Design of Lateral Load Resisting Frames Using Steel Joists and Joist Girders

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INTRODUCTION

Procedures for the structural engineer to properly analyze, design and specify steel joist and Joist Girder moment frames to resist seismic lateral loads will be illustrated in this paper. To accomplish this the reader is assumed to be familiar with the design and analysis procedures of ANSI/AISC 360-05 *Specification for Structural Steel Buildings*, ANSI/AISC 341-05 and ANSI/AISC 341s1-05 *Seismic Provisions for Structural Steel Buildings Including Supplement No. 1* and also familiar with the requirements of ASCE/SEI 7-05 *Minimum Design Loads for Buildings and Other Structures*.

The design methodology described in the paper will be limited to single story structures subjected to seismic loads; however, these procedures are also directly applicable to single and multistory moment frames subjected to wind loads.

ANALYSIS REQUIREMENTS

Forces and moments in single story joist rigid frames need to be determined in a manner similar to other Ordinary Moment Frames (OMF) comprised of steel columns and beams; in this context OMF signifies moment frames comprised of steel columns and joists, or Joist Girders. As with all indeterminate frames, the first step in the design process is to perform a preliminary analysis. In general, it is suggested that the OMF be considered as a pinned-base frame in order to eliminate moment resisting foundations; however, for drift control, partially restrained or fixed bases can be considered. The Specifying Professional is encouraged to consider serviceability criteria and drift control at the preliminary design phase of the project. After selecting trial member sizes for the columns and joists, computer analyses need to be performed to determine forces, moments, and deflections (both 1st -order and 2nd -order) for the load combinations prescribed by the applicable building code. The current AISC Specification for Structural Steel Buildings [AISC, 2005a] requires a 2nd -order analysis. Since a 2nd -order analysis is a non-linear problem, the analysis must be performed for each required load combination. Individual load cases cannot be analyzed and the results summed to obtain a correct result; each analysis must be performed using the cumulative, factored loads associated with each load combination.

It is suggested to use a simplified model referred to as Model 1 (see Figure 1) for the joist frame by modeling the joist as an equivalent beam section with an approximate moment of inertia. The node at the interface of the column and joist should be located at the mid-height of the joist to more closely approximate the relative stiffness of these two elements and to more accurately predict lateral drift in the frame.

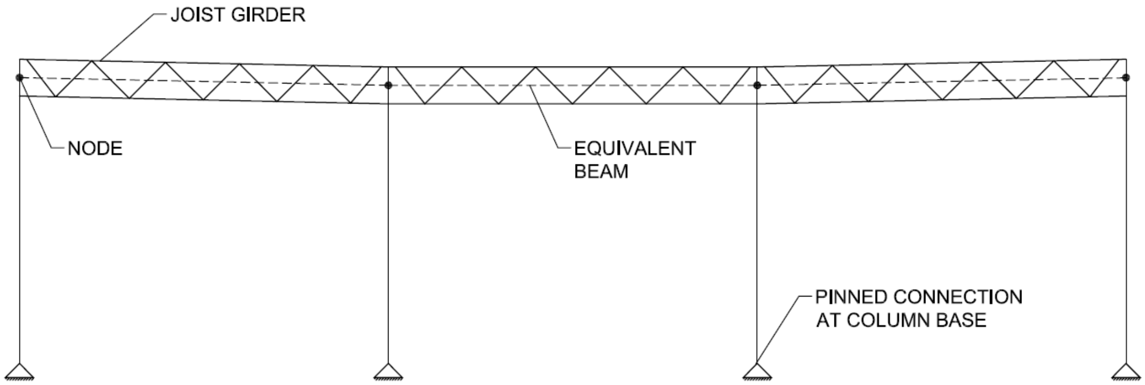


FIGURE 1 SIMPLIFIED STRUCTURAL MODEL 1

For preliminary design the column sizes can be determined by calculating column moments based on assumed shears in the columns from the lateral loads. The maximum moment for a pinned-base column is the moment located at the bottom chord level of the joist. Therefore, the moment can be calculated as the column shear multiplied by the height of the column from the base to the bottom of the joist. In the computer model the moment can be obtained by placing a node at the location of the bottom chord.

Trial joist stiffness can be obtained from the equations for the approximate moment of inertia of a joist or Joist Girder that can be found in the Steel Joist Institute 42nd Edition Catalog [SJI, 2005]. The SJI equation for the approximate moment of inertia of a joist in inches^4 can be found in the introduction to the Standard (ASD or LRFD) Load Tables for **K**-Series, **LH/DLH**-Series joists [SJI, 2005],

$$I_j = 26.767\left(W_{LL}\right)\left(L^3\right)\left(10^{-6}\right) \qquad \text{Eq. 1}$$

where,

W_{LL} = RED figure in the **K**-and **LH/DLH**-Series Load Tables, plf
L = (Span – 0.33), ft. for **K**-Series joists
L = (Clear span + 0.67), ft. for **LH/DLH**-Series joists

The SJI equation for the approximate moment of inertia of a Joist Girder in inches^4 is,

$$I_{JG} = 0.018\text{NPLd (LRFD)} \quad \text{and} \quad = 0.027\text{NPLd (ASD)} \qquad \text{Eq. 2}$$

where,
N = number of panel points
P = pp load at factored load level for LRFD and at nominal load level for ASD, kips
L = girder length, ft.
d = nominal girder depth, in.

The moment of inertia determined above should be reduced by 15% to properly account for the additional shear deflection that can occur in a joist product.

Based on the results of the preliminary analysis, the story drift should be checked first to determine if it meets serviceability criteria. If it does not, then the girder stiffness and column stiffness need to be increased. If the end moments on the joists are greater than the centerline moment of the joist, the size of the joists must be increased and the model must be re-run with new joist stiffness to obtain the correct forces and moments.

