

# conference preview

## JOIST CAUSE

BY JAMES M. FISHER, PE, PhD



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Fisher will deliver the Thursday keynote presentation, "Keys for Successful Designs: Quips and Myths," at NASCC: The Steel Conference, taking place April 22–24 in Atlanta. For more information, see the related News & Events item on page 62.

## Designing single-story buildings with lateral load-resisting frames using joists and joist girders.

**SELECTING THE BEST FRAMING SCHEME** for a building depends on several considerations, not the least of which is the owner's requirements.

It's not possible to give a specific list of rules by which the best scheme can be assured, as every project is different. If "best" means low initial cost, then the owner may face major expenses in the future for operational expenses or problems with expansion. Extra dollars invested at the outset can reduce potential future costs.

Again, every project is different. Here, we'll focus on single-story lateral load-resisting frames using open-web steel joists and joist girders. Of course, both can be used for multistory projects, but single-story buildings comprise most projects. Preliminary design considerations are briefly discussed, as these decisions are paramount to the success of the framing system. As a colleague once told me, "You cannot do just one stupid thing in the design, because once you use bad judgment, additional bad decisions will have to be made."

### Early Decisions

Let's start with some of those early building geometry decisions that must be made in order to get a project started off on the right foot.

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

**Free drainage.** All roofs should be designed and built so that water is not retained on the roof surface. Even in roofs that are constructed with ¼ in. per ft slope, there are instances where free drainage may not occur. A classic example is a roof with no interior drains that drains to an eave gutter. This situation occurs when the first upslope joist or purlin deflects under snow load more than the eave member deflects. Often, the eave member does not deflect as it is supported by the building siding. A check can be made by the specifying professional for ponding stability using the Steel Joist Institute's (SJI) Roof Bay Analysis Tool (read on for more on that tool).

**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, bay sizes ranging from 30 ft × 30 ft to 60 ft × 60 ft have proven economical, and 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. Also, 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-ft by 40-ft or a 30-ft by 50-ft bay, where the 30-ft

dimension is parallel to the long building dimension, often proves to be the most economical. This is because economy is heavily influenced by the wall system when it comes to long, narrow buildings. For example, if a metal wall system is to be used, then the most economical girt system is one in which light-gauge/cold-formed steel girts are used, typically C or Z girts. The maximum span of such girts is approximately 30 ft. If a bay spacing larger than 30 ft is required, then wind columns are required to laterally support the C or Z girts at 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 ft, the designer should investigate the use of steel joists for the girt system as an alternative to wind columns and cold formed purlins.

For structures with a low ratio of perimeter length to area—e.g., square buildings of significant size (200 ft × 200 ft)—the percentage of steel that would be contained in the wall framing is less of a cost factor, and thus a 40-ft × 40-ft bay often proves to be the most economical system. Larger bays of 40 ft × 50 ft, 50 ft × 50 ft or 40 ft × 60 ft 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.

**SJI's Roof Bay Analysis Tool.** This tool assists the specifying professional with optimizing roof bay size as well as determining joist and joist girder depths, spacing, etc. It can also be used to determine whether it is best to span the joist in the long direction or in the shorter direction when a rectangular bay has been selected. The tool can be downloaded for free at [www.steeljoist.org](http://www.steeljoist.org) under the Design Tools tab. The user can input various scenarios to arrive at the least weight or the least cost bay size. Cost data can be input by the user along with other design data. Bays can be evaluated using either ASD or LRFD. In addition, the bay can be evaluated for roof ponding stability, using an iterative analysis. Pull-down menus allow for easy selection of steel deck, joist (K, LH, DLH- Series) and joist girder selections.

**Columns.** Interior columns can normally be braced only at the top and bottom, thus square hollow structural section (HSS) columns are often desirable due to their equal stiffness about both principal axes. Difficult connections with HSS members can be eliminated in single-story frames by placing the joists and joist girders over the tops of the HSS. Other advantages of HSS columns include the fact that they require less paint than equivalent W-shapes and are aesthetically pleasing. W-shapes may be more economical than HSS for exterior columns for the following reasons:

- The wall system (girts) may be used to brace the weak axis of the column.
- Bending moments due to wind loads are predominant about one axis.
- It is easier to frame girt connections to a W-shape than to an HSS section.

