin, Illinois, 60120.

Selection of Teks fasteners not specified herein shall be in accordance with the manufacturer's recommendations.

- G. The deck manufacturer shall have performed or have witnessed by a Registered Engineer, a sufficient number of tests on the composite deck/slab system to have determined load/deflection characteristics and the mode of failure under uniform or symmetrically placed point loads. Based on the test data the design load capacity shall be established by either elastic flexural analysis or ultimate strength analysis based on shear bond failure or flexural failure.

2.03 FABRICATION

- A. Steel Deck shall have formed ribs of the type, finish, dimension and gage shown on Drawings.
- B. Cellular Steel Deck units shall be a minimum of _____ inches wide and _____ inches high. The top sheet shall be a fluted profile of _____ gage. The bottom sheet shall be flat of _____ gage. Both sheets shall be formed from galvanized sheet steel. The cellular units shall conform to U.L. Assembly No. _____.
- C. The Composite Steel deck units shall serve as a form, total positive reinforcement, and partial temperature reinforcement for the concrete slab.
- D. Deck shall be capable of supporting uniform design loads as indicated on the Drawings. Structural capacity of deck sections shall be established from section properties computed in strict accordance with the latest edition of the "Steel Deck Institute Design Manual". The maximum working stress shall not exceed the yield strength divided by 1.65. Minimum thickness of material supplied shall be within 5% of design thickness. The deflection of the deck under design live load shall not exceed 1/360 of the span.
- E. Fabricate deck in lengths to have three continuous spans or more whenever possible. Fabricate Deck to butt ends allowing for a maximum of 1/8" gap.
- F. Bearing lengths shall be established in accordance with the latest edition of the Steel Deck Institute's "Design Manual for Composite Decks, Form Decks and Roof Decks and shall be consistent with the deck capacity established in paragraph 2.03D.

PART 3 - EXECUTION

3.01 INSTALLATION

- A. Deck and accessories shall be installed in accordance with the manufacturer's shop and erection drawings. Minimum bearing shall not be less than 1 - 1/2 inches.
- B. Fasten steel deck units to structural supports using Hex washer head Tek's or arc spot welds according to manufacturer's specifications and erection layouts and as specified herein. Side lap connections shall be screwed or button punched depending on deck profile.
- C. Level and align cellular deck intended for electrical raceways and mechanical piping within 1/8 inch horizontally and vertically. Butt ends, allow for maximum 1/8 inch gap. Install sheet steel covers over gaps wider than 1/8 inch. Tape seal joints watertight.
- D. Arc spot welds (puddle welds) to support shall have a diameter (weld nugget) of 5/8" minimum. Weld metal shall penetrate all layers of deck material at end laps and have adequate fusion to the supporting members. Welding shall be done in accordance with the American Welding Society Standard "Specification for Welding Sheet Steel in Structures", AWS D1.3.
- E. Fastening of deck to supports and side laps.
 - 1. Deck ends at building perimeter: 12" o/c (36/4 min.)
 - 2. Deck end laps: 12" o/c (36/4 min.)
 - 3. Deck sides at building perimeter and deck side laps: Deck units with spans greater than five feet shall be fastened at midspan or at 36" intervals whichever is smaller.
 - 4. See drawings for requirements beyond these minimum requirements.
- F. At ends of decks or where changes of deck direction occur, fasten at 18" o.c. Furnish and install adequate closures and fasten to both sides at 18" o.c.
- G. Cutting of openings through the deck less than 16 square feet in area, and all skew cutting shall be performed in the field.
- H. Install sheet steel strip closures at all floor edge up-turned to thickness of slab to contain wet concrete. Provide closures of sufficient strength to remain in place during concrete placement without distortion.

- I. Install sheet closures and angle flashings to close openings between deck and walls, columns, and openings.
- J. Accessories shall be fastened to supports or deck with mechanical fasteners per manufacturer's recommendations.

3.02 CONSTRUCTION LOADS

- A. Composite steel floor deck units shall serve as a form to support the slab weight and construction loading of 20 psf uniform load.
- B. If heavier construction loads are required, allowable unshored spans shall be reduced accordingly by installation of temporary shoring.
- C. When required, composite steel floor deck units shall be temporarily shored in accordance with the deck manufacturers Shoring Tables. Shoring shall be designed in accordance with applicable local and state building code regulations. Shoring shall remain in place until the concrete flooring attains a minimum of 75% of the concrete design compressive strength and removal is subject to the approval of the Engineer.
- D. Concentrated loads and impact loads during erection and construction shall be avoided. Before the slab is poured, form deck shall be planked in all traffic areas to prevent damage to units.

3.03 CLEAN UP AND FINAL ADJUSTMENTS

- A. Touch up surface coating damage and abrasions using a paint specially formulated for use with galvanized steel. For painted deck use paint compatible with manufacturer's primer.
- B. Installation holes shall be sealed with a closure plate 2 gages thicker than deck and mechanically fastened to deck. Steel deck with holes visible from below will be rejected or sealed. Deck units that are bent, warped, or damaged which would impair the strength and appearance of the deck shall be removed from site.
- C. Steel decking work and accessories, when complete, shall be solid, smooth, and uniform in appearance.
- D. Remove any unused steel deck, edge trimmings, screws, weld washers, butt ends of welding electrodes, and other debris from completed installation.

END OF SECTION

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. Joist and joist gir-

der end moments are always in the form of a force couple. The couple is formed by the force in the chord 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.

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

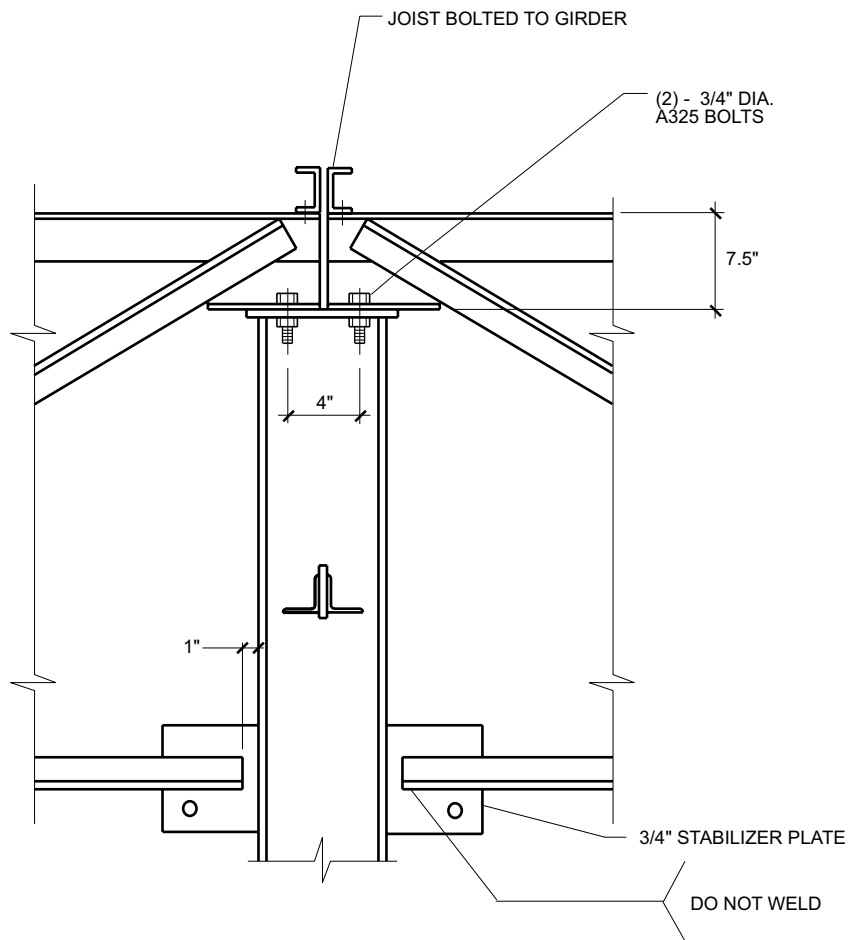


Fig. 7.1.1 The Basic Connection

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:

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.

As mentioned above, these forces must not be ignored in the design. The engineer of record must calculate their magnitude and determine if the Basic Connection can safely support the actual loading.

The engineer of record is responsible for the design of the Basic Connection if it is subjected to any loads other than simple span gravity loadings. The manufacturer of the joists must check the adequacy of the joists and joist girders for the specified end moments created by wind, seismic, or continuity loading. The manufacturer 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. The manufacturer can only determine this if given design documents clearly illustrating the connections. Chapter 6 deals with the proper specification of these forces to the manufacturer.

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 engineer of record to limit the use of the Basic Connection to conditions where the joists and joist girders can be physically designed by the manufacturer to accommodate the intended loads.

The calculations presented below provide the engineer of record 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 engineer of record will also know if special measures are required of the manufacturer 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

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:

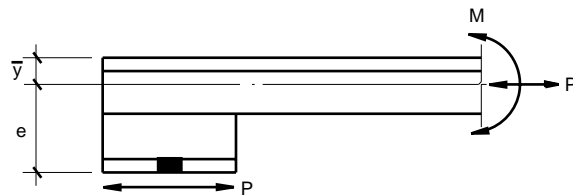


Fig. 7.1.2 Joist Girder Chord Moments

Considering the joist girder seat connection at the column top to be a pin a secondary moment, $M = \pm Pe$ is developed in the top chord. Since 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, $\pm Pe$. Using the standard single bolt-line connection and welding along the seat angle edges, generally, will not be sufficient to develop full rigidity. Calculations and details to accomplish a “fully” rigid 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 (7.5" Seat) Top Chord Leg Size	ASD P_a^* kips	LRFD ϕP_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 Eccentric Top Chord Force For Joists

Examining the Basic Connection shown in Figure 7.1.3 it can be seen that for the axial chord force in the joist to get into the column, the force must pass thru the joist seat, thru the joist girder seat and into the column cap plate.

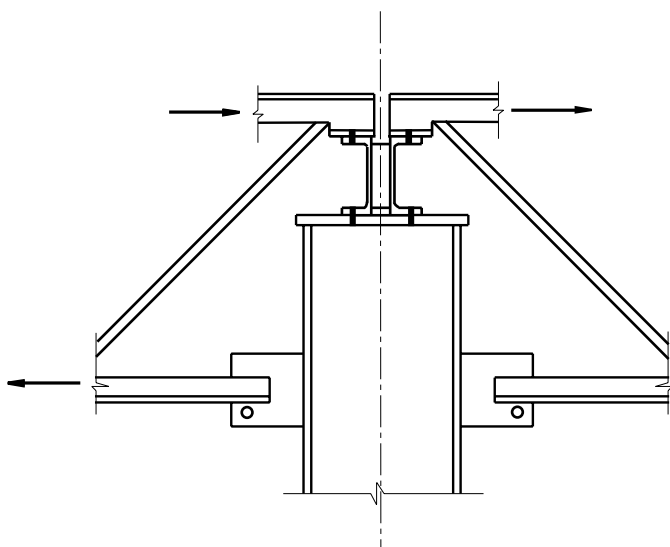


Fig. 7.1.3 Force Transfer

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" then a two hinged mechanism would exist and no lateral force could be resisted. In either case a moment would exist in the top chord of the joist. (It should also be obvious to the reader that the

Basic Connection is not well suited as a moment connection for the joists.)

The moment in the top chord of the joist could be eliminated if both connections were "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 were 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 also require stiffeners to "fix" the joist seat. Performing these measures results in a very uneconomical connection design. However, for small joist 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 rigid connection. 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 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:

- The failure of the joist top chord due to axial load and the chord moment.
- The failure of the joist girder seat, i.e. rollover capacity of the joist girder seat.
- Failure of the welds between the joist seat and the joist girder seat.

The maximum eccentric axial load capacity for a K series joist chord can be determined by finding the allowable axial load and bending moment combination for the top chord angles in the joist.

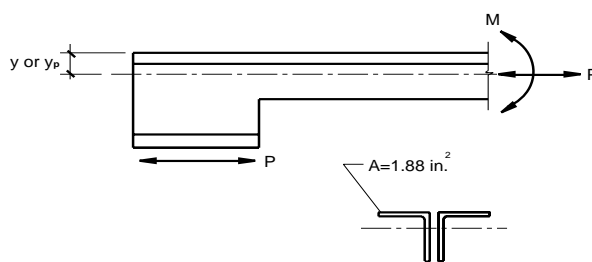


Fig. 7.1.4 Joist Chords

The seat detail is shown in Figure 7.1.4. For this calculation the seat is assumed to be “pinned” at the support. The maximum chord size for a K series joist is two angles 2" x 2" x 1/4". Assuming the deck laterally supports the top chord, and that the maximum effective length of the top chord about its x-x axis is 24 inches, the chord capacity can then be determined from the AISC beam-column interaction equations.

Since the engineer does not know the exact configuration of the joist, and since chord forces due to the horizontal component in the end diagonal also exist, the maximum chord force as developed below can be used only as an upper bound estimate. The engineer of record must indicate the joist chord force requirements on the contract documents.

ASD Solution:

The seat depth is 2.5 inches, thus the moment in the chord equals $P(2.5-0.592) = 1.91P$.

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F'_e}\right) F_b} \leq 1.0 \quad \text{AISC Eq. (H1-1)}$$

$$\frac{f_a}{0.6F_y} + \frac{f_b}{F_b} \leq 1.0 \quad \text{AISC Eq. (H1-2)}$$

where,

$$r_x = 0.609 \text{ in.}$$

$$S_x = 0.494 \text{ in.}^3$$

$$A = 1.88 \text{ in.}^2$$

$$y = 0.592 \text{ in.}$$

$$KL/r_x = 24/0.609 = 39.4$$

$$F_a = 26.04 \text{ ksi} \quad (F_y = 50 \text{ ksi})$$

$$f_a = P/A = P/1.88$$

$$f_b = M/S_x = 1.91P/S_x = 1.91P/0.494 = 3.87P$$

$$F_b = 0.6F_y = 0.6 (50) = 30 \text{ ksi}$$

$$F'_e = 184.4 \text{ ksi;}$$

$$C_m = 0.85 \text{ (reverse curvature assumed)}$$

Solving Equation (H1-1) yields $P \approx 7.5$ kips.

Solving Equation (H1-2) yields $P \approx 6.8$ kips.

Thus, a maximum eccentric joist chord force of 6.8 kips can be used for ASD.

LRFD Solution:

Since the axial load can place the top chord in either tension or compression, to complete the LRFD solution the nominal moment capacity of the double angle top chord must be determined for both stem in compression and stem in tension.

The moment capacity of double angles is limited to $1.5M_y$ when the stem is in tension, since this is a plastic case use of y_p to determine eccentricity of load. The moment capacity of double angles is limited to M_y when the stem is in compression, since this is an elastic case use y to determine eccentricity of load.

When the axial load on the double angle top chord is compressive, the stem of the double angle is in compression. For this case the nominal axial capacity, ϕP_n , is 51 kips and the nominal moment capacity, ϕM_n , is 22.2 in.-kips based on a conservative estimate of the space between the angles of 3/8 inches. The axial load is P and the moment, M , is $(2.5-0.592)P$ or $1.908P$ based on y . Using a value of 0.85 for C_m makes B_1 equal to 1. AISC Equation H1-1a controls to yield a maximum compressive force of 10.4 kips.

When the axial load on the double angle top chord is tensile, the stem of the double angle is in tension. For this case the nominal tensile capacity, ϕP_n , is 84.4 kips and the nominal moment capacity, ϕM_n , is 33.4 in.-kips. The axial load is P and the moment, M , is $(2.5-0.234)P$ or $2.266P$ based on y_p . Since the top chord is in tension, there is no moment magnification. AISC Equation H1-1b controls and yields a maximum tensile force of 13.6 kips.

Thus, a maximum eccentric joist chord force of 10.4 kips can be used for LRFD.

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 calculation of the joist girder seat rollover strength can be accomplished using standard calculation procedures for bolted joints. The effects of welding the seat angles to the supporting element are minimal on the rollover capacity, and can be ignored when stiffeners are not inserted in the seat.

The standard joist girder has a 7.5 inch seat depth. The seat is generally fabricated using 7/16-inch seat angles as shown in Figure 7.1.5.

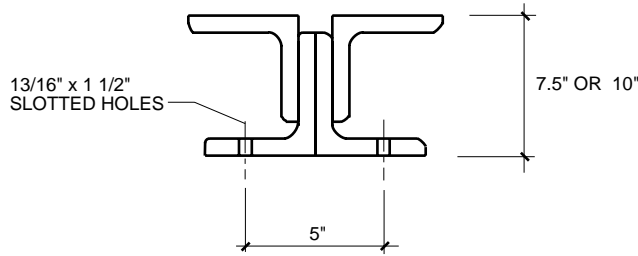


Fig. 7.1.5 Light Joist Girder Seat

The rollover capacity 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 force on the opposite side of the seat. The force system is shown in Figure 7.1.6.