In addition to the vertical loads on the frames, the 2nd -order analysis must account for the loads on any "lean-on" columns as well as the weight of any walls supported by the OMF. In lieu of a three-dimensional model, these effects can be modeled by treating the "lean-on" members as pinned-pinned adjacent columns, connected to the frame at the eave with an axial load in the column equal to the required loads on the "lean-on" elements. This approach is illustrated in Figure 2. The effects of the diaphragm deflection on P-Δ can be accounted for by modeling the top of the pseudo-columns displaced laterally an amount equal to the horizontal diaphragm deflection that would occur between the moment frame and the "lean-on" columns.

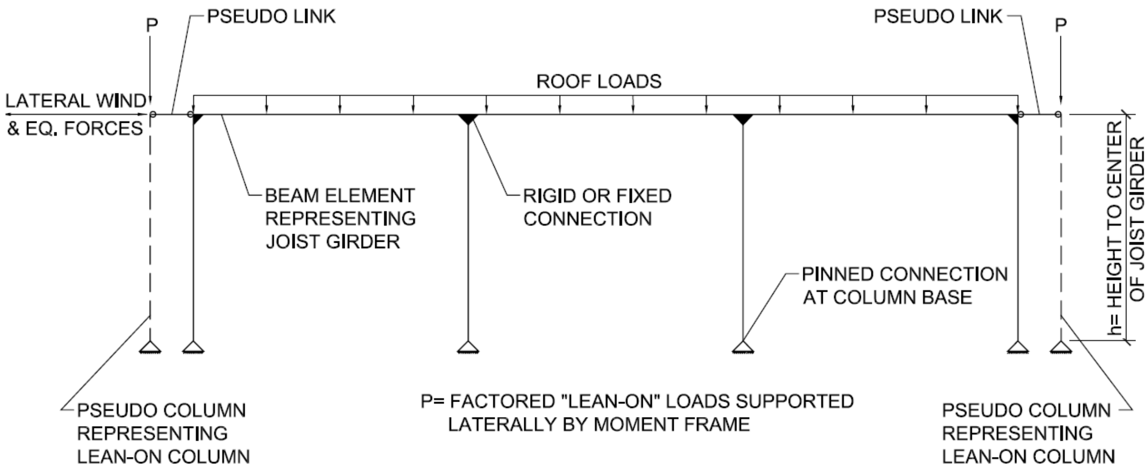


FIGURE 2 MODEL WITH "LEAN-ON" COLUMNS

The Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341-05 [AISC, 2005c], 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, or the maximum moment that can be developed by the system (see ANSI/AISC 341-05, Section 11.2a). 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. This requirement is only required in Seismic Design Categories D, E, and F. The AISC Seismic Provisions require that the girder (joist in this system) to column connection have 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.

The SJI has taken the position that the entire joist, not just the connection, will be designed by the joist manufacturer at this elevated force level because of the importance of ensuring the joist components (i.e. chord or web members) remain elastic and do not buckle. This procedure ensures that the Joist Girder and its members will remain elastic and that buckling will not occur in any of the girder components (i.e. chord or web members).

The moment in the joist to column connection is derived by extrapolating the maximum expected moment in the column (M_{pe}) to the mid-depth of the Joist Girder. For ease of reference, this moment will be referred to as M_{ge}. At an interior column, where moment connected joists are on both sides of the column, the M_{ge} associated with this column will be apportioned to each girder based on Model 2 (see Figure 3).

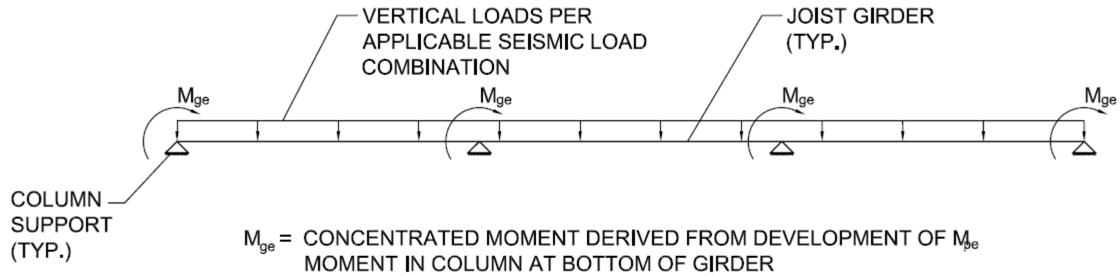


FIGURE 3 STRUCTURAL MODEL 2

The joists can then be analyzed as pinned-supported, continuous members, with concentrated moments equal to M_{ge} at each support. The loads on the joists are derived for loadings consistent with the seismic load combinations required by the applicable building code. Using this model, the appropriate end moments and vertical reactions are determined for the joists.

DESIGN METHODOLOGY FOR SEISMIC LOADS

The design methodology for seismic loads is based on the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 450), 2003 Edition [FEMA, 2004] and on the requirements found in ASCE/SEI 7-05, *Minimum Design Loads for Buildings and other Structures* [ASCE, 2005]. For most steel structures, inelastic behavior is expected if the building is subject to a design-level earthquake. The ability of the structure to withstand this inelastic behavior without collapse is the premise for the majority of the design criteria presented in these references. Essential facilities are designed for higher forces and may have more stringent design requirements. Therefore, the expected level of damage due to a design-level earthquake for these facilities should be less and allow for the continued operation of that facility. It is very possible that even after a design-level earthquake, it may not be economically feasible to repair a building.

Understanding the premise that inelastic behavior is expected in a structure designed to the provisions described in FEMA 450 or ASCE/SEI 7-05 is paramount to understanding the intent of the design requirements. This behavior is acknowledged in the design equations used within the above-noted documents to determine the seismic forces on the building structure. Specifically, the design forces estimated based upon expected ground accelerations during a design-level earthquake are divided by a Response Modification Factor, **R**, which is specific to a given type of construction and framing system. This factor represents an adjustment factor used with a linear analysis model to approximate nonlinear dynamic response in the building structure.

The Response Modification Factor, **R**, incorporates two effects, an overstrength factor and a ductility (or ductility reduction) factor. The overstrength factor accounts for the difference in the force level required to collapse a frame and the seismic design force level for that frame. This overstrength can be attributed to the following:

1. Design efficiency – in general, members are designed with capacities that are equal to or in excess of their design loads.

