STANDARD SPECIFICATIONS
FOR JOIST GIRDERS

Adopted by the Steel Joist Institute November 4, 1985
Revised to November 10, 2003 - Effective March 01, 2005

SECTION 1000.
SCOPE

This specification covers the design, manufacture and use of Joist Girders. Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD) are included in this specification.

SECTION 1001.
DEFINITION

The term "Joist Girders", as used herein, refers to open web, load-carrying members utilizing hot-rolled or cold-formed steel, including cold-formed steel whose yield strength* has been attained by cold working.

The design of Joist Girder chord and web sections shall be based on a yield strength of at least 36 ksi (250 MPa), but not greater than 50 ksi (345 MPa). Steel used for Joist Girder chord or web sections shall have a minimum yield strength determined in accordance with one of the procedures specified in Section 1002.2, which is equal to the yield strength assumed in the design. Joist Girders shall be designed in accordance with this specification to support panel point loadings.

* The term "Yield Strength" as used herein shall designate the yield level of a material as determined by the applicable method outlined in paragraph 13.1, "Yield Point" and in paragraph 13.2, "Yield Strength", of ASTM Standard A370, "Standard Test Methods and Definitions for Mechanical Testing of Steel Products", or as specified in Section 1002.2 of this Specification.

Standard Specifications and Weight Tables for Joist Girders

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SECTION 1002.
MATERIALS

1002.1 STEEL

The steel used in the manufacture of chord and web sections shall conform to one of the following ASTM Specifications:

- Carbon Structural Steel, ASTM A36/A36M.
- High-Strength, Low-Alloy Structural Steel, ASTM A242/A242M.
- High-Strength Carbon-Manganese Steel of Structural Quality ASTM A529/A529M, Grade 50.
- High-Strength Low-Alloy Columbium-Vanadium Structural Steel, ASTM A572/A572M Grade 42 and 50.
- High-Strength Low-Alloy Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 inches (100 mm) Thick, ASTM A588/A588M.
- Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Corrosion Resistance, ASTM A606.
- Steel, Sheet, Cold-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability, ASTM A1008/A1008M.
- Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability, ASTM A1011/A1011M.

or shall be of suitable quality ordered or produced to other than the listed specifications, provided that such material in the state used for final assembly and manufacture is weldable and is proved by tests performed by the producer or manufacturer to have the properties specified in Section 1002.2.

1002.2 MECHANICAL PROPERTIES

The yield strength used as a basis for the design stresses prescribed in Section 1003 shall be at least 36 ksi (250 MPa), but shall not be greater than 50 ksi (345 MPa). Evidence that the steel furnished meets or exceeds the design yield strength shall, if requested, be provided in the form of an affidavit or by witnessed or certified test reports.

For material used without consideration of increase in yield strength resulting from cold forming, the specimens shall be taken from as-rolled material. In the case of material properties of which conform to the requirements of one of the listed specifications, the test specimens and procedures shall conform to those of such specifications and to ASTM A370.

In the case of material the mechanical properties of which do not conform to the requirements of one of the listed specifications, the test specimens and procedures shall conform to the applicable requirements of ASTM A370 and the specimens shall exhibit a yield strength equal to or exceeding the design yield strength and an elongation of not less than (a) 20 percent in 2 inches (51 millimeters) for sheet and strip, or (b) 18 percent in 8 inches (203 millimeters) for plates, shapes and bars with adjustments for thickness for plates, shapes and bars as prescribed in ASTM A36/A36M, A242/A242M, A529/A529M, A572/A572M, A588/A588M, whichever specification is applicable on the basis of design yield strength.

The number of tests shall be as prescribed in ASTM A6/A6M for plates, shapes, and bars; and ASTM A606, A1008/A1008M and A1011/A1011M for sheet and strip.

If as-formed strength is utilized, the test reports shall show the results of tests performed on full section specimens in accordance with the provisions of the AISI Specifications for the Design of Cold-Formed Steel Structural Members and shall indicate compliance with these provisions and with the following additional requirements:

a) The yield strength calculated from the test data shall equal or exceed the design yield strength.

b) Where tension tests are made for acceptance and control purposes, the tensile strength shall be at least 6 percent greater than the yield strength of the section.

c) Where compression tests are used for acceptance and control purposes, the specimen shall withstand a gross shortening of 2 percent of its original length without cracking. The length of the specimen shall not be greater than 20 times its least radius of gyration.

d) If any test specimen fails to pass the requirements of the subparagraphs (a), (b), or (c) above, as applicable, two retests shall be made of specimens from the same lot. Failure of one of the retest specimens to meet such requirements shall be the cause for rejection of the lot represented by the specimens.

1002.3 WELDING ELECTRODES

The following electrodes shall be used for arc welding:

a) For connected members both having a specified yield strength greater than 36 ksi (250 MPa).