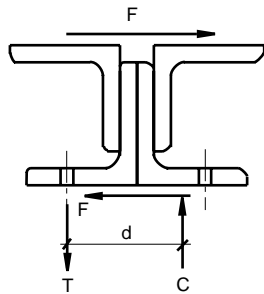


Fig. 7.1.6 Joist Girder Rollover Resistance

The maximum force T is determined from the bending resistance of the seat angle or by the strength of the bolt. If 3/4 inch 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. Using an ultimate strength approach for the moment capacity of the seat angle, the nominal moment capacity M_n equals the plastic modulus of the seat angle times its yield strength:

$$M_n = ZF_y$$

Based on the geometry shown in Figure 7.1.7 the plastic modulus can be calculated as:

$$Z = bt^2/4$$

where,

b = the effective width in bending, taken at the edge of the seat angle fillet, line $a'-a'$

t = seat angle thickness

The effective width b is equal to $2.5d'$, where d' equals the distance from the bolt centerline to the edge of the seat angle fillet. The width b , cannot be larger than the seat angle length B . The factor 2.5 is taken from Reference 39. If an

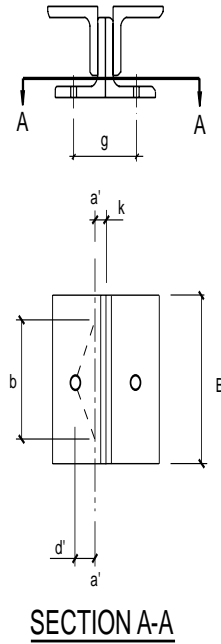


Fig. 7.1.7 Seat Geometry

inflection point is assumed to exist midway between the bolt and the seat angle fillet, then the design strength bolt force T that causes the seat angle to reach M_n can be found as:

$$\phi T = 2 \phi M_n / (d')$$

The compressive force C must equal the tension force T. Conservatively the force C can be assumed to act at a distance $d'/2$ from the seat angle fillet edge. Thus, the distance between T and C can be determined as:

$$d = g - 0.5d'$$

Thus, for the standard 7.5 inch deep joist girder seat the design strength rollover strength ϕF_n can be determined as:

$$\phi F_n = \phi Td/7.5 \text{ (LRFD)}$$

where, $\phi = 0.90$.

For ASD, the allowable rollover force F can be determined as:

$$F = T_a d / 7.5 \text{ (ASD)}$$

where, T_a is equal to T divided by the factor of safety 1.67.

Example 7.1.1 7/16 Inch Seat Angle (Standard Seat)

Determine the rollover resistance for a joist girder seat in which 7/16 inch seat angles are used. Refer to Figure 7.1.7 relative to the position of the seat angles for the determination of d' . Assume $F_y = 50$ ksi and 3/4 inch A325 bolts, and a gage of 5 inches.

Solution:

1. Determine the moment capacity of the seat angle:

$$d' = 0.5g - k = 2.5 - k$$

where $k \approx 7/8$ for the 7/16 inch angles

$$d' = 1.625$$

$$b = 2.5(d') = (2.5)(1.625) = 4.06 \text{ inches}$$

$$Z = bt^2/4 = (4.06)(0.4375)^2/4 = 0.194 \text{ in.}^3$$

$$M_n = ZF_y = (0.194)(50) = 9.70 \text{ in. - kips}$$

2. Determine the design strength (LRFD), and allowable (ASD) bolt force:

$$\phi T_n = 2\phi M_n/d' = (2)\phi(9.70)/1.625 = \phi 11.94$$

$$= (0.9)(11.94) = 10.75 \text{ kips (LRFD)}$$

$$T_a = 11.94/1.67 = 7.15 \text{ kips (ASD)}$$

3. Determine the rollover resistance:

$$d = g - 0.5d' = 5 - (0.5)(1.625) = 4.1875 \text{ in.}$$

$$\text{For LRFD: } \phi F_n = \phi T_n d/7.5 = (10.75)(4.1875)/7.5 = 6.0 \text{ kips.}$$

$$\text{For ASD: } F_a = T_a d/7.5 = 7.15(4.1875)/7.5 = 4.0 \text{ kips}$$

Example 7.1.2 1/4 Inch Seat Angle (Non-standard)

Determine the allowable rollover resistance for a joist girder seat with 1/4 inch thick angles with a bolt gage of 5 inches. F_y of the angles is 50 ksi and 3/4" diameter A325 bolts are used to fasten the seat to the supporting element. Refer to Figure 7.1.7.

Solution:

1. Determine the moment capacity of the seat angle:

$$b = 2.5 d'$$

$$d' = 0.5g - 0.5 - k = 2 - k.$$

where $k \approx 0.625$ for the 1/4 inch angle.

$$d' = 1.375 \text{ inches.}$$

$$b = (2.5)(1.375) = 3.44 \text{ inches.}$$

$$Z = bt^2/4 = (3.44)(0.25)^2/4 = 0.0538 \text{ in.}^3$$

$$M_n = ZF_y = (0.0538)(50) = 2.69 \text{ in. - kips.}$$

2. Determine the design strength (LRFD), and allowable (ASD) bolt force:

$$\phi T_n = 2\phi M_n/d' = (2)\phi(2.69)/1.375 = \phi 3.91$$

$$= 3.52 \text{ kips (LRFD)}$$

$$T_a = 3.91/1.67 = 2.34 \text{ kips (ASD)}$$

3. Determine the rollover resistance:

$$d = g - 0.5d' = 5 - (0.5)(1.375) = 4.3125 \text{ in.}$$

$$\text{For LRFD: } \phi F_n = \phi T_n d/7.5 = (3.52)(4.3125)/7.5 = 2.02 \text{ kips}$$

$$\text{For ASD: } F_a = T_a d/7.5 = 2.34(4.3125)/7.5 = 1.34 \text{ kips}$$

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 engineer of record 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 2-1/2" long on each side of the joist seat is recommended to resist the 6.8 kip joist axial force. These requirements must be considered in greater detail when modifications are made to the Basic Connection.

Summary

For ASD:

1. Joist girder force couples are limited to 4 to 10 kips (per Table 7.1.1) times the distance from the centroid of the bottom chord to the top of the column.
2. The maximum allowable eccentric chord force permitted on a K-series joist (2.5" seat) is 6.8 kips.
3. The maximum lateral shear force (rollover force), F that can be applied perpendicular to a standard joist girder seat (7.5" seat) is 4.0 kips.

For LRFD:

1. Joist girder force couples (ϕP_n) are limited to 6 to 15 kips (per table 7.1.1) 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 (2.5" seat) is 10.4 kips.
3. The maximum lateral shear force (rollover force), ϕF_n that can be applied perpendicular to a standard joist girder seat is 6.0 kips.

7.2 MODIFICATIONS TO THE BASIC CONNECTION

Based on the preceding calculations, it can be seen that only small moments can be transferred to the column using the Basic Connection. Modifications to this connection can be made in order to resist larger moments.

Joist Girder Modifications

Several options exist for the joist girder.

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.

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.

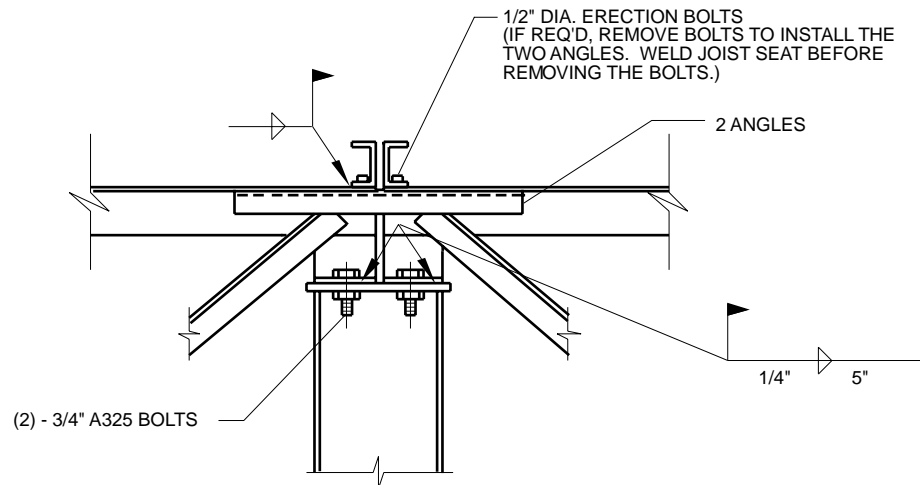


Fig. 7.2.1 Continuity Angles

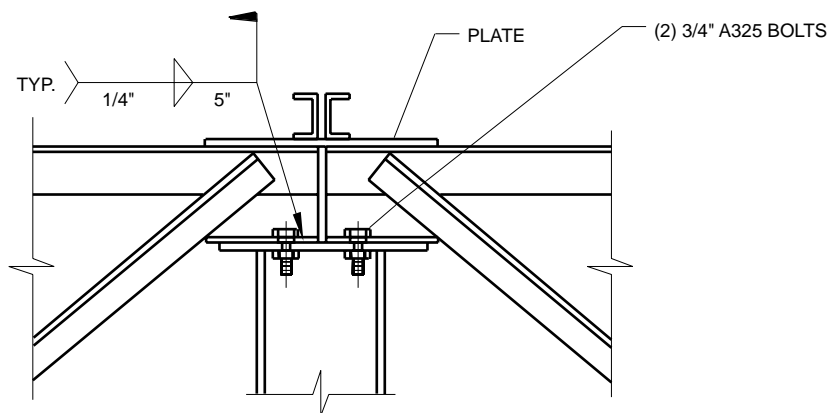


Fig. 7.2.2 Continuity Plates

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 engineer, whereas the modification of the joist girder to accommodate the secondary moment is the responsibility of the manufacturer.

Design of “Fixed” Joist Girder Seats

The attachment of the seat for full rigidity is accomplished most 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.

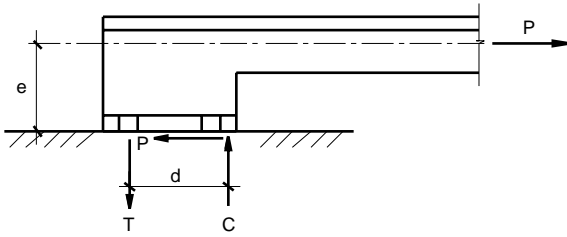


Fig. 7.2.3 Fixed Joist Girder Seat

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 , can be assumed to be 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 Fig. 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 $2.5kL' (0.6F_y)$. L' is determined by setting the compressive force equal to the tension force $2T$, where T is the force in each bolt.

Thus,

$$L' = \frac{2T}{2.5k(0.6F_y)}$$

From Figure 7.2.4 it can be seen that $d = L - 0.5L'$.

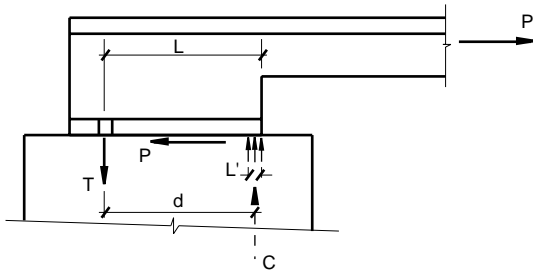


Fig. 7.2.4

To determine (e) the size of the top chord must be known. The specifying engineer can estimate the top chord angle size from Table 7.1.2 and its equations. The procedure is to first calculate the value of the parameter "A" using the equation applicable to the particular joist girder in question. Based on the calculated 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.

A	Minimum Top Chord Width
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: Joist girder chords are always equal legged angles.

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 (ft.)
- D = Joist Girder depth (in.)

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 of 25 kips. Assume the geometry as shown in Figure 7.2.5.

Assume a 40G 8N 12K girder is used. The joist spacing is 5 ft.

ASD Solution:

1. Preliminary Design:

Estimate the top chord size for the 40G 8N 12K girder.

Use Table 7.1.2

$$A = \frac{0.028P}{D} [N^2S - 0.67N + 0.67]$$

$$A = \frac{0.028P}{40} [(8)^2(5) - (0.67)(8) + 0.67] = 2.64$$

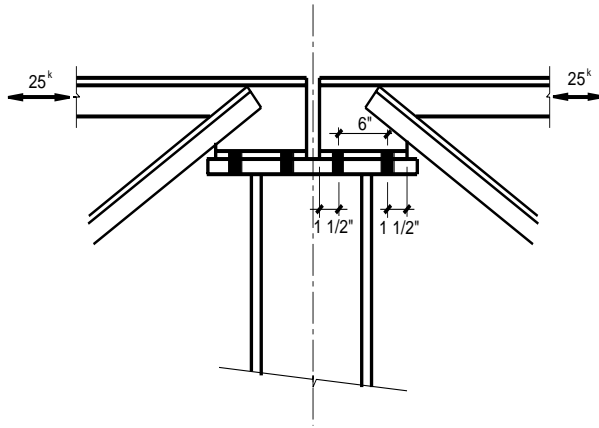


Fig. 7.2.5 Example 7.2.1

From Table 7.1.2 the minimum chord width is 8 inches, thus the chords will likely be 3-1/2 inch angles.

2. Determine the moment resistance:

The moment at the base of the seat equals the force P times its height above the column:

$$M = P(7.5 - \bar{y})$$

where:

\bar{y} = the centroid distance for a 3-1/2 inch angle.

$$M = 25(7.5 - 0.75) = 169 \text{ in. - kips}$$

Based on the rollover calculations made in Example 7.1.1 the allowable bolt tension in a 7/16 inch seat angle is 7.15 kips.

Thus,

$$L' = \frac{2T_a}{(2.5)k(0.6F_y)} = \frac{(2)(7.15)}{(2.5)(0.875)(30)} = 0.218 \text{ in.}$$

From Figure 7.2.5,

$$d = 6 + 1.5 - 0.218/2 = 7.39 \text{ inches}$$

The moment resistance is:

$$M_r = 2Td \text{ (2-bolts)}$$

$$M_r = (2)(7.15)(7.39) = 105.7 \text{ in. - kips} < 169 \text{ req'd}$$

The 7/16 inch seat angles are insufficient to develop the required moment capacity of 169 in. - kips.

To find a solution using 7/16 inch seat angles the distance between bolt holes must be increased. Using 11 inches provides the required strength.

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.

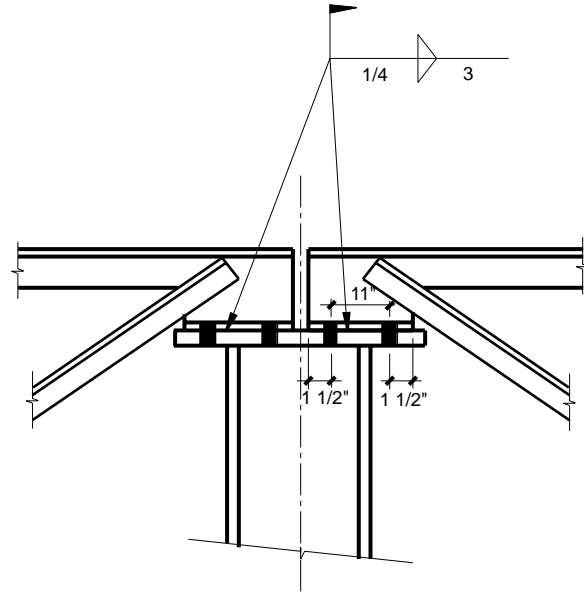


Fig. 7.2.6 Solution Example 7.2.1

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. In either case, the moment created by the eccentric force is resisted by the reinforced chord extension. Both types of extensions are illustrated in Fig. 4.5.4. As mentioned in Section 4.5 a practical eccentric chord force limit is 50 kips for joist girder seat extensions. The engineer of record should clearly indicate locations and force requirements for joist girder E member extensions on the structural drawings.

Joist Modifications

Connection plates can also 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 by using a seat extension on the joist. Vulcraft also refers to the seat extension on a joist as an E member. A typical E member extension is shown in Fig. 7.2.8. In order to take advantage of the increased joist end moment capacity, the joist girder seat must also be stiffened to resist a greater rollover force.

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 in close proximity to the bolts in the seat, the bolt force can be substantially 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 the manufacturer 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 strength of a joist girder seat with stiffeners is illustrated in Example 7.2.2.

Example 7.2.2 Joist Girder Seat Stiffeners

Determine the allowable (ASD) and the design strength (LRFD) transverse shear on the Joist Girder seat shown in Fig. 7.2.7.