2. Drift limits, imposed by seismic design criteria and/or serviceability limit states for the building result in larger member sizes than required for strength limit states.
3. The nominal member strengths are larger than design strengths, due to safety factors (**Ω**) or resistance factors (ϕ) and the fact that actual steel yield strengths are typically higher than published for a given grade of steel.
4. The building design may be governed by other (non-seismic) load combinations.
5. Elastic design methodologies define the strength of a frame by the development of the strength of the weakest element (as compared to the design force) in the frame. After the failure (flexural hinging, yielding, buckling, etc.) of this element, most frames have reserve capacity and will continue to resist load until enough members have failed that the structure becomes unstable and collapses. The excess strength is expressed as the difference between this collapse load and the load generating the first failure in an individual element (hinging, yielding, or buckling).

The second effect included in the **R** factor is a ductility or ductility reduction factor. This effect is associated with the following:

- As the structure begins to yield and deform inelastically, the natural period of the building will increase. This increase in period will result in decreased seismic demand for most buildings and will prevent or reduce a resonant response in the building structure.
- Inelastic action in members dissipates energy. This is often referred to as hysteretic damping in the structure (whereas damping in the elastic model would be considered viscous damping).

The combination of these two effects was considered in developing the **R** values that are used today in the United States. The **R** values currently used are based predominantly on engineering judgment and the performance of various materials and systems in past earthquakes. As would be expected, appropriate detailing of the building structure is required to ensure that the **R** value used is justified. The nature of this "appropriate detailing" is the design criteria included in ASCE/SEI 7-05 [ASCE, 2005] and FEMA 450 [FEMA, 2004].

For steel buildings, the International Building Code [ICC, 2006] requires that all buildings in Seismic Design Category D, E, or F adhere to the requirements of ANSI/AISC 341-05, *Seismic Provisions for Structural Steel Buildings* [AISC, 2005c]. For steel buildings in Seismic Design Category A, B, or C, the engineer is provided the choice of using an **R** value of 3 and designing in accordance with ANSI/AISC 360-05, *Specification for Structural Steel Buildings* [AISC, 2005b] or designing with the higher **R** values provided in Chapter 16, Section 1613 Earthquake Loads [ICC, 2006] and adhering to the requirements of ANSI/AISC 341-05. The American Institute of Steel Construction has typically advised the use of the former procedure, since seismic loads (even using an **R** value of 3) will often times be smaller than lateral wind loads on the building structures in moderate or low seismic areas defined by these seismic design categories. In addition, the increased complexity of design, fabrication and erection associated with the seismic provisions will often times offset any material savings obtained by the use of the higher **R** values.

In using this design approach, there are two additional related variables that need to be discussed: The Overstrength Factor (Ω_0), and the Drift Amplification Factor (C_d). The Overstrength Factor (Ω_0) represents the ratio of the estimated maximum potential seismic load to the design seismic load. This factor is typically used to calculate amplified seismic loads for elements of the seismic force resisting system that are sensitive to overstress or where overstress could lead to failure of the structure. The Drift Amplification Factor (C_d) represents the ratio of expected lateral drift in the structure to the drift calculated for the design-level earthquake forces calculated using the Response Modification Factor (**R**). As previously noted, inelastic behavior is truly expected in the structure when subject to a design-level earthquake. Since an elastic model, with reduced forces (reduced by **R**) are used, the calculated lateral displacements from this model are amplified by the factor (C_d) to account for this inelastic behavior. As with the Response Modification Factor, both of these factors vary with the type of construction and framing system selected.

Structural System Selection

ASCE/SEI 7-05 Table 12.2.1 "Design Coefficients and Factors for Seismic Force-Resisting Systems" [ASCE, 2005] lists various types of Seismic Force-Resisting Systems, the associated factors (**R**, Ω_0, and C_d) that are to be used with these systems and the limitations on the use of these systems. For steel moment frame systems, three categories of systems are noted. These are: Special Moment Frames, Intermediate Moment Frames and Ordinary Moment Frames. Special Moment Frames and Intermediate Moment Frames require the use of a moment connection between the beam and column that has been demonstrated by virtue of testing to allow for varying degrees of inelastic rotation without significant degradation in the flexural strength of the two members. This type of testing has not been performed on joist-column moment connections at this time. Tests have been conducted only on standard connections. There is potential for developing a connection that would achieve inelastic rotation, but as of yet no standardized connection has been developed for this purpose. Therefore, a joist moment frame must be categorized as an Ordinary Moment Frame (OMF). Ordinary Moment Frames are expected to withstand minimal inelastic deformations in their members and connections when subjected to a design-level earthquake. Fully restrained, FR, moment connections in Ordinary Moment Frames are to be designed for a required flexural strength equal to the maximum expected flexural strength of the beam (i.e. $1.1R_yM_{p(beam)}$) or the maximum moment that can be delivered by the system. For a joist moment frame, where the joist has a flexural strength that exceeds the column flexural strength, the maximum moment that can be delivered by the system is the maximum expected flexural strength of the column (i.e. $1.1R_yM_{p(column)}$). The use of this type of system is limited to a one-story building where the hinging of the column will not immediately create a stability problem. The use of this type of system is also limited to the height and other constraints noted in Table 12.2.1.

Analytical research conducted at the University of Minnesota on the design of trussed frames subjected to seismic loading [Beckman, 1996] concluded that these types of frames would be feasible for low-rise, multi-bay structures especially in lower seismic zones. The study also concluded that the cost effective nature of Joist Girders would provide a significant advantage to using rigid trussed frames. Since this study was strictly analytical, one of its recommendations was that experimental testing be carried out to demonstrate the viability of using Joist Girders in frames that would be capable of withstanding earthquake forces. Subsequently, the SJI sponsored a full-scale experimental research program followed by extensive analytical studies at the Georgia Institute of Technology from 2000 to 2004 [Kim, 2003, Kim et. al, 2007]. This research has substantiated the use of a Joist Girder frame system, as part of an Ordinary Moment Frame, for use in any Seismic Design Category for structures within the limits for OMF's, and has validated the design approach recommended in SJI Technical Digest No. 11, *Design of Lateral Load Resisting Frames Using Steel Joists and Joist Girders* [SJI, 2007].