AWS A5.1: E70XX
AWS A5.5: E70XX-X
AWS A5.17: F7XX-EXXX, F7XX-ECXXX flux electrode combination
AWS A5.18: ER70S-X, E70C-XC, E70C-XM
AWS A5.20: E7XT-X, E7XT-XM
AWS A5.23: F7XX-EXXX-XX, F7XX-ECXXX-XX
AWS A5.28: ER70S-XXX, E70C-XXX
AWS A5.29: E7XTX-X, E7XTX-XM

b) For connected members both having a specified minimum yield strength of 36 ksi (250 MPa) or one having a specified minimum yield strength of 36 ksi (250 MPa), and the other having a specified minimum yield strength greater than 36 ksi (250 MPa).

AWS A5.1: E60XX
AWS A5.17: F6XX-EXXX, F6XX-ECXXX flux electrode combination
AWS A5.20: E6XT-X, E6XT-XM
AWS A5.29: E6XTX-X, E6XT-XM
or any of those listed in Section 1002.3(a).

Other welding methods, providing equivalent strength as demonstrated by tests, may be used.

1002.4 PAINT

The standard shop paint is intended to protect the steel for only a short period of exposure in ordinary atmospheric conditions and shall be considered an impermanent and provisional coating.

When specified, the standard shop paint shall conform to one of the following:

a) Steel Structures Painting Council Specification, SSPC No. 15

b) Or, shall be a shop paint which meets the minimum performance requirements of the above listed specification.

SECTION 1003.
DESIGN AND MANUFACTURE

1003.1 METHOD

Joist Girders shall be designed in accordance with this specification as simply supported primary members. All loads shall be applied through steel joists, and will be equal in magnitude and evenly spaced along the joist girder top chord. Where any applicable design feature is not specifically covered herein, the design shall be in accordance with the following specifications:

a) Where the steel used consists of hot-rolled shapes, bars or plates, use the American Institute of Steel Construction, *Specification for Structural Steel Buildings*.

b) For members that are cold-formed from sheet or strip steel, use the American Iron and Steel Institute, *North American Specification for the Design of Cold-Formed Steel Structural Members*.

Design Basis:

Designs shall be made according to the provisions in this Specification for either Load and Resistance Factor Design (LRFD) or for Allowable Strength Design (ASD).

Load Combinations:

LRFD:

When load combinations are not specified to the joist manufacturer, the required stress shall be computed for the factored loads based on the factors and load combinations as follows:

1.4D

1.2D + 1.6 (L, or L_r, or S, or R)

ASD:

When load combinations are not specified to the joist manufacturer, the required stress shall be computed based on the load combinations as follows:

D

D + (L, or L_r, or S, or R)

Where:

D = dead load due to the weight of the structural elements and the permanent features of the structure

L = live load due to occupancy and movable equipment

L_r = roof live load

S = snow load

R = load due to initial rainwater or ice exclusive of the ponding contribution

When special loads are specified and the specifying professional does not provide the load combinations, the provisions of ASCE 7, *"Minimum Design Loads for Buildings and Other Structures"* shall be used for LRFD and ASD load combinations.

1003.2 DESIGN AND ALLOWABLE STRESSES

Design Using Load and Resistance Factor Design (LRFD)

Joist Girders shall have their components so proportioned that the required stresses, f_u, shall not exceed ϕF_n where,

f_u	=	required stress	ksi (MPa)
F_n	=	nominal stress	ksi (MPa)
ϕ	=	resistance factor	
ϕF_n	=	design stress	

Design Using Allowable Strength Design (ASD)

Joist Girders shall have their components so proportioned that the required stresses, *f*, shall not exceed F_n/Ω where,

f	=	required stress	ksi (MPa)
F_n	=	nominal stress	ksi (MPa)
Ω	=	safety factor	
F_n/Ω	=	allowable stress	

Stresses:

(a) Tension: $\phi_t = 0.90$ (LRFD) $\Omega_t = 1.67$ (ASD)

For Chords: $F_y = 50$ ksi (345 MPa)

For Webs: $F_y = 50$ ksi (345 MPa), or $F_y = 36$ ksi (250 MPa)

Design Stress = $0.9F_y$ (LRFD) (1003.2-1)

Allowable Stress = $0.6F_y$ (ASD) (1003.2-2)

(b) Compression: $\phi_c = 0.90$ (LRFD) $\Omega_c = 1.67$ (ASD)

For members with $\ell/r \le 4.71\sqrt{E/QF_y}$

$$F_{cr} = Q\left[0.658^{\left(\frac{QF_y}{F_e}\right)}\right]F_y \qquad (1003.2\text{-}3)$$

For members with $\ell/r > 4.71\sqrt{E/QF_y}$

$$F_{cr} = 0.877F_e \qquad (1003.2\text{-}4)$$

Where F_e = Elastic bucking stress determined in accordance with Equation 1003.2-5.

$$F_e = \frac{\pi^2 E}{\left(\ell/r\right)^2} \qquad (1003.2\text{-}5)$$

For hot-rolled sections, "Q" is the full reduction factor for slender compression elements.