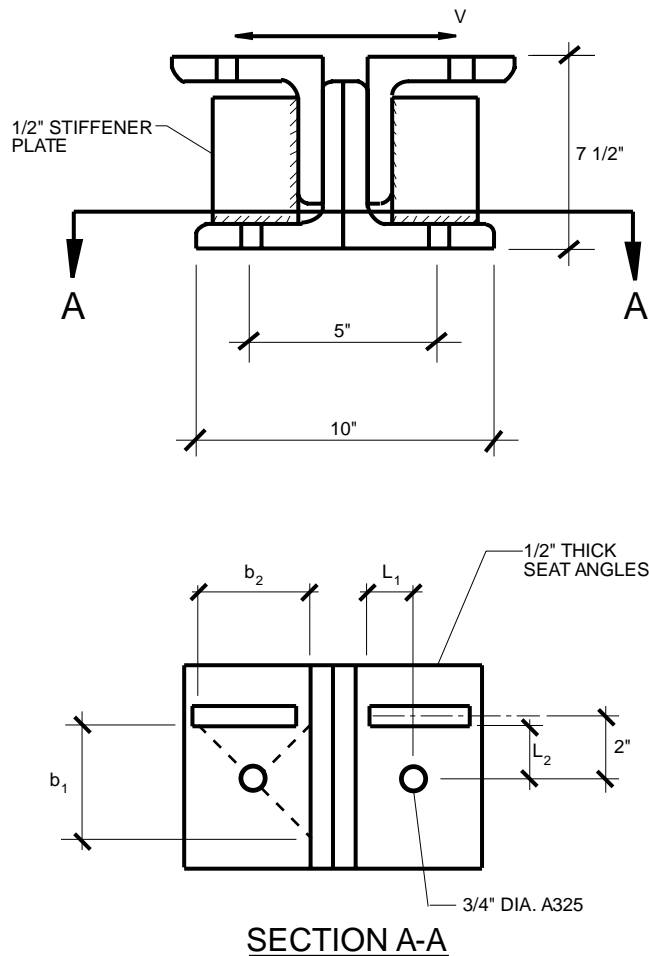


Fig. 7.2.7 Example 7.2.2

ASD Solution:

1. Determine the effective bending length along the 7/16 inch seat angle and along the stiffener:

For the seat angle:

The effective length = $b_1 = 2.5 L_1$, where 2.5 is from Reference 39.

$$L_1 = 2.5 - k = 2.5 - 0.875 = 1.625 \text{ inches.}$$

$$b_1 = 2.5(1.625) = 4.06 \text{ inches.}$$

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.

$$L_2 = 2.0 - t_s/2 - \text{fillet weld size.}$$

Assuming a 1/4 inch fillet weld,

$$L_2 = 2 - 0.25 - 0.25 = 1.5 \text{ inches.}$$

$$b_1/2 = 3.44/2 = 1.72 \text{ inches} > 1.5 \text{ inches.}$$

$$\text{Use } b_1 = 1.5 + 1.72 = 3.22 \text{ inches.}$$

For the stiffener:

$$b_2 = 2.5L_2 = (2.5)(1.5) = 3.75 \text{ inches.}$$

The length of b_2 cannot exceed the width of the stiffener.

$$\text{The stiffener width} = 5'' - t_{\text{seat}} - t_{\text{chord}}$$

$$\text{Based on a } 3/4 \text{ in. chord: } 5'' - 0.4375'' - 0.75'' = 3.8125$$

$$3.75 < 3.8125 \quad \text{o.k.}$$

2. Determine the plastic moment capacity along the 7/16 inch seat angle and along the stiffener:

$$M_p = ZF_y, \text{ where } Z = bt^2/4$$

Along the seat angle:

$$Z = (3.22)(0.4375)^2/4 = 0.154$$

$$M_p = F_y Z = (50)(0.154) = 7.70 \text{ in.-kips.}$$

Along the stiffener:

$$Z = (3.75)(0.4375)^2/4 = 0.179.$$

$$M_p = F_y Z = (50)(0.179) = 8.95 \text{ in.-kips.}$$

3. Determine the allowable bolt force:

The maximum 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 .

$$\text{The maximum bolt force } T_{\text{max}} = 7.70/(L_1/2) + 8.95/(L_2/2).$$

$$T_{\text{max}} = 9.48 + 11.93 = 21.41 \text{ kips.}$$

$$\text{Using a safety factor of 1.67, } T_a = 21.41/1.67 = 12.82 \text{ kips.}$$

T_a must be less than the AISC allowable bolt tension.

$$\text{The allowable tension} = 19.4 \text{ kips.} \quad \text{o.k.}$$

Note: Prying forces are generally neglected when bolts are located in re-entrant corners as shown in Fig. 7.2.7.

4. Determine the allowable rollover shear:

Conservatively the compression force can be assumed to act at the center of the opposite bolt.

The allowable resisting moment .

$$M_r = 5T_a = (5)(12.82) = 64.1 \text{ in. - kips.}$$

The maximum shear V , equals M_r divided by the seat depth.

$$V = 64.1/7.5 = 8.55 \text{ kips.}$$

If the seat is not welded to the column cap then 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:

$$F_t = \sqrt{(44)^2 - 2.15f_v^2}.$$

Since two bolts resist the shear,

$$f_v = V/(\text{bolt area})$$

$$f_v = 8.55/(2 \times 0.44) = 9.72 \text{ ksi}$$

$$f_t = T_a/(\text{bolt area})$$

$$f_t = 12.82/0.44 = 29.14 \text{ ksi}$$

Solving for $F_t = 41.63 \text{ ksi.}$

$$29.14 < 41.63 \quad \text{o.k.}$$

The shear in the bolts could be eliminated by welding the seat angle to the column cap.

5. Determine the stiffener weld requirements:

Force in stiffener equals the percent of the bolt load that goes into the stiffener. This can be determined based on the proportion of moment that goes into the b_2 length.

$$\text{Stiffener force} = T_a [8.95/(8.95+7.70)] = 0.54 T_a$$

$$\text{Force in stiffener} = (0.54)(12.82) = 6.92 \text{ kips}$$

Welding on one side only:

Fillet size:

$$D(0.707)(21)(3.75) = 6.92 \text{ kips.}$$

$$\therefore D = 0.124''; \text{ Use } 1/4 \text{ inch fillet weld.}$$

6. Check stiffener size:

$$\text{Force in stiffener} = 6.92 \text{ kips}$$

$$f_a = \text{Force in stiffener}/\text{Area of stiffener.}$$

$$f_a = 6.92/(0.5 \times 3.8125) = 3.63 \text{ ksi} < 22 \text{ ksi.} \quad \text{o.k.}$$

∴ The allowable transverse shear = 8.55 kips

LRFD Solution

Steps 1. and 2. are the same as the ASD solution.

3. The bolt design strength $\phi T_n = \phi[7.70/(L_1/2) + 8.95/(L_2/2)]$.

$$\text{Using } \phi = 0.9, \phi T_n = (0.9)(21.41) = 19.27 \text{ kips}$$

The 3/4 in. dia. bolt design tensile strength = 29.8 kips.
T must be less than the AISC bolt tensile strength
19.27 kips < 29.8 kips **o.k.**

Prying forces are again neglected as per the ASD solution.

4. Determine the design strength rollover shear:

The design resisting moment.

$$M = 5T = (5)(19.27) = 96.35 \text{ in. - kips}$$

The design strength shear V, equals M divided by the seat depth.

$$V = 96.35/7.5 = 12.85 \text{ kips}$$

If the seat is not welded to the column cap then the bolts must be checked for combined tension and shear. Using the AISC Specification for threads excluded from the shear plane for bearing type bolts the allowable bolt tension stress equals:

$$F_t = 117 - 1.5 f_v \leq 90$$

Since two bolts resist the shear,

$$f_v = V/(\text{bolt area})$$

$$f_v = 12.85/(2 \times 0.44) = 14.60 \text{ ksi}$$

$$f_t = T/(\text{bolt area}) = 19.27/0.44 = 43.8 \text{ ksi}$$

Solving for $F_t = 95.10 \text{ ksi}$

$$43.8 < 95.10 \text{ **o.k.**}$$

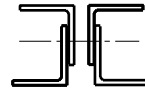
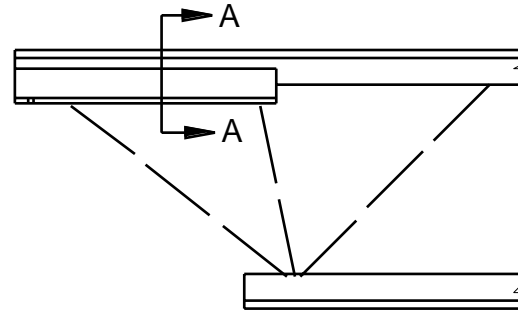
∴ Design strength transverse shear = 12.85 kips

Steps 5. and 6. are not repeated.

Increasing Joist Chord Capacity with Seat Extensions

A seat extension detail (E member) for a standard joist is shown in Fig. 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.



SECTION A-A

Fig. 7.2.8 E Member Extension

The E member is designed by the manufacturer; however, the manufacturer 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 engineer know whether or not the E member solution is feasible.

By examining the maximum chord size for a K joist (2x2x1/4") with an E member extension, an upper bound on the use of an E member can be obtained. The solution is provided below.

The properties of an extended seat for a K12 joist are shown in Figure 7.2.9.

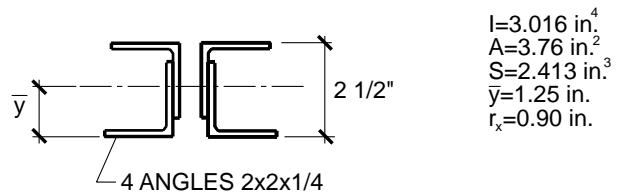


Fig. 7.2.9 Extended Seat for a K12 Joist

The moment in the extended seat equals the force, P, at the bottom of the seat, times \bar{y}

$$M = 1.25 P$$

The AISC combined bending and axial load equations can be solved in order to determine the maximum allowable force P.

$$f_a = P/3.76 ; f_b = 1.25P/2.413$$

In order to determine F_a and F_e' , an unbraced length of the chord must be determined. If the deck is assumed to laterally brace the chord about its y-y axis, then F_a and F_e' are based on the x-x properties. For a K series joist, the maximum unbraced length of the chord is approximately 48 inches, and assuming $K=1.0$, $KL/r_x = (1)(48)/0.9 = 53.3$. Thus, $F_a = 23.8$ ksi and $F_e' = 52.5$ ksi. C_m is equal to 0.85, and $F_b = 30$ ksi. Solving the interaction equations for P allowable yields, $P = 38.3$ kips. Thus, the maximum eccentric force allowed on a K series joist with an E member extension is approximately 38 kips (ASD).

Checking the joist chord capacity beyond the seat extension, using an unbraced length of 24 inches, yields an allowable concentric axial load of approximately 49 kips based on $KL = 24$ inches. Therefore the extended seat capacity controls the design, not the chord capacity itself. Since the end panel geometry may be slightly different than that assumed above, the engineer of record should not arbitrarily use an E member extension without notifying the manufacturer of the force requirements.

Repeating the above example using the AISC LRFD Specification:

Assume the ratio of $P_u/\phi P_n \geq 0.2$ and equation H1-1a governs:

$$\frac{P_u}{\phi P_n} + \frac{8}{9} \left(\frac{M_{ux}}{\phi M_{nx}} + \frac{M_{uy}}{\phi M_{ny}} \right) \leq 1.0$$

where $M_{uy} = 0$ and $M_{ux} = B_1 M_{ntx}$ with $M_{ntx} = 1.25P$

C_m is equal to 0.85, assume $B_1 = 1$

Find ϕP_n :

Let $KL = 48$ in.; $KL/r_x = 48/0.90 = 53.33$
and $\phi F_{cr} = 34.52$ ksi

$$\phi P_n = A \phi F_{cr} = (3.76)(34.52) = 129.8 \text{ kips}$$

Find ϕM_{nx} :

Check $L_b < L_p$; $L_b = 48$ in.

$$L_p = \frac{300r_y}{\sqrt{F_{yf}}} = \frac{300(1.366)}{\sqrt{50}} = 57$$

$$L_b < L_p; \phi M_n = \phi M_y = \phi S_x F_y = (0.9)(2.474)(50) = 111.33 \text{ in.-kips}$$

Equation H1-1a:

$$\frac{P_u}{129.8} + \frac{8}{9} \left(\frac{1.25P_u}{111.33} \right) = 1$$

Solve for P_u : $P_u = 56.55$ kips

Check assumptions:

$$P_u/\phi P_n = 56.55/129.8 = 0.44 > 0.2 \quad \text{o.k.}$$

$$B_1 = \frac{C_m}{1 - \frac{P_u}{P_{el}}} \geq 1$$

$$C_m = 0.85;$$

$$P_{el} = \frac{AF_y}{\left(\frac{KL}{r_x \pi} \sqrt{\frac{F_y}{E}} \right)^2} = \frac{(3.76)(50)}{\left(\frac{48}{(0.90)\pi} \sqrt{\frac{50}{29000}} \right)^2} = 378.3$$

$$B_1 = \frac{0.85}{1 - \frac{56.55}{378.3}} = 1.0 \leq 1.0 \quad \text{o.k.}$$

$$P_u = 56.55 \text{ kips}$$

Checking the joist chord capacity beyond the seat extension yields a design ultimate load of 71 kips based on $KL = 24$ inches.

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.

ASD:

For ASD calculations, the seat moment capacity can be based on the allowable bolt tension, T_a

For 7/16 inch seat angles $T_a = 7.15$ kips.

For 7/16 inch stiffened seat angles $T_a = 12.82$ kips.

Using 1/2 inch seat stiffeners with 7/16 inch seat angles can increase the lateral shear force (rollover force) of a joist girder seat to 8.55 kips.

E member extensions can be used on joists to increase the eccentric force capacity up to 38 kips.

LRFD:

For LRFD calculations, the seat moment capacity can be based on the design strength bolt tension, ϕT_n .

For 7/16 inch seat angles $\phi T = 10.75$ kips

For 7/16 inch stiffened seat angles $\phi T = 19.3$ kips

Using 1/2 inch seat stiffeners with 7/16 inch seat angles can increase the lateral shear force (rollover force) of a joist girder seat to 12.85 kips.

E member extensions can be used on joists to increase the eccentric force capacity up to 56.6 kips.

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. 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 engineer must perform a structural analysis in order to determine the forces in the bottom chord. Comments regarding rigid frame analysis are made in Chapter 4. The specification of these forces to the manufacturer is discussed in Chapter 6. It is the responsibility of the manufacturer to design the bottom chords of the joists and girders for the specified forces; however, it is the responsibility of the engineer to design the connection to the column. The engineer 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 manner in which 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. 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 engineer can calculate the approximate rod extension force. In general, the rod length must be at least 2-1/2 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 engineer at the time he must design the connection, the design must 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. The gap between joist bottom chords will vary between 1/2 of an inch to 1 inch. The space between the bottom chords for joist girders is typically one inch. The use of 3/4 inch stabilizer plates is 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, thus a 7/8 inch plate may be preferable. 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 $\pm 1/2$ 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 tube 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.

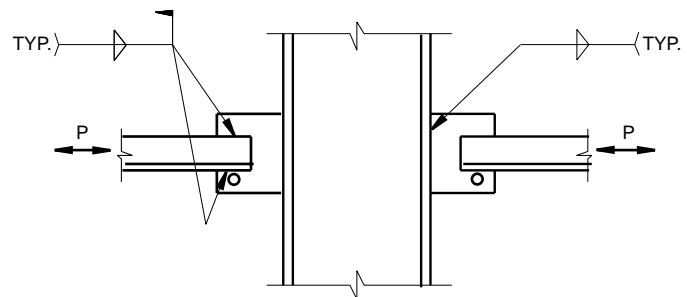


Fig. 7.3.1 Bottom Chord Attachment to Flanges

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 buckling. The AISC specification does not address this specific geometrical situation; however, basic principals 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 $5k$.
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 Fig. 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.

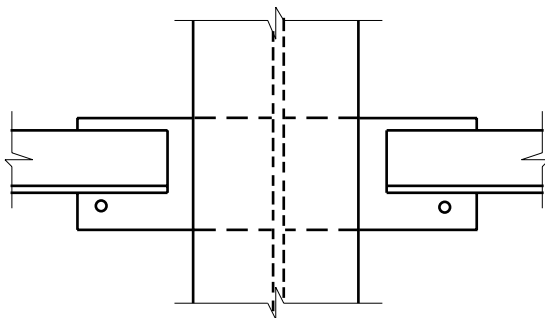


Fig. 7.3.2 Framing to Column Web

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.