Welding Requirements

The joist manufacturer must be aware of special weld requirements as imposed by ANSI/AISC 341-05, *Seismic Provisions for Structural Steel Buildings* [AISC 2005c]. It is the opinion of the authors that the welding requirements cited below are required for moment frames (OMF) when the structure is located in Seismic Design Category D, E, or F, or in Seismic Design Category A, B, or C, if **R** values greater than 3 are used.

- **Section 7 Connections, Joists, and Fasteners** of the AISC Seismic Provisions is to be followed for chord splices in the SLRS, and for the connections between the joists in the SLRS and the columns.
- For the fillet welded connections between the joist chords and web members in the SLRS, **Section 7.3 Welded Joints** shall be followed with the exception that the welding is performed per SJI requirements. In addition, **Section 7.3a General Requirements** shall be followed in its entirety.
- From **Appendix W2. Structural Design Drawings and Specifications, Shop Drawings and Erection Drawings:**

W2.1. Structural Design Drawings and Specifications
Structural design drawings and specifications shall include, as a minimum, the following information:

(1) locations where backup bars are required to be removed
(2) locations where supplemental fillet welds are required when backing is permitted to remain
(3) locations where fillet welds are used to reinforce groove welds or to improve connection geometry
(4) locations where weld tabs are required to be removed
(5) splice locations where tapered transitions are required

W5.4. Maximum Interpass Temperatures
Maximum interpass temperatures shall not exceed 550°F (290°C), measured at a distance not exceeding 3 in. (75 mm) from the start of the weld pass. The maximum interpass temperature may be increased by qualification testing that includes weld metal and base metal CVN testing using AWS D1.1 Annex III. The steel used for the qualification testing shall be of the same type and grade as will be used in production.

The maximum heat input to be used in production shall be used in the qualification testing. The qualified maximum interpass temperature shall be the lowest interpass temperature used for any pass during qualification testing. Both weld metal and HAZ shall be tested. The weld metal shall meet all the mechanical properties required by Section 7.3a, or those for demand critical welds of Section 7.3b, as applicable. The heat affected zone CVN toughness shall meet a minimum requirement of 20 ft-lbf (27 J) at 70°F (21°C) with specimens taken at both 1 and 5 mm from the fusion line.

Moment Connections

Figure 4 shows a recommended joist to interior column connection for a seismic Ordinary Moment Frame. The same detail would be recommended for a wind moment frame when the wind end moments produce chord axial forces greater than 40-45 kips. The timing of the bottom chord to column stabilizer plate weld must be specified to ensure that it matches the load combination moments that would be provided in Table 1. A similar connection detail can be used for an exterior column connection by having the connection only on one side of the column.

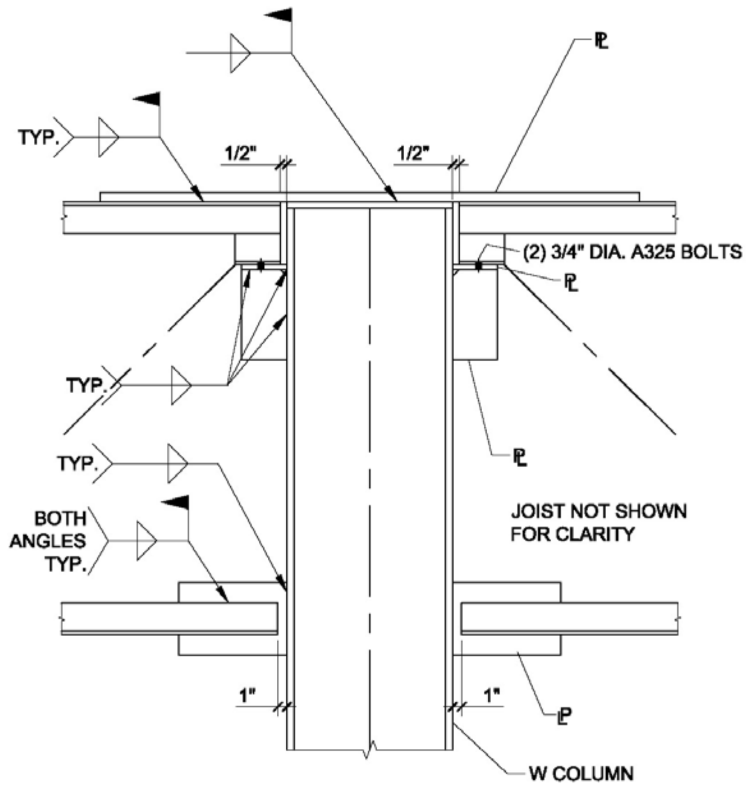


FIGURE 4 MOMENT CONNECTION TO AN INTERIOR COLUMN

SPECIFICATION OF LOADS AND OTHER DESIGN REQUIREMENTS

The Specifying Professional must provide design information to the joist manufacturer so that the joists can be designed to meet the project requirements. The results of the analyses must be specified for each of the required load combinations. In addition, stiffness requirements must be specified to the joist manufacturer, since each analysis result is dependent on the stiffness of the columns and joists in the moment frames.

Specifying Design Loads and Load Combinations

For a clear definition of loads for joists used as part of the lateral load resisting system, the following guidelines should be followed:

I. All externally applied loads should be defined by Load Category (Live, Dead, Snow, Wind, Earthquake, Collateral, etc.).

a. Avoid use of pre-combined load callouts such as 'Total Load', 'Factored Load', or 'Net Uplift Load', as these cannot be readily separated into their various load components, for correct assembly of load combinations with appropriate multipliers.
b. For Dead Loads, if Net Uplift is a design consideration, be sure to include both a maximum dead load for inclusion with gravity loads, and a minimum dead load for inclusion with upward acting loads. One convenient method of managing this is to specify the minimum dead as Dead Load (D) and specify the difference between minimum dead and maximum dead as Collateral Load (C).