**Serviceability.** The design of the lateral load envelope (i.e., the roof bracing and wall support system) must provide for the code-imposed loads, which establish the required strength of the structure. A second category of criteria establishes the serviceability limits of the design. These limits are rarely codified and are often selectively applied project by project based on the experience of the parties involved.

In AISC Design Guide 3: *Serviceability Design Considerations for Steel Buildings* ([aisc.org/dg](http://aisc.org/dg)) several criteria are given for the control of building drift and wall deflection. These criteria, when used, should be presented to the building owner as they help establish the quality of the completed building.

## Joist and Joist Girder Braced Systems

Now let's take a look at some elements of single-story buildings using joists and joist girders.

**Roof diaphragms.** Roof diaphragms used in conjunction with vertical wall bracing is typically the most economical bracing system. Diaphragms are most efficient in relatively square buildings, but an aspect ratio of three to four can be accommodated economically.

**Vertical bracing.** In braced buildings, the roof diaphragm forces are transferred to a vertical braced frame, which in turn transfers the loads to the foundation level. In most cases the vertical bracing is located at the perimeter of the structure so as not to interfere with plant operations. The vertical bracing configuration most frequently used is a X-braced system using angles or rods designed only to function as tension members. However, in areas of high seismicity, a vertical bracing system that incorporates tension/compression members is often required. In these cases, other bracing forms may be used, such as chevron bracing or eccentrically braced frames.

A typical vertical brace using joist girders. The top chord extension is used to eliminate bending in the chord caused by the eccentricity of the shear at the joist seat.



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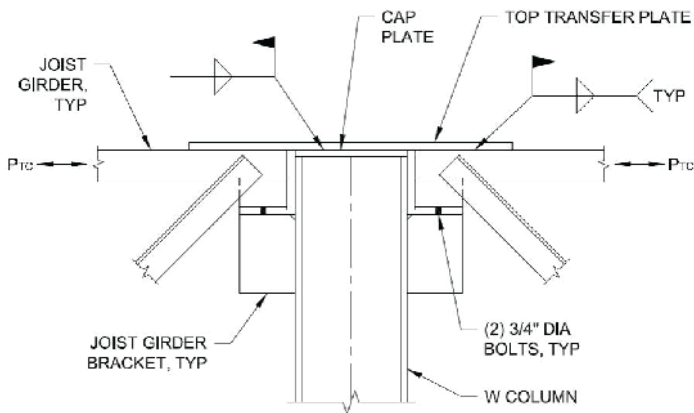


Fig. 1. Force transfer using top plate.

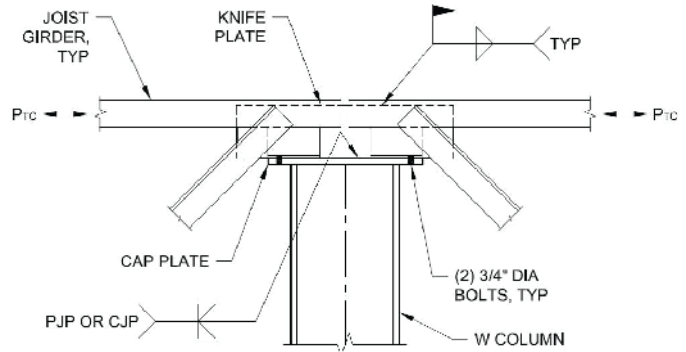


Fig. 2. Force transfer using knife plate.

A typical vertical bracing using joists or joist girders is shown in Figure 1. The top chord extension is used to eliminate the bending in the top chord caused by the eccentricity of the shear at the joist seat.

In buildings with large aspect ratios, bracing may be required in internal bays to reduce the brace forces and to reduce foundation overturning forces. The joist girder details shown in Figures 1 and 2 are ideally suited for diaphragm shear collectors (drag struts). Similar details can be used for joists. And Table 1 shows a typical schedule that can be used to convey loading criteria to the joist manufacturer.