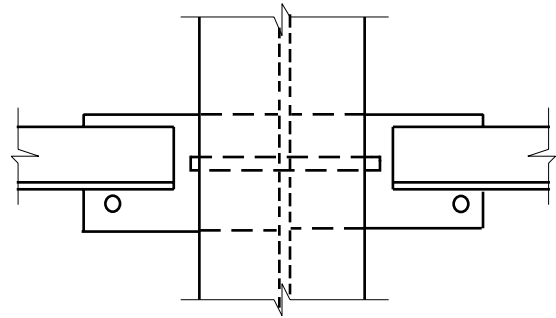


Fig. 7.3.3 Stiffener Reinforcement to Stabilizer Plates

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. The manufacturer 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 end of the bottom chord and 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 a 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.

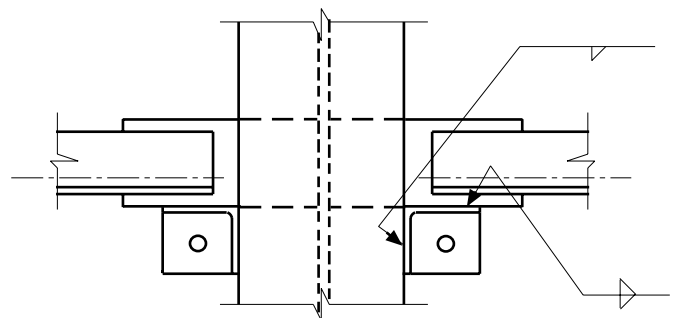


Fig. 7.3.4 Angle Reinforcement to Stabilizer Plates

The design of a bottom chord system is given in Example 7.3.1.

Example 7.3.1 Bottom Chord Force Transfer

Design the bottom chord connection shown in Figure 7.3.5 for the load cases shown:

Given:

Load Cases:

Case 1 $P_1 = 100K$ $P_2 = -100K$

Case 2 $P_1 = -125K$ $P_2 = -75K$

Case 3 $P_1 = 75K$ $P_2 = 125K$

Assume that the column is a W10x33, $F_y = 50$ ksi, A36 steel is used for all plate material.

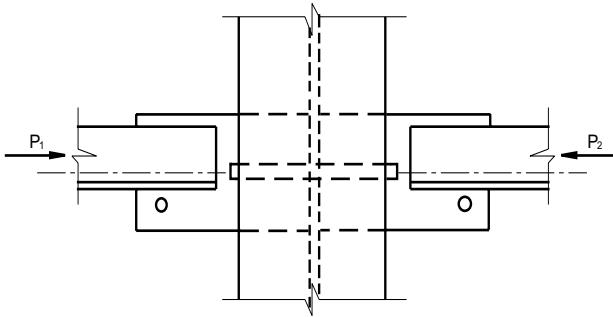


Fig. 7.3.5 Example 7.3.1

ASD Solution:

1. Design the stabilizer plate and the chord welds:

The stabilizer plate and stabilizer-to-chord welds must be designed for a 125 kip force (Cases 2 and 3).

Stabilizer Plate:

Assume 7/8" plate.

$$F_a = 22 \text{ ksi}$$

$$A_{\text{req'd}} = P/F_a$$

$$A_{\text{req'd}} = 125/22 = 5.68 \text{ in.}^2$$

$$\text{Height} = A_{\text{req'd}}/\text{thickness}$$

$$\text{Height} = 5.68/0.875 = 6.5 \text{ in.}$$

∴ Use 8 inch plate

Weld Requirements:

Chord to Stabilizer Plate:

$$P = 125 \text{ kips}$$

Try 4 welds 5/16":

$$\text{Length req'd} = P/\text{weld strength}$$

$$\text{Length req'd} = 125/(4 \times 0.707 \times 21 \times 0.3125) = 6.73"$$

∴ Use 4 - 5/16" Fillet welds 7" long

Also use 5/16 in. fillet welds to connect the stabilizer to the column web. (Based on the 7/8 in. thick stabilizer plate.)

2. Design of the force transfer stiffeners:

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 1/2 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.

Stress in stiffeners:

$$f_v = 12.5/(0.5 \times 4) = 6.25 \text{ ksi} < 0.4 F_y < 14.5 \text{ ksi} \quad \text{o.k.}$$

Weld stress:

$$f_v = 12.5/(0.3125 \times 0.707 \times 4) = 1.14 \text{ ksi} < 21 \text{ ksi} \quad \text{o.k.}$$

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.

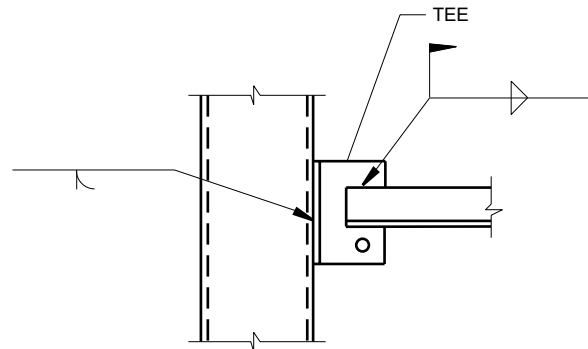


Fig. 7.3.6 T-Reinforcement

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.

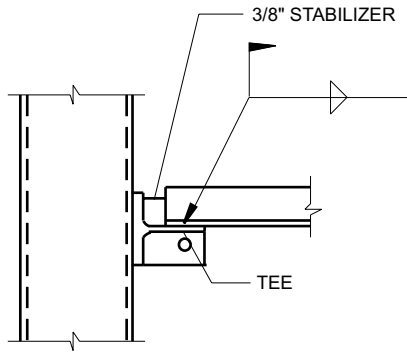


Fig. 7.3.7 T-Reinforcement with Stabilizer

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.

Typical Connection 7.4.1

Determine the maximum permissible joist girder chord force for the detail shown in Figure 7.4.1 using a W8x24 column.

ASD Solution:

1. A special seat permits an allowable top chord force of 30 kips.
2. Bottom chord connection:
Chord to stabilizer plate weld stress:

$$f_s = \frac{30}{(0.707)(3/16)(4)(3)} = 18.9 \text{ ksi} < 21 \text{ ksi} \quad \text{o.k.}$$

Stabilizer plate stress:

$$f_a = P/A = 30/(0.75 \times 6) = 6.66 \text{ ksi} \quad \text{o.k.}$$

Stress in weld of stabilizer plate to column flange:

$$f_s = \frac{30}{(0.707)(5/16)(2)(6)} = 11.3 \text{ ksi} < 21 \text{ ksi} \quad \text{o.k.}$$

Local web yielding: AISC Eq. (K1-2)

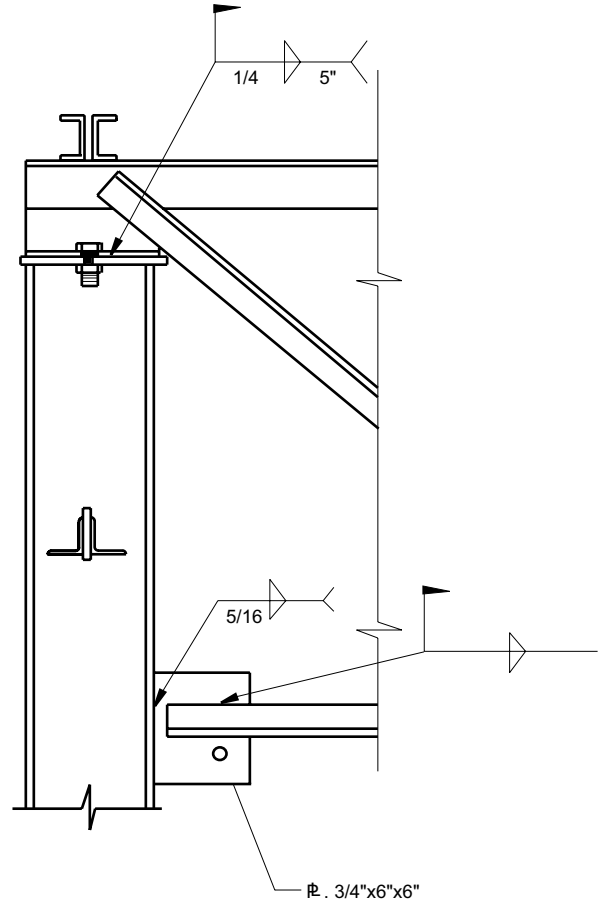


Fig. 7.4.1 Typical Connection 7.4.1

$$\frac{R}{t_w(N + 5k)} \leq 0.66F_y$$

where:

R = concentrated load, kips

t_w = thickness of web, in.

N = length of bearing, in.

k = distance from outer face of flange to web toe of fillet, in.

Solving:

$$\frac{30}{(0.245)(6 + 5 \times 0.875)} = 11.81 < 24 \text{ ksi} \quad \text{o.k.}$$

Web Crippling: AISC Eq. (K1-4)

$$R = 67.5t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw}t_f}{t_w}}$$

where:

F_{yw} = specified minimum yield stress of beam web, ksi

d = overall depth of the member, in.

t_f = flange thickness, in.
 t_w = thickness of the web, in.

Solving:

$R =$

$$67.5(0.245)^2 \left[1 + 3 \left(\frac{6}{7.93} \right) \left(\frac{0.245}{0.400} \right)^{1.5} \right] \sqrt{\frac{(36)(0.400)}{0.245}}$$

$$= 64.9 > 30 \text{ kips} \quad \text{o.k.}$$

Therefore, the connection shown is adequate for a 30 kip joist girder force couple.

LRFD Solution:

1. A special seat permits a design strength of 45 kips.
2. Bottom chord connection:

Check column web:

Web Yielding

$$\begin{aligned} \phi R_n &= \phi(5k+N)F_{yw}t_w \\ &= (1.0)(5(0.875)+6)(36)(0.245) \\ &= 91.5 \text{ kips} \end{aligned}$$

Web Crippling:

$$\phi R_n = \phi 135 t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\phi R_n = \phi 135 t_w^2 [\alpha] \sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\alpha = \left[1 + 3 \left(\frac{6}{7.93} \right) \left(\frac{0.245}{0.400} \right)^{1.5} \right] = 2.088$$

$$\phi R_n = (0.75) 135 (0.245)^2 (2.088) \sqrt{\frac{36(0.400)}{0.245}}$$

$$\phi R_n = \phi 135 t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\phi R_n = 97.3 \text{ kips}$$

Therefore the connection is adequate for a 45 kip joist girder force couple.

Typical Connection 7.4.2

Determine the maximum joist chord force for the connection shown in Figure 7.4.2. Assume two joist girders frame into the column top. The column is a HSS 8"x8"x1/4".

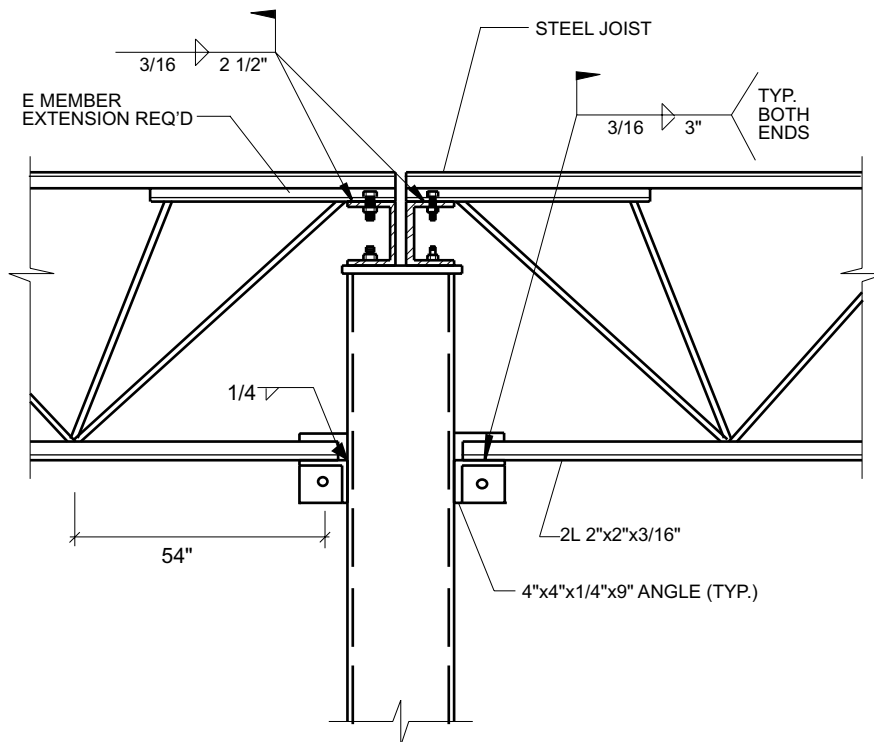


Fig. 7.4.2 Typical Connection 7.4.2

ASD Solution:

1. The rollover capacity for a joist girder seat with 7/16 inch seat angles is 4.0 kips (ASD). (See Example 7.1.1). Therefore each joist top chord force could equal 4.0 kips (since two joist girder seats are present). The weld capacity connecting the joist to the joist girder seat is:

$$P = (0.188)(0.707)(21)(2)(2.5) = 13.96 \text{ kips.}$$

2. Bottom chord connection:

Tension capacity of (2)-2x2x3/16 inch angles:

$$P = A F_t = (1.43)(30) = 42.9 \text{ kips}$$

Compression capacity (length equals 54"):

$$r = 0.617; L/r = 87.5$$

Assuming fixed ends:

$$K = 0.65; KL/r = 56.9$$

$$F_a = 23.24 \text{ ksi}$$

$$P = A F_a = (1.43)(23.24) = 33.23 \text{ kips.}$$

Capacity of the (2)-3 inch long 3/16 in. fillet welds:

$$\text{Capacity} = (21)(6)(3/16)(0.707) = 16.7 \text{ kips}$$

Bending capacity of the 4x4x1/4 inch angle:

Assume only horizontal leg resists flexure.

$$S_x = (0.25)(4)^2/6 = 0.667 \text{ in.}^3$$

$$M = S_x F_b$$

$$M = (0.667)(22) = 14.4 \text{ in. - kips.}$$

From $M = PL/4$:

$$P = 4M/L$$

$$P = (4)(14.4)/8 = 7.2 \text{ kips.}$$

Determine the allowable force that can be delivered to the face of the HSS:

From the AISC "Specification for the Design of Steel Hollow Structural Sections" the nominal (the specification is written only for LRFD) concentrated force distributed transversely to the face of a rectangular HSS is:

$$R_n = 10F_y t b_1 / (B/t) \leq F_{y1} t_1 b_1$$

$$\phi = 1.0$$

where:

F_y = the specified yield strength of the HSS

t = the design wall thickness of the HSS

b_1 = the width of the loaded plate

B = the width of the HSS

F_{y1} = the specified yield strength of the plate

t_1 = the thickness of the plate

Thus,

$$R_n = (10)(46)(0.233)(7.0)/(8/0.233) = 21.85 \text{ kips}$$

Using a factor of safety of 2.0, the allowable load is 10.93 kips.

3. Determine the maximum chord forces:

Top chord: = 4.0 kips - rollover.

Bottom chord: 33.4 kips - controlled by the welds.

4. Maximum end moment:

The maximum end moment based on the joist girder seat equals 4.0 times (the joist depth - the joist seat depth).

LRFD Solution:

1. The rollover capacity for a joist girder seat with 7/16 inch seat angles is 6.0 kips. (See Example 7.1.1.) Therefore each joist top chord force could equal 6 kips (since two joist girder seats are present).

The weld capacity connecting the joist to the joist girder seat is based on the ultimate stress of the fillet weld which for E70XX electrodes is:

$$\phi F_n = (0.75)(0.60)(70) = 31.5 \text{ ksi}$$

Therefore, for the 3/16 inch fillet welds, the total weld capacity is

$$\phi P_n = (0.707)(0.1875)(31.5)(2)(2.5) = 20.9 \text{ kips}$$

2. Bottom chord connection:

Tension capacity:

$$\phi P_n = \phi A F_y = 0.9(1.43)(50) = 64.4 \text{ kips}$$

Compression capacity (length equals 54"):

$$r = 0.617; L/r = 87.5$$

Assuming fixed ends:

$$K = 0.65; KL/r = 56.9$$

$$\phi F_{cr} = 33.53 \text{ ksi}$$

$$\phi P_n = (1.43)(33.53) = 47.95 \text{ kips}$$

Capacity of the (2)-3 inch long 3/16 inch fillet welds:

$$\phi P_n = (2)(3)(3/16)(0.707)(31.5) = 25.05 \text{ kips}$$

Bending capacity of the 4x4x1/4 inch angle:

Assume only horizontal leg resists flexure and use plastic moment capacity.

$$Z_x = bt^2/4 = (0.25)(4)^2/4 = 1 \text{ in.}^3$$

$$\phi M_n = (0.9)(1)(36) = 32.4 \text{ in.-kips}$$

From $\phi M_n = P_u L/4$:

$$P_u = 4M/L$$

$$P_u = (4)(32.4)/8 = 16.2 \text{ kips}$$

Determine the nominal force that can be delivered to the face of the HSS.