II. System internal forces which behave linearly, and may be algebraically summed, such as strut forces from a deck diaphragm, braced frame, shear wall, etc. should also be defined by Load Category, just the same as the externally applied loads.

III. All potentially controlling design load combinations must be specified to the joist manufacturer, for investigation during the design of the joists and girders. Along with the required joist design load combinations, the Specifying Professional must also indicate whether the design procedure is to be ASD or LRFD. Either method may be specified, but it is important for the load combinations and design methodology to be properly aligned.

IV. In determining which load combinations may be potentially controlling, it is important to consider the individual components which make up the joist, and the load combinations which may result in a maximum tensile force, compressive force, or flexural moment for each individual component. Note for example that for the Joist Girder design presented in Appendix A, the column plastic moment conditions control the design of the top and bottom chords as well as all primary (diagonal) webs. However, the largest gravity load combination controls the design of the secondary (vertical) webs.

V. In determining which load combinations may be potentially controlling for a given joist component, it is also important to remember that Wind and Seismic Loads are included in completely separate load combinations with completely different vertical loads. Thus, although one type of lateral load may be significantly larger than the other, both may need to be considered in the design of the joist. Also, for joists which are considered to be a seismic collector element, both E and E_m values must be specified for inclusion in different load combinations, unless one is eliminated by the Specifying Professional as not being a potentially controlling design load combination.

VI. Due consideration must be given to multiple lateral load directions for each potentially controlling code-specified load combination. In order to adequately specify all potentially controlling design load combinations, the Specifying Professional may find it necessary to list the same basic load combination multiple times for consideration of lateral loads acting in different directions. Also, if wind uplift forces are different for different wind directions, then both wind uplift values should be listed for consideration in appropriate load combinations.

VII. System internal forces which behave non-linearly, such as joist end moments and axial loads determined via a 2^{nd} -order frame analysis (as required by the 2005 AISC Specification) must be specified for each individual load combination. These 2^{nd} -order analysis system internal forces do not behave linearly, and therefore cannot be algebraically summed by the joist manufacturer.

VIII. For joists used as part of a moment resisting frame, consideration and appropriate specification must also be provided for joist end moments and axial loads resulting from load combinations which include only gravity load categories, with no lateral loads. As it is difficult to design a connection which will be moment resisting for lateral load combinations and pinned for gravity load combinations, most moment resisting frames must also consider effects of joist end fixity under gravity loads.

IX. Due consideration must also be given to managing end fixity and associated joist end moments, resulting from applied dead loads. Although there is historical precedence for specifying that connections between joist bottom chords and columns not be welded until after all dead loads have been applied, from a practical standpoint, this is often not feasible. By the time roofing membranes and interior partitions are installed, the steel erector is usually long gone from the jobsite, and by the time interior soffits and suspended ceilings are hung, the joists and Joist Girders are inaccessible. It is usually more practical to simply include all dead loads, except joist self-weight, in the determination of joist end moments, and require the connections to be fully welded as soon as all columns are plumbed, and before adding any further loads to the structure. The instructions given to the joist manufacturer for designing the joists need to match the instructions given to the steel erector for what stage the joist connections are to be welded.

In addition, stiffness requirements must be specified to the joist manufacturer, since the analysis is dependent on the stiffness of the columns and joists in the moment frames. The assumed joist moment of inertia (I_{xx}) should be specified, along with a tolerance range (+/- 10% is usually considered sufficient). The joist manufacturer will then include the target joist moment of inertia in the selection of chord materials. If, for some reason, the target cannot be achieved within the specified range, then the joist manufacturer must communicate the discrepancy to the Specifying Professional, for further design coordination.

Shown below are the Basic LRFD Load Combinations from ASCE/SEI 7-05 Section 2.3.2 [ASCE, 2005]. Similar combinations can be found in ASCE/SEI 7-05 Section 2.4.1 for ASD.

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

In addition to these combinations, Chapter 12, "Seismic Design Requirements for Building Structures" in ASCE/SEI 7-05 [ASCE, 2005] contains the information needed to design for the seismic load effects and where specifically required, how these effects are to be modified to account for system overstrength. The seismic load effect, E, shall be taken as: $E = E_h + E_v$ in the Basic LRFD Load Combination 5 while $E = E_h - E_v$ in the Basic LRFD Load Combination 7, where E_h and E_v are the effects of horizontal and vertical seismic forces, respectively.

$$E_h = \rho Q_E$$

$$E_v = 0.2 S_{DS} D$$

and Load Combinations 5 and 7 become,

5. $(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$
7. $(0.9 - 0.2S_{DS})D + \rho Q_E + 1.6H$

Where conditions require that the overstrength factor be applied, the seismic load effect, E, shall be taken equal to E_m and $E_m = E_{mh} + E_v$ in the Basic LRFD Load Combination 5 while $E_m =$

$E_{mh} - E_v$ in the Basic LRFD Load Combination 7, where E_{mh} and E_v are the effects of horizontal seismic forces including overstrength and vertical seismic forces, respectively.

$$E_{mh} = \Omega_o Q_E$$

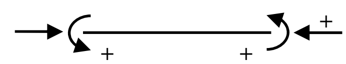
and Load Combinations 5 and 7 become,

5. $(1.2 + 0.2S_{DS})D + \Omega_o Q_E + L + 0.2S$
7. $(0.9 - 0.2S_{DS})D + \Omega_o\, Q_E + 1.6H$

For single story moment resisting frames several of the load combinations can be simplified or eliminated by the designer based on his or her experience that they will not control the design.