Table 1. Axial Load Joist Girder Schedule

JOIST GIRDER SCHEDULE <sup>1 2 3 4 5</sup>					
Girder Mark Number	Designation (Total Load/Live Load)	Axial Load <sup>6</sup>		Add-Load (kips)	Notes
		Seismic Load 1.0E (kips)	Wind Load 1.0W (kips)		
G1	56G 7N 12.5K/5.8K	160	85	2.0	
G2	56G 7N 14.4K/5.8K	160	85	4.0	

<sup>1</sup> Manufacturer to design joist girders using ASD. Nominal design loads shown are to be used in the applicable ASD code load combinations.

<sup>2</sup> Deflection criteria: Live load deflection  $\leq L/240$ .

<sup>3</sup> See net wind uplift diagram for uplift loads on girders.

<sup>4</sup> See framing plan for additional loads to be included in joist girder design, including mechanical loads.

<sup>5</sup> See framing plan for joist spacing along girder.

<sup>6</sup> Top chord axial load, tension, or compression load.

## Joist Girder Framing Systems

If ordinary moment frames (OMF) are used, lateral stability parallel to the frame is provided by the frame. For loads perpendicular to the frames and for wall bearing and “post and beam” construction, lateral bracing must also be provided. It is important to reemphasize that future expansion may dictate the use of an ordinary moment frame or a flexible (movable) bracing scheme.

Since most single-story structures are light and low in profile, wind and seismic forces are relatively low. Frame action can be easily and safely achieved by providing moment frames on each column line. The most economical situation is to provide the moment frames only at the sidewall columns.

There are several situations for which ordinary moment frames are likely to be superior as compared to braced frames.

- Braced frames may require bracing in both walls and roof. Bracing frequently interferes with plant operations and future expansion. If either consideration is important, ordinary moment frames may be the answer.
- The bracing of a roof system can be accomplished through X-bracing or a roof diaphragm. In either case the roof becomes analytically a large horizontal beam spanning between the walls or bracing which must transmit the lateral loads to the foundations. For large span-to-width ratios (greater than 3:1) the bracing requirements become excessive. A building with dimensions of 100 ft by 300 ft with potential future expansion in the long direction may best be suited for moment frames to minimize or eliminate bracing, which would interfere with future changes.
- Consideration must be given to future expansion and/or modification, where columns are either moved or eliminated. Such changes can generally be accomplished with greater ease where simple-span conditions exist.

However, I would caution designers on the following points:

- The design loads (wind, seismic, and continuity) must be given on the structural plans so that the proper design can be provided by the joist manufacturer. The procedure must be used with conscious engineering judgment and full recognition that standard joist girders are designed as simple-span members subject to concentrated panel point loads (see the *SJI Specification*). Bottom chords are typically sized for tension only. The simple attachment of the bottom chord to a column to provide lateral stability will cause gravity load end moments that cannot be ignored. The designer should not try to select member sizes for these bottom chords since each manufacturer’s design is unique and proprietary.
- It is necessary for the designer to provide a well-designed connection to both the top and bottom chords to develop the induced moments without causing excessive secondary bending moments in the joist chords.
- The system must have adequate stiffness to prevent drift related problems such as cracked walls and partitions, broken glass, leaking walls and roofs, and malfunctioning or inoperable overhead doors.

Table 2. Joist Girder Moment Schedule

SJI TD 11: *Design of Lateral Load Resisting Frames Using Steel Joists and Joist Girders* suggests analysis models that can be used to determine the required joist girder moments. Tables 2 and 3 include schedules that can be supplied to the joist manufacturer for designing the joist girders.

Table 3. Joist or Joist Girder Additional Requirements

Joist Mark Number	J2
Min. Moment of Inertia I <sub>chord</sub> (in. <sup>4</sup> )	1300
Min. Top Chord Thickness (in.)	0.5
Min. Top Chord Horiz. Leg (in.)	5
Min. Bottom Chord Thickness (in.)	0.375
Min. Bottom Chord Horiz. Leg (in.)	4
Min. Seat Angle Thickness (in.)	0.31
Additional Requirements*	Design Joist Girder Webs to transfer Axial loads from Top Chord to Bottom Chord

\*Additional Requirements row is for additional information the engineer of record wishes to convey to the joist manufacturer. The note shown is just one example of the information that can be provided in this column.