From the ASD solution:

$$R_n = 21.85 \text{ kips}$$

3. Determine the maximum chord force of the effective joist:

The maximum tension force in the bottom chord is controlled by the bottom chord connection angle connecting the bottom chord rod extensions: $P_u = 16.2$ kips.

The maximum compression force in the top chord is controlled by the rollover capacity of the joist girder seats ($P_u = 6$ kips).

4. Maximum end moment:

The maximum end moment equals the maximum chord force times the distance between the force couple (joist depth - joist seat depth).

For the above example, the maximum chord force equals 6 kips.

Typical Connection 7.4.3

Determine the joist chord force (ASD allowable, LRFD nominal) for the connection shown in Figure 7.4.3. Assume two joist girders frame into the column top. The column is an HSS 8"x8"x1/4". A full bottom chord extension is used on the joist.

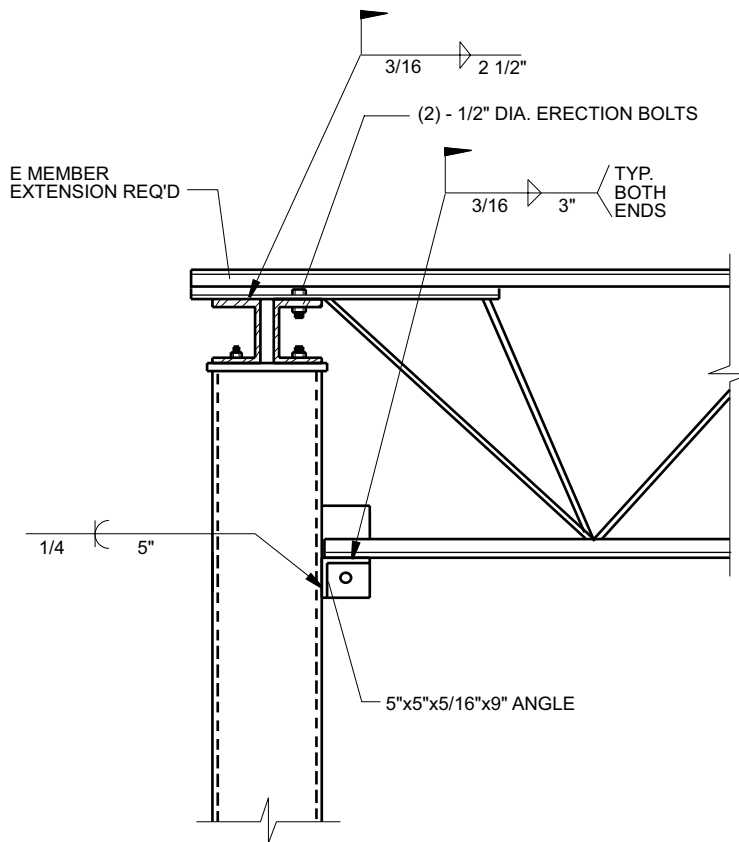


Fig. 7.4.3 Typical Connection 7.4.3

ASD Solution:

1. Top chord connection:

Based on the calculations made in Example 7.4.2, the capacity can be based on the rollover capacity of the joist girder seats. The total rollover capacity available = $(2)(4.0) = 8.0$ kips.

2. Bottom chord connection:

Weld capacity, chord to angle: $(0.188)(0.707)(21)(6) = 16.75$ kips.

Determine the 5x5x5/16 angle capacity:

Assume only horizontal leg resists flexure.

$$S_x = 1.30 \text{ in.}^3$$

$$M = S_x F_b$$

$$M = (1.30)(22) = 28.65 \text{ in. - kips}$$

From $M = PL/4$:

$$P = 4M/L$$

$$P = (4)(28.65)/8 = 14.32 \text{ kips}$$

Determine the allowable force that can be delivered to the face of the HSS:

From Example 7.4.2: 10.93 kips

Maximum chord force equals 8.0 kips. (Joist girder seat rollover governing.)

LRFD Solution:

1. The rollover capacity for a joist girder seat with 7/16 inch seat angles is 6.0 kips. (See Example 7.1.1.) Therefore the joist top chord force could equal 12.0 kips (since two joist girder seats are present).

The weld capacity connecting the joist to the joist girder seat is based on the ultimate stress of the fillet weld which for E70XX electrodes is:

$$\phi F_n = (0.75)(0.60)(70) = 31.5 \text{ ksi}$$

Therefore, for the 3/16 fillet welds, the total weld capacity is

$$\phi P_n = (0.707)(0.1875)(31.5)(2)(2.5) = 20.9 \text{ kips}$$

Note the rollover capacity controls the maximum force in the top chord.

2. Bottom chord connection:

Tension capacity of 2-2x2x3/16 inch angles:

$$\phi P_n = \phi A F_y = 0.9(1.43)(50) = 64.4 \text{ kips}$$

Compression capacity of 2-2x2x3/16 inch angles (length equals 48"):

Assuming pinned ends and adequate intermediate connectors:

$$\phi P_n = 39 \text{ kips}$$

Capacity of the (2)-3 inch long 3/16 fillet welds:

$$\phi P_n = (0.707)(0.1875)(31.5)(2)(3) = 25.0 \text{ kips}$$

Bending capacity of the 5x5x5/16 inch angle:

Assume only horizontal leg resists flexure and use plastic moment capacity.

$$Z_x = bt^2/4 = (0.3125)(5)^2/4 = 1.95 \text{ in.}^3$$

$$\phi M_n = (0.9)(1.95)(36) = 63.3 \text{ in.-kips}$$

From $\phi M_n = P_u L/4$:

$$P_u = 4M/L$$

$$P_u = (4)(63.3)/8 = 31.6 \text{ kips}$$

Determine the nominal force that can be delivered to the face of the HSS:

From Example 7.4.2: $R_n = 21.85 \text{ kips}$

3. Determine the maximum chord force of the joist:

The maximum tension force in the bottom chord is controlled by the force on the HSS face: $P_u = 21.85 \text{ kips}$.

The maximum force in the top chord is controlled by the rollover capacity of the joist girder seats: $P_u = 12.0 \text{ kips}$. This force controls the maximum joist end moment.

4. Maximum end moment:

The maximum end moment equals the maximum chord force times the distance between the force couple (joist depth - joist seat depth).

For the above example, the maximum chord force equals 12.0 kips.

Note, the maximum top chord compression force in the above example exceeds the maximum eccentric compressive force capacity of the top chord of the joist ($P_u = 10.4 \text{ kips}$). In order to reach the maximum force based on joist girder seat rollover capacity, an E member extension would be required on the joists.

Typical Connection 7.4.4

Determine the joist chord force for (ASD allowable, LRFD nominal) the connection shown in Figure 7.4.4. Assume two joist girders frame onto the column top. The column is a W8x24.

ASD Solution:

1. Top chord connection:

Rollover capacity of a stiffened seat with 7/16" seat angles:

From Example 7.2.1 the rollover capacity is 8.55 kips. For two seats 17.1 kips.

Weld capacity = $(0.188)(0.707)(21)(10) = 27.91 \text{ kips}$.

2. Bottom chord connection:

Welds from chord to stabilizer:

$$P = (0.188)(0.707)(21)(12) = 33.5 \text{ kips.}$$

Allowable stabilizer axial load:

$$P = (0.75)(6)(22) = 99 \text{ kips.}$$

Weld of stabilizer to column flange.

$$P = (0.3125)(0.707)(21)(12) = 55.68 \text{ kips.}$$

Check the column web:

CONNECTION DESIGN

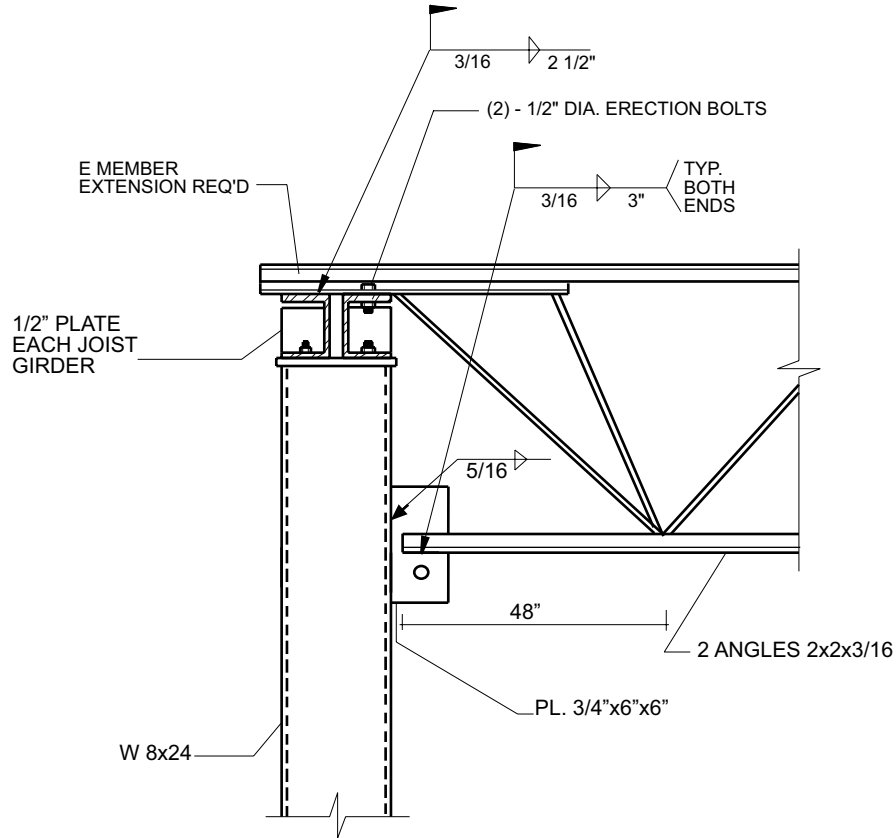


Fig. 7.4.4 Example 7.4.4

Maximum reaction for web yielding:

$$\begin{aligned} R &= 0.66 F_y [t_w(N+5k)] \\ &= 24[0.245(6+5 \times 0.875)] \\ &= 61 \text{ kips} \end{aligned}$$

Maximum reaction for web crippling = 64.9 kips

(From Typical Connection 7.4.2)

3. The maximum chord force = 22.4 kips (based on joist girder seat rollover capacity).
4. The chord force must be specified to the joist manufacturer. The E member extension, and the seat stiffeners should be shown on the structural drawings.

LRFD Solution:

1. Top chord connection:
From Example 7.2.2 the rollover capacity of a stiffened seat is 12.85 kips. For two seats the rollover capacity is 25.7 kips.

Weld capacity:

$$\phi P_u = (0.707)(0.1875)(31.5)(2)(5) = 41.8 \text{ kips}$$

2. Bottom chord connection:

From Example 7.4.3 the maximum bottom chord force is 32 kips in compression.

Chord to stabilizer plate weld capacity:

$$\phi P_n = (0.707)(0.1875)(31.5)(4)(3) = 50.1 \text{ kips}$$

Stabilizer plate capacity:

Assume compression yielding controls.

$$\phi P_n = \phi A F_y = (0.85)(0.75)(6)(36) = 137.7 \text{ kips}$$

Weld of stabilizer plate to column flange:

$$\phi P_n = (0.707)(0.3125)(31.5)(2)(6) = 83.5 \text{ kips}$$

Check column web:

Web Yielding

$$\begin{aligned} \phi R_n &= \phi(5k+N)F_{yw}t_w \\ &= (1.0)(5(0.875)+6)(36)(0.245) \\ &= 91.5 \text{ kips} \end{aligned}$$

Web Crippling:

$$\phi R_n = \phi 135 t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\phi R_n = \phi 135 t_w^2 [\alpha] \sqrt{\frac{F_{yw} t_f}{t_w}}$$

$$\alpha = \left[1 + 3 \left(\frac{6}{7.93} \right) \left(\frac{0.245}{0.400} \right)^{1.5} \right] = 2.088$$

$$\phi R_n = (0.75) 135 (0.245)^2 (2.088) \sqrt{\frac{36(0.400)}{0.245}}$$

$$= 97.3 \text{ kips}$$

3. The maximum chord force is 25.7 kips based on joist girder seat rollover capacity.
4. The chord force must be specified to the joist manufacturer. The E member extension, and the seat stiffeners should be shown on the structural drawings.

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.

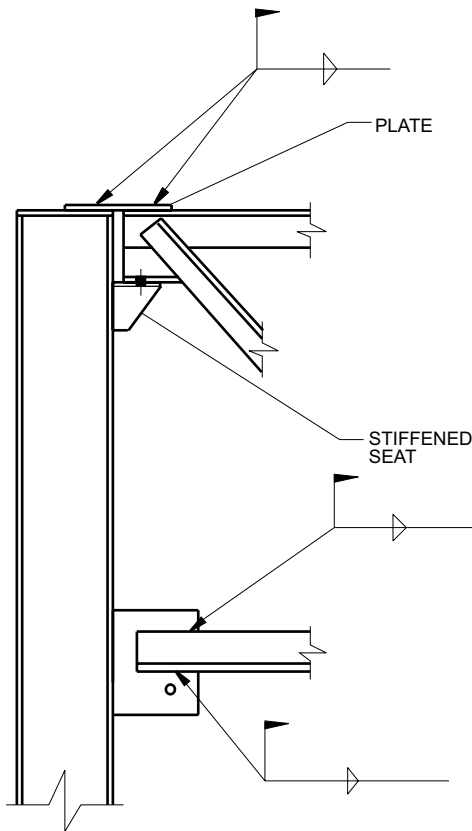


Fig. 7.5.1 Roof Moment Plate

The connection is ideally suited for floor girders and floor joists as well as roof girders and roof joists. To design

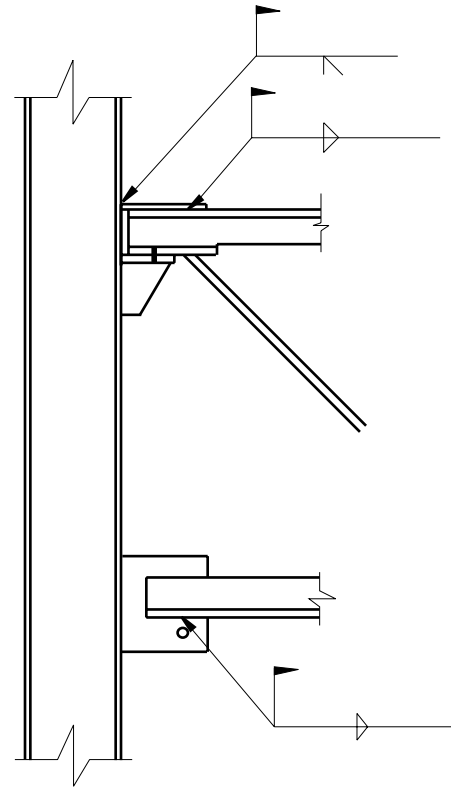


Fig. 7.5.2 Floor Moment Plate

the moment plate the engineer simply 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 obtain the maximum force in the top plate. The plate and its attachments to the column and joist or girder are designed using standard procedures. 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.

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 principals must be used to determine an unstiffened seat angle thickness, because the AISC Manual of Steel Construction 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.

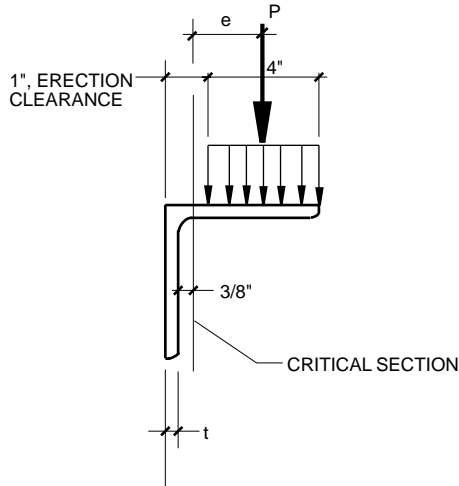


Fig. 7.5.3 Unstiffened Seat

If the vertical reaction of the girder is assumed to be centered in the 4 inch bearing length, and the critical bending location in the angle is assumed to be 3/8 inches from the angle face.

$$e = 3 - t - 3/8 = 2.625 - t \quad \text{Eq. 7.4-1}$$

The bending moment at the critical section
 $M = P_e$

$$\text{The bending stress, } f_b = \frac{M}{S} = \frac{P_e}{\frac{1}{6}bt^2} = \frac{6P_e}{bt^2}$$

Using the allowable stress as $F_b = 0.75 F_y$

$$t^2 = \frac{6P_e}{0.75F_y b} = \frac{8P_e}{F_y b}$$

For a 5 inch bolt gage, the typical seat angle will be 8 inches long. Therefore,

$$t^2 = Pe/F_y \quad \text{Eq. 7.4-2}$$

Thus, to solve for a seat angle thickness using ASD, one must first assume an angle thickness and then solve equations 7.4-1 and 7.4-2.