Table 1 shown below is an example LRFD schedule that often controls. Note that although multiple load category considerations are shown in a single tabulated line such as (L_r or S) or (W or 0.70E), this is simply to show potentially controlling conditions. In specifying joist loads for a real project, the Specifying Professional must either determine which of these is controlling, and display only one, or else must display both as separate potentially controlling load combinations to be investigated by the joist manufacturer. For load combinations which include wind or seismic, it may be necessary to break these down further into separate load combinations considering different directions of lateral loading. End moments are affected by the amount of the dead load to be resisted in the moment frame. Consequently and as previously discussed, timing of the bottom chord to column stabilizer plate weld can affect the magnitude of the moments in the SLRS. If the end moments in Table 1 are calculated with less than 100% of the dead load applied, a note stating when this weld is to be made is required.

TABLE 1 LRFD LOAD COMBINATION SCHEDULE FOR JOIST GIRDER

Mark: G1	**Girder Designation: 48G8NSP**					
LRFD Load Combination:	Panel Load (kips)	Left End Moment (kip-ft.)	Right End Moment (kip-ft.)	TC Force (kips)	BC Force (kips)	Remarks
1.4D + 1.4C						
1.2D + 1.2C + 1.6(L_r or S)						
1.2D + 1.2C + 1.6W + 0.5(L_r or S)						
1.2D + 1.2C + 1.0E +0.2S						
$(1.2 + 0.2S_{DS})$ (D+C) + ρQ_E + 0.2S						
0.9D + 1.6W						

Presentation of Loads

Although useful for comparison and verification purposes, the Panel Load field, shown in Table 1 for each load combination, is only applicable to very simple loading conditions with equal loads equally spaced. In general, externally applied loads for lateral load resisting joists are more clearly communicated by specifying design loads by category and allowing the joist manufacturer to appropriately sum the loads per the specified load combinations.

There are many instances where the Joist Girder loads are not uniformly spaced, or where the loads along the length of the Joist Girder are not equal. The Specifying Professional can indicate these loads in various ways. One method is to use a load diagram and load schedule as shown in Table 2. This method has the advantage of presenting loads very clearly and concisely for each girder and works very well for projects with design loads that are well defined early in the project and unlikely to change. The primary disadvantage of this method is the difficulty of revising load diagrams for changes to design loads late in the project, or managing loads for which final magnitudes and locations may not be determined until late in the project (such as roof mechanical equipment or sprinkler mains). These types of late load revisions can become quite cumbersome on projects with complex loading, requiring numerous different Joist Girders with different design load criteria.

For projects with more complex loading, it may be easier to simply specify the base loads for each load category in psf, then show additional loads either on the roof/floor framing plan, on key plans (such as wind pressure plan diagrams and snow drift plan diagrams), or include in notes and diagrams keyed to the roof/floor framing plan.

TABLE 2 JOIST GIRDER LOAD SCHEDULE

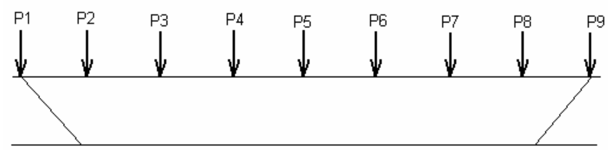
Mark: G1	**Girder Designation: 48G8NSP**								
Load Category:	**P1 (kips)**	**P2 (kips)**	**P3 (kips)**	**P4 (kips)**	**P5 (kips)**	**P6 (kips)**	**P7 (kips)**	**P8 (kips)**	**P9 (kips)**
Dead Load (D)									
Collateral Load (C)									
Roof Live Load (L_r)									
Snow Load (S)									
Wind Load (W) (Windward)									
Wind Load (W) (Leeward)									

Table 3 shows an example format for a Main Wind Force Resisting System Design Pressure Table. This is a very simple format for displaying the different design wind pressures for different roof zones. The vertical pressures listed in the table would be applicable to the roof Joist Girders. Roof joists would normally be considered Components and Cladding and would require a separate table and/or key plan wind pressure diagram.

TABLE 3 MAIN WIND FORCE RESISTING SYSTEM DESIGN PRESSURE TABLE

	DESCRIPTION	**DESIGN PRESSURE (psf)**
HORIZONTAL	MAXIMUM COMBINED WINDWARD AND LEEWARD WALL PRESSURE - INTERIOR ZONE - END ZONE (20 ft WIDE)	 **14.5** **21.8**
	MAXIMUM WINDWARD WALL PRESSURE - INTERIOR ZONE - END ZONE (20 ft WIDE)	 **12.3** **16.7**
	MAXIMUM LEEWARD WALL PRESSURE - INTERIOR ZONE - END ZONE (20 ft WIDE)	 **-9.9** **-12.8**
VERTICAL	MAXIMUM WINDWARD ROOF PRESSURE - INTERIOR ZONE - END ZONE (20 ft WIDE)	 **-18.2** **-26.2**
	MAXIMUM LEEWARD ROOF PRESSURE - INTERIOR ZONE - END ZONE (20 ft WIDE)	 **-11.6** **-15.0**

Special Design Criteria related to Seismic Detailing

In order for the joist manufacturer to properly design the joists for the seismic requirements, in addition to the force and stiffness requirements, the manufacturer must also know certain facts about the SLRS as designed by the Specifying Professional. The Specifying Professional is required to designate on the Structural Design Drawings, and/or in the Project Specifications, the items listed in Section 5.1 of the AISC Seismic Provisions for Structural Steel Buildings [AISC, 2005c], if the Seismic Design Category is other than A, B, or C, or if an R value greater than 3.0 is used for the design. No special requirements are imposed for Seismic Design Category A, B, or C, if an R value of 3 or less is used for the design.

From the following list of items taken from Section 5.1 of the AISC Seismic Provisions, several are important to the joist manufacturer:

(1) Designation of the *seismic load resisting system* (SLRS)
(2) Designation of the members and connections that are a part of the SLRS
(3) Configuration of the connections
(4) Connection material specifications and sizes
(5) Locations of *demand critical welds*
(6) Locations and dimensions of *protected zones*
(7) Welding requirements as specified in Appendix W, Section W2.1

It should be noted that for an OMF there are no protected zones. If the Specifying Professional imposes Q/C welding requirements then the joist supplier must be notified as these requirements can have a major impact on manufacturing costs.