SJI provides six different spreadsheets to assist in the design of moment conditions. Each spreadsheet can be used to calculate the strength of connections based on the necessary limit states, includes a reference manual explaining the calculations, and provides for the design of joist girder framing into one side or both sides of the column. The six connection spreadsheets provided are:

- Connection to the Strong Axis of Wide Flange Columns
- Connection to the Strong Axis of Wide Flange Columns—Intermediate Levels
- Connection to the Weak Axis of Wide Flange Columns
- Connection to HSS Columns—Top Plate
- Connection to HSS Columns—Knife Plate
- Connection to Wide Flange Columns—Knife Plates

JOIST GIRDER END MOMENTS (kip-ft) <sup>1</sup> – PART 1 (ROOF)						
Girder Mark Number	Dead Load Moment D		Roof Live Load Moment L <sub>r</sub>		Snow Load Moment S	
	Left	Right	Left	Right	Left	Right
G1	34.0	34.0	30.7	30.7	–	–

JOIST GIRDER END MOMENTS (kip-ft) <sup>1</sup> – PART 2 (ROOF)						
Girder Mark Number	Rain Load Moment R		Wind Moment 1.0W		Seismic Moment 1.0E	
	Left	Right	Left	Right	Left	Right
G1	–	–	±105	±105	±120	±120

<sup>1</sup> End Moment Sign Convention, Positive moments: + 

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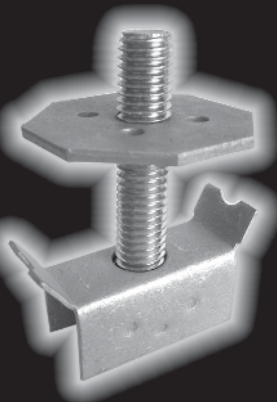




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Although the spreadsheets are specifically written for designing moment connections, they can also be used for cases where joist girder chord axial load transfer is required. As with the Roof Bay Analysis Tool, all of these resources can be downloaded from [www.steeljoist.org](http://www.steeljoist.org) under the Design Tools tab.

### Seismic Joist Girder Frames

The AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341, [aisc.org/specifications](http://aisc.org/specifications))—which apply when the seismic response modification coefficient,  $R$ , (as specified in the applicable building code) is taken greater than 3—require that the joist-girder-to-column moment connections in an OMF be designed for a moment equal to  $1.1R_yM_p$  of the girder, (see Chapter E, Section E1). The limit associated with the maximum moment level in the girder assumes that the columns have more flexural capacity than the girders (i.e., strong column, weak beam). In this system, where the joists typically have more flexural strength than the columns, the fuse in the system would be the column, and the maximum force that can be developed by the system is that force which generates the maximum expected moment ( $M_{pe}$ ) in the column. This moment is equal to  $1.1R_yM_p$  of the column. **Note that this is only required for Seismic Design Categories D, E, and F.** The *Seismic Provisions* requires that the girder- (joist girder in this system) to-column connection has the capacity to resist forces generated in the connection when the column develops this moment. The premise of the OMF frame design for this type of system (strong beam, weak column) is that all columns participating in the lateral load-resisting frame have hinged (or developed  $M_{pe}$ ) just below the bottom chord of the joists. ■

*Want to learn more about key considerations when using open-web steel joists and joist girders in lateral load-resisting systems for wind and seismic loads? Attend the session "A Primer on Lateral Load-Resisting Frames Using Steel Joists and Joist Girders" at the 2020 NASCC: The Steel Conference, presented by Bruce Brotherson with Vulcraft—Nucor and Walter Worthley with Valley Joist. This year's conference takes place April 22–24 in Atlanta. For more information and to register, visit [aisc.org/nascc](http://aisc.org/nascc).*