For LRFD, the solution is as follows:

$$\phi M_n \geq M_u$$

$$\phi M_n = \phi Z_x F_y \quad M_u = P_u e$$

$$\phi = 0.9 \quad Z_x = bt^2/4 \quad e = 3 - t - 3/8 = 2.625 - t$$

For a 5 inch bolt gage, the typical seat angle will be 8 inches long ($b=8$). Therefore,

$$Z_x = bt^2/4 = (8)t^2/4 = 2t^2$$

$$(0.9)(2t^2)(F_y) = P_u(2.625 - t)$$

Rearranging the above equation will yield a quadratic in t which can be solved for t for any given P_u .

On some occasions the knife plate connection shown in Figure 7.5.1 may be desired.

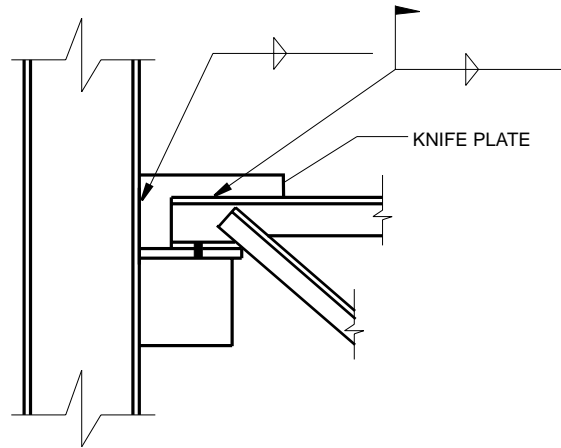


Fig. 7.5.4 Knife Plate Floor Connection

This connection saves field welding to the column and in some cases may eliminate the need for column stiffeners. The check for column stiffeners is identical to the check at the stabilizer plate location.

The manufacturer 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, 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 engineer should check with the manufacturer prior to using the knife plate connection.

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. This situation frequently exists at the perimeter of a roof or floor diaphragm when shear collectors are not provided. Based on an elastic analysis, the calculated rollover resistance of the joist seat to a lateral force is extremely low. An ultimate strength approach provides significantly higher resistance values. The difference between the two approaches is shown in the example below.

Example 7.6.1 Joist Seat Rollover Resistance

Determine the resistance to rollover of the seat shown below.

1. Calculate the resistance based on first yielding.
2. Calculate the resistance based on ultimate strength.

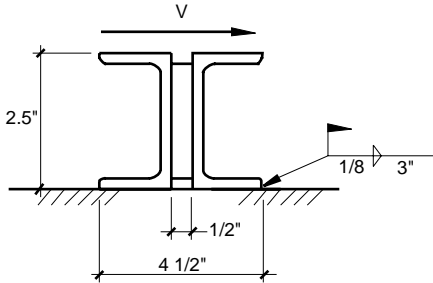


Fig. 7.6.1 Joist Seat

- a. The seat is made from 2"x2"x1/8" angle with a yield strength of 50 ksi.
- b. The seat has sufficient internal strength (based on Vulcraft's fabrication procedures) to force the failure to be a field weld failure or a failure associated with bending of the seat angle.
- c. The resisting forces are assumed as shown:

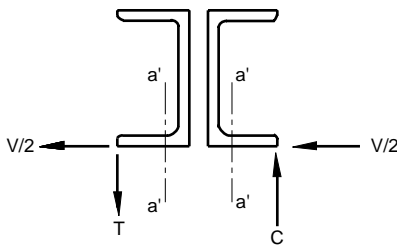


Fig. 7.6.2 Resisting Forces on Joist Seat

Solution:

1. Resistance based on first yield:

The yield moment can be determined from the section modulus of the seat angle. The seat angles section modulus, $S = bt^2/6$, where b at $a'-a'$ is determined as shown in Fig. 7.6.3.

$$\begin{aligned}
 b &= \text{weld length} + (2-k) \\
 b &= 3 + (2 - 7/16) = 4.56 \text{ in.} \\
 S &= (4.56)(0.125)^2/6 = 0.0119 \text{ in.}^3 \\
 M_y &= SF_b \\
 F_b &= 0.75 F_y = (0.75)(50) = 37.5 \text{ ksi.} \\
 M_y &= (0.0119)(37.5) = 0.446 \text{ in. - kips}
 \end{aligned}$$

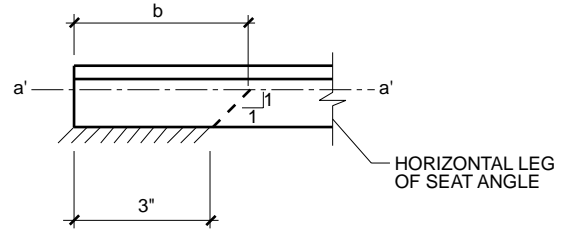


Fig. 7.6.3 Effective Angle Width

The maximum force T or C equals $M_y/(2-7/16)$
 $= 0.286$ kips

$$T = C = (2.5V)/4.5 = 0.56V$$

$$\therefore V \approx 510 \text{ pounds}$$

It can be seen that the strength is controlled by yielding in the seat angle rather than the strength of the field weld.

2. Resistance based on ultimate strength:
 At failure the seat is assumed to be deformed as shown Fig. 7.6.4.

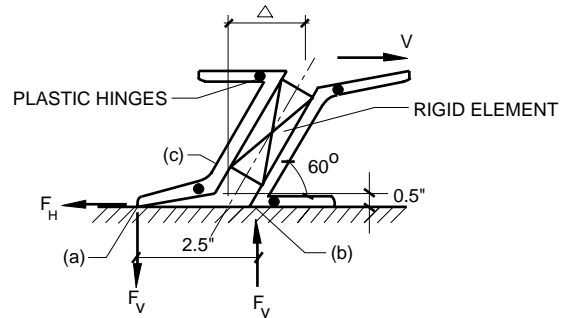


Fig. 7.6.4 Seat Failure Mode

It is assumed that the primary resistance to overturning is provided by the weld at point (a) and by a couple formed by the forces F_V . Plastic hinges are assumed to have formed in the seat angles. The seat leg which lifts from the base is assumed to hinge approximately 1/2 inch above the base. This assumption is based upon the location of the rigid end diagonal which is welded between the angles.

The ultimate weld resistance can be determined as follows:

Weld strength:

$$2.5 F_V = 2.5 V, \text{ thus } F_V = V$$

$$\begin{aligned}
 \Sigma \text{ Forces in the horizontal direction,} \\
 F_H = V
 \end{aligned}$$

The total force at (a) = $\sqrt{F_H^2 + F_V^2} = 1.414V$

The resisting force in the weld at point (a) equals $R = 1.414V$.

The design strength of the weld, based on the AISC LRFD Specification (using Appendix J) equals:

$$\begin{aligned}\phi R &= \phi F_w A_w \\ F_w &= 0.60 F_{EXX} (1.0 + 0.5 \sin^{1.5} \theta) \\ &= 63 \text{ ksi} \\ A_w &= (0.125)(0.707)(3) = 0.265 \text{ in.}^2 \\ \phi R &= (0.75)(63)(0.265) = 12.52 \text{ kips} \\ V &= \phi R / 1.414 = 8.85 \text{ kips}\end{aligned}$$

Seat angle strength:

The maximum strength of the seat angle equals the shear yield strength of the seat angle times the shear area at point (c). Using the von Mises yield criteria:

$$V = (50 / \sqrt{3})(0.125)(4.56) = 16.45 \text{ kips}$$

Thus, the design strength is 8.85 kips.

Using a factor of safety of 2.0 the allowable shear force equals 4.42 kips.

As can be seen from the deformed shape, the seat assembly would displace laterally a significant amount at the ultimate load. Based upon the geometry assumed in Fig. 7.6.4, the lateral deflection would be approximately 1.15 inches. It is suggested that the deflection at service loads be limited to 0.25 inches. It should be noted that even at this limit there would be some yielding of the connection.

Thus by proportioning, the allowable shear equals $(0.25)(8.85)/1.15 = 1.92 \text{ kips}$.

The allowable lateral force capacity for any given joist seat can be based upon the ultimate strength procedure shown above, provided the basic assumptions as given are not violated. The assumption relative to the internal strength of the seat assembly is beyond the control of the building designer. This strength cannot be easily calculated. Some upper bound is required.

An upper bound on the internal seat strength can be based on tests which were performed by Vulcraft.

Seat Type	Bearing \angle size	Ave. Failure Load, lbs.
8H3	1-1/2x1x.109	8940
24H6	1-1/2x1x.145	13500
26H8	1-1/2x1x.163	15630*

* Test discontinued.

Summary of Test Results

Ten tests were conducted on three different seat angle thicknesses. All the seats in these ten tests were fabricated from clipped angles to form a 2-1/2 inch seat. The top chord consisted of 1-1/2 inch angles in each case. The end diagonal of the joist was welded between the seat angles using Vulcraft's standard fabrication practice for H and K joists. Four tests were conducted on seats for 8H3 joists, four tests on seats for 24H6 joists, and two tests for 26H8 seats. The typical seat configurations are shown in Figure 7.6.5 and 7.6.6.

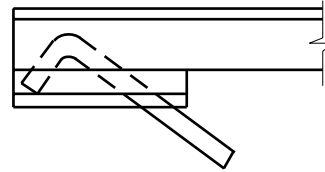


Fig. 7.6.5 8H3 and 24H6 Seats

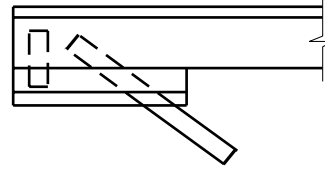


Fig. 7.6.6 26H8 Seats

The tests were conducted by welding the seats to a fixed support. Design vertical loading was applied to the joist prior to the lateral force being applied. Shown in Figure 7.6.7 is the welding and lateral load arrangement:

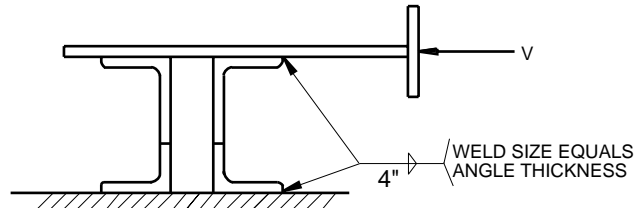


Fig. 7.6.7 Load Arrangement

The fillet weld size was equal to the seat angle thickness. Summarized below are the results of the ten tests.

For the 8H3 and 24H6 seats, the failure mode was that of weld failure at the base of the seat angle or between the loading plate and the top chord. Failure load on the 26H8 joist was not reached due to the capacity of the loading system. Internal weld failure did not occur on any of these tests. Significant lateral distortion was noted at failure loads on all ten tests.

Two additional tests were conducted using lapped bearings on 26H8 seats. The seat configuration is shown in Figure 7.6.8. The average ultimate load for these two tests was 14,630 pounds. The failure mode was that of weld failure between the seat angle and the support.

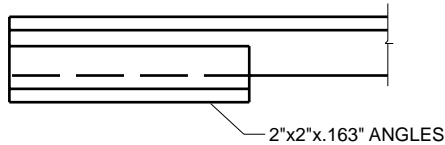


Fig. 7.6.8 Lapped Bearing

Based on the results of these tests, a conservative internal strength values for lateral load is 9000 pounds. Thus, if the ultimate analysis procedure as given here is used to predict the rollover capacity, an upper bound on the strength limit should be taken as 9000 pounds. For most welding configurations it would appear that the capacity will be controlled by lateral deflections.

It is interesting to compare computed ultimate strength values with experimental values. For the 8H3 seats, the computed ultimate strength using the method presented herein is 8,808 pounds, and for the 24H6 seats is 11,717 pounds, as compared to test values of 8,940 and 13,500 pounds respectively.

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 in particular 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.
- 2) “Recommended Code of Standard Practice for Steel Joists and Joist Girders”, published by the Steel Joist Institute.

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 of 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 five major topic areas: 1) General, 2) Bidding, 3) Drawings and Specifications, 4) Collateral Material, 5) Construction Practice.

The buyer is expected to provide “complete architectural plans and specifications, structural steel drawings and purlin plans, 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 bid for roof deck would include the deck, ridge and valley plates and sump pans. Other accessories must be specified. The base bid for floor and form deck would include only the deck. Other accessories must be specified. Unless otherwise specified the support of steel deck is not included in the base bid.

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 covers six collateral materials.

1. Insulation: all steel roof decks shall be covered with a material of sufficient insulating value to prevent condensation under normal operating conditions. It is expected that the insulation be adequately secured using adhesives or mechanical fasteners.
2. When open rib acoustical deck is provided insulation batts are to be installed by the roofing contractor.
3. Roofing: a suitable roof covering is expected.
4. Sheet metal: unless otherwise specified all closures, flashings, etc. used in roof deck construction shall be detailed and furnished by the sheet metal contractor.

5. Field painting: all field painting and touch up is expected to be the responsibility of the buyer.
6. Shear connections: the design, sizing, layout of shear connections is not expected to be the responsibility of the steel deck manufacturer.

The construction phase involving the site storage, and installation of steel decking is covered in each of the three SDI specifications as appropriate to each usage. 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 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, in reality, 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.

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 joists, joist girders and accessories.
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 as defined in Section 5.5 of the Code.
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 include the following:

1. Steel joists.
2. Joist girders.
3. Joist extended ends.
4. Ceiling extensions.
5. Extended bottom chord used as strut.
6. Bridging and bridging anchors.
7. Joist girder bottom chord bracing.
8. Headers supported by and carrying K-series joists.
9. One coat of shop paint (a primer per SJI Specifications), when specified.

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 for LH-series joists.
2. Headers for DLH-series joists
3. Reinforcement in slabs over joists.
4. Centering material and attachments.
5. Miscellaneous framing between joists for openings.
6. Loose individual or continuous bearing plates 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. Wood nailers.

10. Moment.

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.

The plans furnished by the seller include detailed plans and lists showing the number, type, location, spacing, anchorage and mark of all joists, joist girders and accessories. The shop paint is also identified.

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 No. 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 with regard to billing and payment:

- a) Lump sum contracts are to be billed proportionately to shipments; and
- 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 buyers side are numerous. The relations on the buyer's side can best be illustrated by using the example of five different joist support types:

1. Structural steel frame.
2. Pre-engineered metal building frame.
3. Light gage bearing walls.
4. Concrete frame.
5. Masonry bearing walls.

and by considering relations among the owner, the architect/engineer and the contractor.

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".⁴ This code gives trade practices relating to the fabrication and erection of structural steel. It contains this definition of structural steel:

" 'Structural Steel', 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. Unless otherwise specified in the contract documents, these elements consist of material as shown on the structural steel plans and described as: [the code then provides a list of elements]." "Cold-formed steel products" [deck] and "open-web, long-span joists and joist girders" are not included in the list of structural steel, but rather they are listed in "Other Steel or Metal Items", a category of items not included in Structural Steel "even when such items are shown on the structural steel plans or are attached to the structural frame." The two codes must be bridged in the contract documents and the contracts. One almost universal method is to make the erection of structural steel and the erection of steel deck, steel joists and joist girders the work of one erector. This insures 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 does not cover the erection of steel deck, steel joists and joist girders. The principal document which does is SJI Technical Digest No. 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.

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, the common way to have these components erected into one uniform structural framework is to have one party erect all these components under one contract. 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 frame work 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 which define the responsibilities of the parties involved in the bidding, purchasing, supplying and erecting structural steel. Such documents normally consist of the Design Drawings, 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 sub-contractor.

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 vis-a-vis the SJI Code of Standard Practice. All of these groups have addressed issues of material and workmanship standards, but not specifically in relation to joists.