If the applicable building code requires the use of the AISC Seismic Provisions for Structural Steel Buildings [AISC, 2005c] and the joist is a part of the SLRS, the requirements previously described and summarized below must be followed.

Section 7.1 of the Seismic Provisions requires a ductile limit state to govern design. For bolted splices, fracture limit states cannot govern, and bolt shear cannot govern. It is also implied that weld strengths should not govern. Therefore, the controlling limit state must be either yield of the member or bearing of bolts on connected elements.

Section 7.2 of the Seismic Provisions states for bolted joints, "All bolts shall be pretensioned high strength bolts and shall meet the requirements for *slip-critical* faying surfaces in accordance with the *Specification* Section J3.8 with Class A surface. Bolts shall be installed in standard size holes or in short-slotted holes perpendicular to the applied load." The *Specification* being referred to is the AISC Specification for Structural Steel Buildings [AISC, 2005b]. It should be noted that the faying surface requirement is not required for end plate connections. Section 7.2 also states, "The *available shear strength* of bolted joints using standard holes shall be calculated as for bearing-type joints in accordance with *Specification* Sections J3.7 and J3.10, except that the nominal bearing strength at bolt holes shall not be taken greater than $2.4dtF_u$."

For welds, the requirements of Section 7.3 of the Provisions must be followed. Specifically, this requires welding to be performed in accordance with the American Welding Society, AWS D1.8 Structural Welding Code – Seismic Supplement [AWS, 2005], and electrodes must meet certain minimum Charpy V-Notch (CVN) toughness requirements. Joists incorporated into SMF systems would be required to meet this criteria. However, it is the opinion of the SJI and the authors that joists incorporated into horizontal diaphragms as collectors or chords, need only to adhere to these requirements for end connections and any splices in the chords if the seismic forces do not go through the web members. Welded connections of web members to the top chord and any bottom chord welds would therefore not need to meet these criteria. It should be noted that it is usually cost effective for the joist manufacturer to use full length chord material to avoid splice requirements.

If the joists are a part of an Ordinary Moment Frame (OMF) the requirements of Section 11 of the AISC Seismic Provisions for Structural Steel Buildings [AISC, 2005c] must be followed. For the fully restrained (FR) moment connections the requirements of Section 11.2a must be followed.

The joist manufacturer must list on the Erection Drawings the following items from Section 5.3 of the AISC Seismic Provisions for Structural Steel Buildings [AISC, 2005c]:

(1) Designation of the members and connections that are part of the SLRS
(2) Field connection material specifications and sizes, if applicable
(3) Locations of pretensioned bolts, if applicable
(4) Field welding requirements as specified in Appendix W, Section W2.3, if applicable

LATERAL BRACING

It is very important that the joists that are a part of the SLRS be properly laterally braced. The requirements provided here are based on the 2005 AISC Specification for Structural Steel Buildings [AISC, 2005b] Appendix 6 Stability Bracing for Columns and Beams. The calculations and details for the lateral bracing of the joists are generally provided by the joist manufacturer.

Joists generally require braces adjacent to each column in the SLRS in order to prevent buckling of the bottom chord where the bottom chord is in compression. The columns require a brace at the location of the joist stabilizer plate to prevent lateral buckling of the column at the assumed plastic hinge location. It is suggested that the bottom chord bracing strength and stiffness be based on the nodal bracing requirements of the Specification.

The required brace strength is

$$P_{br} = 0.01P_r \qquad \text{AISC A-6-3}$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{8\,P_r}{L_b}\right) \text{ (LRFD)} \qquad \beta_{br} = \Omega\left(\frac{8\,P_r}{L_b}\right) \text{ (ASD)} \qquad \text{AISC A-6-4}$$

where,

P_r = axial compressive strength in the chord
ϕ = 0.75 (LRFD), Ω = 2.00 (ASD)
L_b = distance between braces (the unbraced length)

It should noted from the AISC Specification that when L_b is less than L_q, where L_q is the maximum unbraced length for the required column or chord force with K = 1.0, then L_b is permitted to be taken equal to L_q. This provision can be very beneficial in reducing the bracing stiffness required.

For moment frames using Joist Girders, the lateral bracing for the column, and for the Joist Girder bottom chord generally consists of single angles extending from the brace point on the column or bottom chord up to the bottom chord of a supported joist. The stiffness of the bracing system is thus dependent upon the stiffness of the angle bracing member, its attachments, and the stiffness of the joist to which the brace is attached. The stiffness of the single angle comes from its axial stiffness and can typically be neglected when compared to the stiffness of the joist in bending.

Assuming that the joist is attached to a roof diaphragm the stiffness of the joist can be determined from the deflection of the joist due to a 1 kip load applied vertically to the joist at the location where the brace is attached to the joist.

The following simply-supported beam equation from the AISC Manual of Steel Construction [AISC, 2005a], Table 3-23 for the case of a simple beam – concentrated load at any point, can be used to determine the deflection that would occur for a concentrated load acting at any point on a joist.

$$\Delta = \frac{Pa^2b^2}{3EI_jL} \qquad \text{Eq. 3}$$

where,

a = the distance from the end of the joist to where the brace is attached to the joist

b = L – a

I_j = Approximate joist stiffness from Eq. 1

Thus the joist stiffness equals

$$\beta = \frac{P}{\Delta} = \frac{3EI_jL}{a^2b^2} \qquad \text{Eq. 4}$$

when the load P is equal to 1 kip.

Figure 5 shows a typical bracing detail for the Joist Girder bottom chord and Figure 6 shows a typical bracing detail for the column at the expected location where the plastic hinge would form. The bracing member for the Joist Girder (see Figure 5) should be placed at or near a bottom chord panel point.

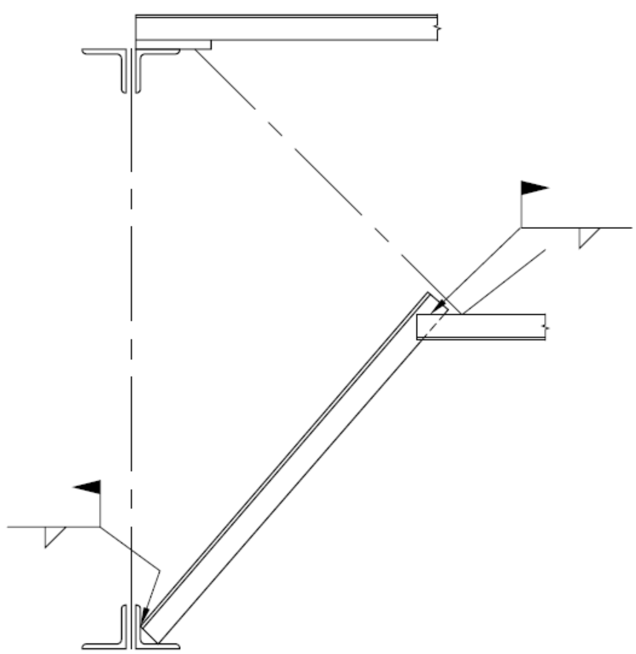


FIGURE 5 JOIST GIRDER BRACE

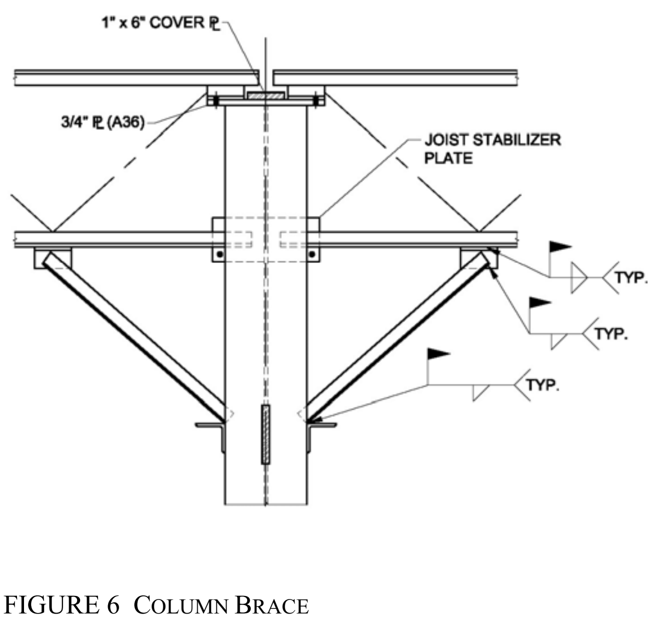


FIGURE 6 COLUMN BRACE

CONCLUSION

Two models are suggested for lateral load resisting frames using steel joists and Joist Girders. The first model (Model 1) is an elastic model of the entire frame used to evaluate member forces (including joist end moments and shears), reactions and story drifts. This model is used for column design and used to determine the joist end moments and shears for non-seismic load combinations and for seismic load combinations for structures in Seismic Design Categories A, B, and C. Second-order effects need to be accounted for in these analyses.

The second model (Model 2) is used to determine the joist end moments and connection forces consistent with the seismic design philosophy for an OMF system in Seismic Design Categories D, E, and F. In this model, the columns are assumed to have formed plastic hinges, and therefore, the model consists of a continuous, pinned-supported girder with the appropriate M_{ge} moments applied at each support. This model is only evaluated for vertical loads consistent with the seismic load combinations prescribed by the applicable building code for structures in Seismic Design Categories D, E, and F.

REFERENCES

American Institute of Steel Construction (2005a), **Steel Construction Manual**, Thirteenth Ed., AISC, Chicago, IL.

American Institute of Steel Construction (2005b), ANSI/AISC 360-05, *Specification for Structural Steel Buildings, March 9, 2005*, AISC, Chicago, IL.

American Institute of Steel Construction (2005c), ANSI/AISC 341-05, *Seismic Provisions for Structural Steel Buildings, March 9, 2005*, Including Supplement No. 1, November 16, 2005, AISC, Chicago, IL.

American Institute of Steel Construction (2005d), ANSI/AISC 358-05, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, December 13, 2005*, Including Supplement No. 1, November 16, 2005, AISC, Chicago, IL.

American Society of Civil Engineers (2005), ASCE/SEI 7-05, *Minimum Design Loads for Buildings and other Structures*, Including Supplement No. 1, ASCE, Reston, VA.

American Welding Society (2005), ANSI/AWS D1.8/D1.8M *Structural Welding Code – Seismic Supplement*, AWS, Miami, FL.

American Welding Society (2006), ANSI/AWS D1.1/D1.1M *Structural Welding Code – Steel*, AWS, Miami, FL.

Beckman, M. W. (1996), "Seismic Analysis and Design of Multi-bay Rigid Trussed Frames," MS Thesis, Department of Civil Engineering, University of Minnesota, Minneapolis, MN.

Federal Emergency Management Agency (2001), *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 369)*, 2000 Edition, Part 2: Commentary, Building Seismic Safety Council, National Institute of Building Sciences, Washington, D.C.

Federal Emergency Management Agency (2004), *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450)*, 2003 Edition, Part 1: Provisions, Building Seismic Safety Council, National Institute of Building Sciences, Washington, D.C.

International Code Council (2006), *International Building Code*, ICC, Falls Church, VA.

Kim, U. S. (2003), "Seismic Behavior of Steel Joist Girder Structures," PhD Dissertation, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA.

Kim, U. S., Leon, R. T., and Galambos, T. V. (2007), "Seismic Design of Steel Joist Girder Structures", AISC Engineering Journal, Third Quarter.

SJI (2005), **42nd Edition Catalog** Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders: **K**-Series, **LH**-Series, **DLH**-Series, Joist Girders, Steel Joist Institute, Myrtle Beach, SC.

SJI (2007), Technical Digest No. 11, **Design of Lateral Load Resisting Frames Using Steel Joists and Joist Girders,** Steel Joist Institute, Myrtle Beach, SC.