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 non-structural 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 program 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 sufficient 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 generally 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 subcontractors 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 and erection drawings (shop drawings) showing the work required for the steel frame work 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 material lists 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 lists. The only responsibility of the manufacturer is to provide material conforming to the requirements given on the list, not the plans and specifications. When lists are prepared by others for the manufacturer, the special skills that the manufacturer has in reading plans and specifications in light of 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 lists 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. With regard to Specifications they 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, non-standard 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 with regard to special, non-standard 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 and 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

1. American Plywood Association, "APA Design/Construction Guide – Residential and Commercial", Tacoma, WA, 1998.
2. Approval Guide, "A Guide to Equipment, Materials, and Services Approved by Factory Mutual Research for Property Conservation", Factory Mutual Research Corporation, Norwood, MA 02062, September 2000.
3. "Code of Recommended Standard Practice For Composite Deck, Form Decks, and Roof Deck Construction", Steel Deck Institute, Fox River Grove, IL, 1995.
4. "Code of Standard Practice for Steel Buildings and Bridges", American Institute of Steel Construction, Chicago, IL, 2000.
- 5.. DeFalco, Fred and Frank J. Marino, "Column Stability in Type 2 Construction", *AISC Engineering Journal*, April, 1966.
6. "Design Manual for Composite Decks, Form Decks and Roof Decks", Steel Deck Institute, Fox River Grove, IL, 1995.
7. "Diaphragm Design Manual", Steel Deck Institute, Fox River Grove, IL, 1995.
8. Disque, Robert O., "Directional Moment Connections – A Proposed Design Method for Unbraced Steel Frames", *AISC Engineering Journal*, First Quarter, 1975.
9. Driscoll, George C. Jr., "Effective Length of Columns with Semi-Rigid Connections", *AISC Engineering Journal*, Fourth Quarter, 1976.
10. "European Recommendations For The Design of Composite Floors with Profiled Steel Sheet", European Convention for Constructional Steelwork, Committee II, CONSTRADO, Craydon, England, 1975.
11. "Evaluation Reports – Materials, Products, Methods and Types of Construction", International Conference of Building Officials, Whittier, CA, various dates.
12. "Expansion Joints in Buildings, Federal Construction Council, Technical Report No. 65", National Academy of Sciences – National Research Council, Washington, D.C., 1974.
13. "Fire Resistance Directory", Underwriters Laboratories, Inc., Northbrook, IL, 2000.
14. Fisher, James M. and Michael A. West, "Serviceability Design Considerations for Low-Rise Buildings, Steel Design Guide Series 3", American Institute of Steel Construction, Chicago, IL, 1990.
15. Galambos, Theodore V., "Guide to Stability Design Criteria for Metal Structures", 4th Edition, John Wiley & Sons, New York, NY, 1987.
16. "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, 1987.
17. "Load and Resistance Factor Design Specification for Steel Buildings and Commentary, American Institute of Steel Construction, Chicago, IL, 1999.
18. "Loss Prevention Data for Roofing Contractors", Factory Mutual Research Corp., Factory Mutual System, Norwood, MA, various dates.
19. "Low Rise Building Systems Manual", Metal Building Manufacturers Association, Cleveland, OH, 1996.
20. "Minimum Design Loads for Buildings and Other Structures", ASCE 7-98 (Formerly ANSI A58.1), American Society of Civil Engineers, New York, NY, 2000.
21. "N.R.C.A. Roofing and Waterproofing Manual", The National Roofing Contractors Association, Chicago, IL, 1996.

REFERENCES

22. "Recommended Code of Standard Practice for Steel Joists and Joist Girders", Steel Joist Institute, Myrtle Beach, SC, 1994.
23. Seely, Fred B., and James O., Smith, "Advanced Mechanics of Materials", John Wiley and Sons, Inc., New York, NY, 1962.
24. "Seismic Design for Buildings", TM 5-809-10, Departments of the Army, the Navy, and the Air Force, 1982.
25. "Specifications and Commentary for Composite Steel Floor Decks", Steel Deck Institute, Fox River Grove, IL, 1995.
26. "Specifications and Commentary for Non-composite Steel Form Deck", Steel Deck Institute, Fox River Grove, IL, 1995.
27. "Specification and Commentary for Steel Roof Deck", Steel Deck Institute, Fox River Grove, IL, 1995.
28. "Specification for the Design of Cold-Formed Steel Structural Members", American Iron and Steel Institute, Washington, D.C., August 19, 1986 with December 11, 1989 Addendum.
29. "Specification for the Design and Construction of Composite Slabs and Commentary on Specifications for the Design and Construction of Composite Slabs", ANSI/ASCE 3-91, American Society of Civil Engineers, New York, NY, 1991.
30. "Specification for the Design, Fabrication and Erection of Structural Steel Buildings with Commentary", American Institute of Steel Construction, Chicago, IL, 1989.
31. "Specifications for Electric Overhead Traveling Cranes", Crane Manufacturers Association of America, 1987.
32. "Standard for the Installation of Sprinkler Systems", NFPA 13-1983, National Fire Protection Association, Quincy, MA, 1983.
33. "Standard Methods of Fire Tests of Building Construction and Materials", ASTM E119-00, West Conshohocken, PA 19428-2959.
34. "Standard Specifications for Joist Girders", Steel Joist Institute, Myrtle Beach, SC, 1999.
35. "Standard Specifications for Long Span Steel Joists, LH-Series and Deep Long Span Steel Joists, DLH-Series", Steel Joist Institute, Myrtle Beach, S.C., 1994.
36. "Standard Specifications for Open Web Steel Joists, K-Series", Steel Joist Institute, Myrtle Beach, S.C., 1994.
37. "Steel Floor and Roof Deck", Vulcraft, A Division of Nucor Corporation, Charlotte, N.C., 1998.
38. "Steel Joists and Joist Girders", Vulcraft, A Division of Nucor Corporation, Charlotte, NC, 1998.
39. Young, Warren C., "Roark's Formulas for Stress and Strain", McGraw Hill, New York, NY, Sixth Edition, 1989.

A

American Concrete Institute, 25, 153

ACI 318, 20
requirements, 20

ACI 523.1 Guide for Cast-in-Place Low Density Concrete, 10

acoustical insulation, 19

AISC
"Manual of Steel Construction", 116, 118, 120
"Specification for the Design of Steel Hollow Structural Sections", 141
Code of Standard Practice, 153, 154
Code of Standard Practice for Steel Buildings and Bridges, 152
Design Guide No. 11 "Floor Vibrations Due to Human Activity", 92
Design Guide No. 3, 25
Manual of Steel Construction, 145
Specification, 16, 71

AISI, 153
"Specification for the Design of Cold-Formed Steel Structural Members", 13, 115, 118, 120

American Institute of Timber Construction, "Timber Construction Manual", 31

American Plywood Association, 11
"Plywood Diaphragm Construction", 31

American Society of Civil Engineers (ANSI/ASCE 3-91), 24

analysis, second order, 68

arbitration, 152

architect/engineer, 150, 154, 155

architectural plans, 150

ASCE, "Specifications for the Design and Construction of Composite Slabs and Commentary on Specifications for the Design and Construction of Composite Slabs.", 120

ASCE 7-98 Minimum Design Loads for Buildings and Other Structures, 4, 11, 12, 13

ASCE Standard Specification for the Design and Construction of Composite Slabs, 3

ASTM
A36 - Structural Steel, 115, 118, 120
A611 - Structural Steel, Sheet, Carbon, Cold-Rolled, 7, 115, 118, 120
A653 (G60) galvanizing, 20
A653 - Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot Dip Process, 115, 118, 120, 121

Standard E119, "Standard Test Methods for Fire Tests of Building Construction and Materials", 97

AWS

D1.1 - Structural Welding Code, 86, 116, 118, 120
D1.3 - Specification for Welding Sheet Steel in Structures, 116, 118, 119, 120

B

base bid, 150

basic connection, 17, 18, 44, 45, 50, 55, 56, 59, 65
ties, 51

bay size, 15, 25

bay spacing, 16

beam, bond, 36

beam seat, 77

beam-column, 50

bearing plate, 118

bent plates, 93

BOCA National Building Code, 2

bolts, combined tension and shear, 133

bow string, 2

brace, lateral, 17, 71

bracing, 58
bottom chord, 114
horizontal, 38, 41, 42
horizontal roof, 55
horizontal truss, 41
roof
one direction, 40
two directions, 41
scheme, 155
sway, 78
temporary, 37, 153, 154
vertical bracing system, 35, 42
X-, 56

Brick Institute of America, 14

bridging, 16, 79, 86, 109, 153
anchorage, 112
attachment, 153
considerations, 110
cutting, 98
diagonal, 111
horizontal, 111
special, 38, 155
standard, 113
x-bridging, 16, 112

standard bridging, standard, 112

buckling, lateral sway, 50

budgets, 155

building

designer, 3

exposure, 12

geometry, 12, 43, 55

Building Code Requirements for Masonry Structures

ACI-ASCE 530, 4

Building Code Requirements for Reinforced Concrete

ACI 318, 4

Building Research Advisory Board of the National

Academy of Sciences, 14

buyer, 150

C

C332 Standard Specification for Light-Weight Aggregates for Insulating Concrete, 10

cable trays, 78

camber, 36, 75, 86, 98

carpenters, lathers, insulators, 150

cast-in-place concrete, 19

ceilings, 24, 151

channel, inverted, 82

chord

bending, 33, 79

capacity, 87

connections, 32

forces, 109

reinforcement, 88, 90

rod bottom extension, 48

chords

axial load, 125

bottom chord force transfer, 138

bottom extensions, 131, 136

buckling, 137

eccentric, 124

extended bottom, 151

forces, 125

gap, 136

increasing capacity, 134

maximum allowable eccentric force, 128

maximum compression force in the top chord, 142

maximum eccentric force, 124, 125, 126

maximum size, 125

maximum tension force bottom chord, 142

stability of the bottom chord, 137

cladding system, 25

clearance requirements, 155

CMAA Specification #74, Revised 1987, 70

CMU wall, 36

Code of Standard Practice, 1, 3

cold-formed steel products, 152

column

AISC nomograph, 66

bottom chord attachment to flanges, 136

bracket, 48, 53

buckling load, 50

cap, 129, 133

cap plate, 17, 124, 125

cap plate thickness, 131

considerations, 136

design, 49, 54

ductwork size, 27

effective length factor, 49, 54, 137

effective web width, 137

Euler buckling load, 50

inelastic K method, 49

K factor, 50

leaner, 49, 55

maximum slenderness ratio, 58

nomograph, 49

stability, 41, 55

stiffeners, 146

web, 137, 143

composite joist, 27

concrete

lightweight, 28

lightweight insulating, 10

concrete beams, 153

concrete contractors, 150

concrete deck, 10

concrete frame, 152

concrete stops, 23, 118, 121

connection, beam to girder, 75

connections, 16, 28, 41, 54

basic, 123, 124, 125, 128, 139

beam-to-column, 52, 55

bracketed, 49

design, 123

detail, 44

doubler plates, 137

field, 154

for horizontal bracing systems, 41

joist girder wind, 54

knife plate, 48, 49

knife plate floor connections, 146

modifications to the basic connection, 128

- moment, 44, 48
- plates, 131
- rigid, 2, 125
- rigid moment, 44
- rigid seat, 48
- sidelap, 30, 35, 119
- sidelap welds, 30
- special, 155
- Type 2 wind, 52
- typical, 139, 140, 142, 143
- welded basic, 45
- welded basic with ties, 46
- wind, 52
- yieldable top plate, 55

construction documents, 1, 83

content of plans, 155

continuity

- angles, 129
- moments, 129, 131, 145
- plates, 129
- tie, 45, 135, 139
- tie plate, 49
- transfer, 129

contract, 36, 150

- lump sum, 152

contract documents, 126, 153

- definition, 153

conveyors, 76

- chain driven, 77
- continuous belt, 77
- equipment, 76
- trolley, 77
- vibratory, 77

cost effective design, 43

cover plates, 118, 121

crane

- bumper, 71
- cab operated, 70
- Crane Manufacturers Association of America, Inc., 70
- longitudinal brace, 74
- longitudinal thrust, 72
- monorails, 70
- patented runways, 71
- pendant operated, 70
- radio operated, 70
- runway, 71
- runway beam, 16, 71
- runway hanger, 72, 73
- runway misalignment, 70
- runway stops, 70

- skewing of the crane bridge, 70
- stop, 71
- support joist, 71
- support system, 70
- top running, 70
- trolley movement, 70
- trolley weight, 71
- underhung, 70

curbs, 79

curtainwall system, 2

D

deck

- 0.6 and 1.0 C-series decks, 10
- acoustical, 150
- allowable reaction, 20, 22
- attachment of deck perimeter, 153
- attachment to shear walls, 36
- bearing length, 22
- button punching, 30
- cellular steel, 121, 122
- centering, 19
- centering material, 151
- composite, 19, 20
- composite floor, 19
- composite steel, 120, 121
- composite steel floor, 122
- continuity, 104
- corrugated metal form, 19
- deflected, 86
- deflection, 7, 19, 24
- edge angle, 35, 36
- effective slab width, 22
- electrical raceway, 20
- embossed sides, 20
- ends, 119
- erector, 153
- fastening, 29
- flute closures, 23
- fluted, 118
- form, 10, 19, 20
- galvanized form, 19
- gypsum concrete, 10
- intermediate rib, 7
- load table, 19
- metal closure strips, 118
- minimum reinforcement, 20
- narrow rib, 7
- openings, 116
- painted form, 19
- plank and laminated wood, 10
- plywood, 23
- positive moment reinforcement, 20

- precast concrete, 10
 - primer coat, 7
 - procurement, 154
 - recommended spans, 7
 - reinforcement, 78
 - reinforcing bars, 20
 - ribbed steel forms, 118
 - roof, 151
 - screwed sidelap, 57
 - sound control, 19
 - steel, 7
 - steel form deck metal centering, 119
 - support angle, 35
 - types, 7
 - unshored, 20
 - unshored condition, 19
 - vertical bracing, 37
 - Vulcraft C-series, 19
 - wide rib, 7
 - wood, 10, 23
 - wood plank, 23
 - deep foundation, 16
 - deflection, 25
 - eave, 65
 - lateral, 149
 - maximum, 32, 59
 - deflections, roof, 13
 - design, optimum, 31
 - design drawings, 84
 - diaphragm
 - capacity tables, 7
 - shear capacities, 7
 - steel deck, 2
 - strength, 7
 - values, 23
 - diaphragms, 29, 55
 - chord, 33, 34, 35
 - chord force, 32
 - connections, 32
 - deflection, 32
 - design manual, 30
 - diaphragm action, 55
 - edge angle, 58
 - force attachment, 36
 - plywood, 31
 - roof, 55
 - roof and floor, 29
 - shear, 35, 38
 - strength and stiffness, 29, 31
 - timber, 31
 - types, 29
 - welded, 31
 - wood, 29, 31
 - wood deck, 30
 - dimensional tolerances, 93
 - Disque, 54
 - distribution beam, 84
 - distributional steel, 23
 - double angles, moment capacity, 126
 - double sloped building, 99
 - draw, 39
 - drift, 51
 - calculation, 55
 - consideration, 51, 55
 - indices, 53
 - duct, sizes, 26
 - ducts, 86
 - systems, 78
 - dynamic excitation, 82
- ## E
- E member, 131, 139
 - E member extension, 68, 134, 135
 - economical, 16, 38
 - approach, 55
 - design, 43
 - joist guide, 104
 - system, 29
 - electrical lines, 86
 - electrical raceway, 19
 - embedded weld plates, 153
 - engineer of record, 13
 - erection, 110, 136
 - bolts, 151
 - drawings, 154
 - layouts, 150
 - of structural steel, 153
 - erector, 154
 - estimate, 152
 - expandable wall, 100
 - expansion, future, 56
 - expansion joint, 14, 25, 43, 55, 100, 101, 112
 - shear transfer, 39, 40
 - exposure and thermal conditions, 12
 - extra charges, 150

F

fabrication, 154
 fabrication and erection tolerances, 71
 fabricators, 154
 factor of safety, 31
 Factory Mutual, 7, 14, 16
 Approval Guide, 4, 7
 Class 1, 1-90 roof construction, 13
 Class I, 115
 failure mechanisms, 125
 fastener, sidelap, 7
 fastener types, 29
 fasteners
 mechanical, 119, 121, 150
 power driven, 30, 57
 self drilling screws, 30
 fatigue, 70, 77, 109
 considerations, 71
 Federal Construction Council Technical Report No. 65
 "Expansion Joints in Buildings", 14
 finishes, 19
 galvanized, ASTM A924 or ASTM A653 (G60), 19
 painted, 19
 unfinished, 19
 fire ratings, 97
 fire resistance, 97
 Fire Resistance Directory, 4
 fireproof, 20
 fireproofing, 121
 fit-up, 154
 flange plate, 54
 floor, 19
 construction, 23
 deck, 19
 diaphragm, 23
 moment plate, 145
 slabs, 151
 system, 19, 25, 27, 91
 vibration, 19, 91, 92
 floors, efficient framing, 1
 fork truck, 21, 24
 foundations, 155
 frame, structural steel, 152

frames
 braced, 42
 multistory, 42, 44
 rigid, 43, 55, 66
 rigid frame building, 49, 65
 rigid frame system, 56
 single story, 43
 wind connections, 51
 framing
 considerations, 15, 25
 depths and widths, 26
 system, 41
 to column web, 137
 frequency, 82, 91
 natural, 82, 91, 92
 operating, 82
 functional requirements, 15

G

general contractor/construction manager, 150, 154, 155
 geometrical and tolerance requirements, 136
 girls, 2, 16
 gravity and shear load transfer to masonry, 37
 Guide for Specifying Steel Joist with Load and Resistance Factor Design, 3
 guy wire, 82
 attachment, 83

H

hanger, 12, 78
 and bracing, 71
 attachment, 77
 clamp type, 72
 crane beam and monorail support, 71
 hanger loads, 12, 70
 pipe, 77
 hatches, 79
 headers, 28, 78, 151
 HSS walls, 138

I

ICBO, 29
 ICBO Evaluation Reports, 31
 ICBO Reports on diaphragms, 32
 Identification Index, 11

impact, 70
 in-plane shear forces, 29
 in-plane slenderness ratio, 58
 inspection, 83, 87
 installation holes, 122
 insulating concrete, 10
 insulation, 150
 insulation batts, 150
 International Building Code (IBC), 3
 International Conference of Building Officials, 4
 isolators, 82

J

joints, 38
 expansion joint, 38
 vertical slip, 14
 joist
 chords, 126
 CSV series, 10
 custom design, 1, 2, 20, 28, 104
 depth, 2, 16
 designation, 81, 104, 155
 diagram and girder schedule, 61
 DLH-series, 2
 double pitched, 2
 E member extension, 52
 eccentric top chord force capacity, 132
 end moment, 109
 K series, 87, 125
 KCS series, 72, 80, 98, 106, 107
 specifications, 74
 LH-series, 1
 loaded scissor, 102
 modifications, 131
 moment capacity, 86
 moment of inertia, 49
 non standard types, 102
 panel points, 79, 86
 permissible end moment, 125
 reinforced chord extension, 131
 reinforced chords, 47
 reinforcement, 82, 83, 86, 87
 adding new joists, 86
 round rod extension, 136
 schedule, 109
 schedule of moment, 109
 scissors, 2

seat, 49, 125, 147
 seat rollover resistance, 147
 seats subjected to rollover forces, 146
 selection, 104
 size and spacing, 155
 SLH series, 2
 sloping, 101, 103
 spacing, 16, 19, 20, 25
 span direction, 25
 special, 3, 71, 80, 81, 106
 special design, 2
 special edge, 38
 special profile, 2, 101
 splices, 103
 standard, 106
 standard and custom, 3
 standard designation, 3
 tag, 84
 tie angles, 33
 tie plate, 33
 tilted, 99
 VS series, 2
 web members, 87

joist and deck erector, 150

joist and girder depth, 16
 optimum, 16

joist girder
 chord capacity, 131
 chord moments, 124
 detail, 44
 fixed seat, 130, 135
 flange plate, 54
 force couples, 128
 load diagram, 112
 minimum top chord width, 130
 modifications, 129
 note, 37
 sample specification, 113
 schedule, 113
 seat, 125, 132
 seat detail, 129
 seat extension, 131
 seat rollover, 130
 seat rollover capacity, 145
 seat rollover strength, 126
 seat stiffeners, 132
 special seat, 47
 standard designation, 110
 top chord, 47
 VG series, 110
 web configurations, 111

joist girders, depth and loading, 155

L

lateral sway, 55

light gage bearing walls, 152

light gage steel fabricators and erectors, 150

Load and Resistance Factor Design Specification for Structural Steel Buildings, 4

Load and Resistance Factored Design (LRFD), 3

Load Tables and Weight Tables for Steel Joists and Joist Girders, 2

loading, floor, 23

loads, 155

- collateral, 11, 24, 78
- combinations, 4, 13
- concentrated, 13, 19, 21, 87, 151
- concentrated load, 106
- concentrated on joists, 79
- construction, 19, 112, 154
- dead, 4, 11
- design, 155
- diagram, 3, 81, 107, 109, 110
- distribution, 84
- distribution to joists, 84
- during construction, 19
- dynamic, 21
- earthquake, 4
- edge shear, 59
- equivalent uniform, 23, 106
- gravity, 15
- ground snow data, 12
- hanging, 70, 85
- KCS series tables, 75
- lateral, 2, 41, 45, 55
- lateral load resistance, 29
- lateral load system, 29
- lifted, 71
- live, 4, 12, 24
 - reduction, 12, 24
- mechanical, 81
- net uplift diagram, 110
- pressure and suction reactions, 41
- rain, 4, 12
- rain on snow, 12
- seismic, 41, 78
- selection of the lateral load system, 55
- snow, 4, 12
- snow accumulation, 82, 86
- snow drift, 12
- special, 3
- tables, 20, 30, 31, 104

- thermal, 14
- uniform, 19
- uniform gravity, 104
- uplift, 13, 82, 109
- uplift wind, 36
- wheel, 71
- wind and seismic lateral, 31

local web yielding, 67, 139

Loss Prevention Data Sheets, 4

Loss Prevention Guide, 7

- Data Sheets 1-7 "Wind Forces on Building and Other Structures, 12
- LPG 1-28 "Wind Loads to Roof Systems and Roof Deck Securement", 7, 13
- LPG 1-29 "Above-Deck Roof Components", 7, 13

Low Rise Building Systems Manual, 4

M

material strengths, 155

maximum compressive force, 126

Maximum K-Series Joist Moments, 105

Maximum LH Joist Moments, 105

mechanical systems, 155

mechanical unit, 79

Metal Building Manufacturer's Association, 153

Metal Lath/Steel Framing Association, 153

moment magnification, 50, 66, 68, 126

moment plate connection, 48

monorail, 70, 71

multiple bid packages, 155

N

National Concrete Masonry Association, 14, 153

National Design Specification for Wood Construction, 4

National Fire Protection Association (NFPA), 78

- NFPA 13 Standard for the Installation of Sprinkler Systems, 78

National Roofing Contractors Association, 14

net uplift, 13

nominal moment capacity, 127

O

openings, 78, 109, 122, 151, 155

- double frame, 79, 80

OSHA, 16
 out-of-plane slenderness ratio, 58
 outriggers, 94
 owner, 150, 153
 requirements, 15
 Owner's Designated Representative, 153

P

paint
 colored primer, 71
 field painting, 151
 shop, 152
 touch-up primer, 121
 parking garage, 20
 partial composite action, 28
 partitions, 14, 24, 151
 folding, 99
 plans and specifications, 152, 154
 plaster and other ceilings, 25
 plenums, 82
 Plywood Design Specification, 4
 ponding, 12, 94
 initiation, 96
 portal analysis, 52
 Pour Stop Selection Table, 93
 pre-cast slabs, 23
 pre-engineered building manufacturers and erectors,
 150
 pre-engineered metal building frame, 152
 pre-engineered metal buildings, 153
 prying forces, 133, 134
 purlin plans, 150

Q

quality control, 154

R

radius of gyration, 58
 Recommended Code of Standard Practice for Steel
 Joists and Joist Girders, 94

Recommended Practice for Engineering Brick Masonry,
 4
 reinforcement, 20, 84, 86
 concentrated load, 108
 joist diagonal, 90
 joist end seat, 91
 responsibilities, 150, 153
 responsibility of the buyer, 152
 rod extension, 131, 136
 rollover, 59, 124, 132
 rollover capacity, 38, 48, 49, 125, 127, 132, 141, 143,
 144, 149
 rollover capacity of joist girder seats, 126
 roof
 area dividers, 14
 ballast, 13
 ballasted system, 15
 blocked drains, 12, 95
 blocked edges, 30
 built-up, 6
 bulb tees, 10
 controlled flow roof drain design, 95
 dead weights, 11
 drainage, 81, 121, 155
 empounded water, 96
 framing, 16
 liquid applied membranes, 6
 membranes, 14, 15
 minimum pitch, 15
 moment plate, 145
 pitches, 10
 projections, 82
 recommended spans, 8
 relief drains, 12
 scupper, 95
 single-ply, 6
 slope, 15, 155
 standing seam, 6, 15, 16, 38, 55, 99, 110, 112
 Nucor, 16
 steep slope, 6
 strap bracing, 38
 structural steel, 15
 system, 55
 through-fastener, 6, 15
 types, 6
 typical opening, 79
 roof and floor diaphragm system, 55
 Roof Diaphragm Key Plan, 32
 Roofing Materials and Systems Directory, 13
 RTU, 78, 79, 80, 82
 zone, 81

zoning, 80

S

schedule, 3, 109

joist and joist girder, 112

scupper, parapet, 12

SDI

“Design Manual for Composite Decks, Form Decks and Roof Decks”, 116, 118, 119, 120, 121

“Design Manual for Floor Decks and Roof Decks”, 116

“Specifications and Commentary for Composite Steel Floor Deck”, 120

“Specifications and Commentary for Non-Composite Steel Form Deck”, 118

Code of Recommended Standard Practice for Composite Deck, Form Deck and Roof Deck Construction, 150

Code of Standard Practice, 150

Design Manual, 121

Design Manual for Composite Decks, Form Decks and Roof Decks, 3

Diaphragm Design Manual, 116, 118, 120

seat

depth, 25

depth changes, 99

seats

allowable rollover force, 127

angle strength, 148

angles, 125, 129

attachment, 129

attachment, 153

attachment of stabilizer plate, 153

bending resistance, 127

bolt tension, 133

capacity, 37

coped, 105

depth, 100, 105, 106

design strength rollover shear, 134

detail, 125

end, 86

extended, 134

extended ends, 94, 99, 151

extending, 49

extensions, 131, 134

failure mode, 147

fixed joist girder seats, 129, 130

moment capacity, 135

moment capacity of the seat angle, 127

non-standard, 128

reinforcement, 75

resisting forces, 147

rollover capacity of typical joist seat, 35

special depth, 155

stiffener, 135, 144

seller, 150

sequence, construction, 37

serviceability, 91

serviceability considerations, 13, 24

serviceability requirements, 32, 51

shaker table, 77

shallow foundation, 16

shear, allowable end, 87

shear capacity, 1, 24, 106

shear collector, 35

with K joist, 35

with LH Joist, 35

shear connections, 151

shear connectors, 28

shear diagrams, 87

shear forces, 35

shear strength, 22

shear transfer to masonry, 36

shear transfer to precast, 36, 37

shear wall, concrete or masonry, 43

sheet closures, 122

sheet metal, 150

shop drawings, 114, 154

shoring, 122

shrinkage, 20

sidelaps, 29, 119, 122

single story industrial building, 51

site visit, 84

SJI

standard designations, 1, 2

Standard specification, 2, 3

Technical Digest, 4

skylights, 79

slenderness ratio, 50, 111

sliding clip, 15

slip joint, 36

slots, long, 131

span, direction of joist, 16

spandrel member, 25

- spandrel systems, 93
- special bays, 155
- special joists, 106
- Specification for Structural Steel Buildings-Allowable Stress Design and Plastic Design, 4
- Specification for the Design and Construction of Load-bearing Concrete Masonry, 4
- Specification for the Design of Cold-Formed Steel Structural Members, 3
- specification of joists and girders, 104
- specifications, 150
- Specifications and Commentary for Steel Roof Deck, 7
- sprinkler, 77
 - pipes, 78
 - support, 77
 - typical sprinkler system weights, 78
- square bay, 15
- stability, 43, 49, 110
 - lateral, 38, 112
- stability forces, 113
- stabilizer, stiffener reinforcement, 137
- stabilizer axial load, 143
- stabilizer plate, 17, 44, 136, 137, 138, 153
 - angle reinforcement, 137
 - attachment to HSS columns, 138
 - capacity, 144
- Standard Building Code, 2
- standard joist girder seat angles, 131
- Standard Specification for Steel Joists, 1
- steel closure strips, 121
- steel closure strips, ridge and valley plates, 116
- Steel Deck Institute, 7, 19, 20, 24
 - Composite Deck Design Handbook, 21
 - criteria, 19
 - Diaphragm Design Manual, 31
- steel fabrication contract, 152
- Steel Floor and Roof Deck, 7
- Steel Joist Institute, 1, 4, 13, 24
 - Code of Standard Practice, 19, 106, 151, 153, 154
 - Business Relations, 151
 - Estimating, 151
 - Handling and Erection, 151
 - Inspection, 151
 - Joists and Accessories, 151
 - Plans and Specifications, 151
- fifty year digest, 83
- Recommended Code of Standard Practice for Steel Joists and Joist Girders, 150
- specification, 17
- Technical Digest No. 3, "Structural Design of Steel Joist Roofs to Resist Ponding Loads", 96
- Technical Digest No. 5, "Vibration of Steel Joist and Concrete Slab Floors", 91
- Technical Digest No. 9 "Handling and Erection of Steel Joist and Joist Girders", 103, 114, 115, 152
- stiffener, 49, 53, 125, 132, 137, 139
- stiffeners in girder seat, 52
- storage, 154
- story stability, 50, 55, 65, 66, 68
 - story stability check, 50
- strap brace, 38
- strap bracing detail, 42
- structural steel drawings, 150
- structural steel fabricators and erectors, 150
- Structural Wood Fiber Deck, 11
- strut
 - compression, 37, 41
 - drag, 35
- struts, 155
- studs, 28
- sub-purlin, 99
- subcontractors, 154
- superstructure, 155
- supports, elevations, 151
- systems, evaluating an existing, 83

T

- T-reinforcement, 138
- Tech. Report No. 65, "Expansion Joints in Buildings", 15
- temperature, 20
 - data, 14
 - design temperature, 14
 - mean temperature, 14
- temporary protection, 154
- tension flange bracing, 16
- thermal movement, 58
- tie angle thickness, 129
- tie angles, 33, 45
- tie connection, 34

tie plate, 57
 torsion, 100
 torsionally unstable, 43
 tractive force, 70
 transverse cracking, 20
 Tri-Services design manual on seismic design, 30
 tributary areas, 12
 trim pieces, 23

U

UL “Fire Resistance Directory”, 97
 UL 580 Tests for Uplift Resistance of Roof Assemblies, 13
 Underwriters Laboratories, 13
 Uniform Building Code, 2, 52
 unwelded length, 54
 uplift braces, 101

V

vents, 79, 82
 vertical adjustment, 71
 vibration, 24, 25, 82, 91
 critical damping, 91
 duration, 91
 equipment, 92
 isolators, 82
 units, 82
 Vulcraft catalog, 2, 19
 Vulcraft catalog, “Steel Floor and Roof Deck”, 19, 30
 Vulcraft Deck Catalog, 10
 Vulcraft Factory Mutual Data, 9
 Vulcraft VLI-series, 19
 Vulcraft VLP-series, 19
 Vulcraft VLPA-series, 19

W

walkway, 80
 wall, metal system, 16
 wall system, 16

walls
 hard wall system, 36
 masonry, 153
 masonry bearing, 152
 masonry or concrete, 32
 vertical bracing, 55
 web crippling, 20, 22, 67, 139, 140, 144
 web crippling data, 22
 web doubler, 53
 web opening, 25
 web stiffening, 44, 48
 web yielding, 140, 144
 weights of framing, 25
 weld
 arc spot, 29
 E70 fillet, 71
 elongated, 29
 patterns, 29, 57
 welded support fastener, 57
 welded wire fabric, 20
 welding, 87
 field, 45
 welding washers, 30
 welds
 arc spot welds, 119
 capacity, 141, 144
 fillet sizes, 136
 minimum fillet, 128
 puddle welds, 122
 requirements, 128
 stabilizer plate to column flange, 144
 Western Wood Products Association, “Western Woods Use Book”, 31
 wind, 82
 columns, 41
 connections, 52, 55
 forces, 12
 loads, 4, 112
 pressure, 41
 suction, 41
 uplift, 12, 99, 111, 155
 velocity, 12
 wood nailers, 30, 152

X

x-braced roof system, 55
 x-bridging at walls, 98

NOTES

This image shows a single sheet of white paper with horizontal ruling lines. The lines are evenly spaced and run across the width of the page. There are no margins, text, or other markings on the paper.



www.vulcraft.com

NUCOR
VULCRAFT - GROUP