Design Stress = $0.9F_{cr}$ (LRFD) (1003.2-6)

Allowable Stress = $0.6F_{cr}$ (ASD) (1003.2-7)

In the above equations, ℓ is taken as the distance, in inches (millimeters), between panel points for the chord members and the appropriate length for web members, and *r* is the corresponding least radius of gyration of the member or any component thereof. *E* is equal to 29,000 ksi (200,000 MPa).

Use 1.2 ℓ/r_x for a crimped, first primary compression web member when a moment-resistant weld group is not used for this member; where r_x = member radius of gyration in the plane of the joist.

For cold-formed sections, the method of calculating the nominal column strength is given in the AISI, *North American Specification for the Design of Cold-Formed Steel Structural Members.*

(c) Bending: $\phi_b = 0.90$ (LRFD) $\Omega_b = 1.67$ (ASD)

Bending calculations are to be based on using the elastic section modulus.

For chords and web members other than solid rounds: $F_y = 50$ ksi (345 MPa)

Design Stress = $0.90F_y$ (LRFD) (1003.2-8)

Allowable Stress = $0.60F_y$ (ASD) (1003.2-9)

For web members of solid round cross section: $F_y = 50$ ksi (345 MPa), or $F_y = 36$ ksi (250 MPa)

Design Stress = $1.45F_y$ (LRFD) (1003.2-10)

Allowable Stress = $0.95F_y$ (ASD) (1003.2-11)

For bearing plates: $F_y = 50$ ksi (345 MPa), or $F_y = 36$ ksi (250 MPa)

Design Stress = $1.35F_y$ (LRFD) (1003.2-12)

Allowable Stress = $0.90F_y$ (ASD) (1003.2-13)

(d) Weld Strength:

Shear at throat of fillet welds:

Nominal Shear Stress = $F_{nw} = 0.6F_{exx}$ (1003.2-14)

LRFD: $\phi_w = 0.75$

Design Shear Strength = $\phi R_n = \phi_w F_{nw} A = 0.45F_{exx} A$ (1003.2-15)

ASD: $\Omega_w = 2.0$

Allowable Shear Strength = $R_n/\Omega_w = F_{nw} A/\Omega_w = 0.3F_{exx} A$ (1003.2-16)

A = effective throat area

Made with E70 series electrodes or F7XX-EXXX flux-electrode combinations .... $F_{exx} = 70$ ksi (483 MPa)

Made with E60 series electrodes or F6XX-EXXX flux-electrode combinations .... $F_{exx} = 60$ ksi (414 MPa)

Tension or compression on groove or butt welds shall be the same as those specified for the connected material.

1003.3 MAXIMUM SLENDERNESS RATIOS

The slenderness ratio ℓ/r, where ℓ is the length center-to-center of support points and r is the corresponding least radius of gyration, shall not exceed the following:

Top chord interior panels 90

Top chord end panels 120

Compression members other than top chord 200

Tension members 240

1003.4 MEMBERS

(a) Chords

The bottom chord shall be designed as an axially loaded tension member. The radius of gyration of the bottom chord about its vertical axis shall not be less than $\ell/240$ where ℓ is the distance between lines of bracing.

The top chord shall be designed as an axial loaded compression member. The radius of gyration of the top chord about the vertical axis shall not be less than Span/575.

The top chord shall be considered as stayed laterally by the steel joists provided positive attachment is made.

(b) Web

The vertical shears to be used in the design of the web members shall be determined from full loading, but such vertical shear shall be not less than 25 percent of the end reaction.

Interior vertical web members used in modified Warren type web systems that do not support the direct loads through steel joists shall be designed to resist an axial load of 2 percent of the top chord axial force.

Tension members shall be designed to resist at least 25 percent of their axial force in compression.

(c) Fillers and Ties

In compression members composed of two components, when fillers, ties or welds are used, they shall be spaced so the ℓ/r ratio for each component does not exceed the ℓ/r ratio of the member as a whole. In tension members composed of two components, when fillers, ties or welds are used, they shall be spaced so that the ℓ/r ratio of each component does not exceed 240. The least radius of gyration shall be used in computing the ℓ/r ratio of a component.

(d) Eccentricity

Members connected at a joint shall have their center of gravity lines meet at a point, if practical. Eccentricity on either side of the centroid of chord members may be neglected when it does not exceed the distance between the centroid and the back of the chord. Otherwise, provision shall be made for the stresses due to eccentricity. Ends of Joist Girders shall be proportioned to resist bending produced by eccentricity at the support.

In those cases where a single angle compression member is attached to the outside of the stem of a tee or double angle chord, due consideration shall be given to eccentricity.

(e) Extended Ends

Extended top chords or full depth cantilever ends require the special attention of the specifying professional. The magnitude and location of the loads to be supported, deflection requirements, and proper bracing shall be clearly indicated on the structural drawings.

1003.5 CONNECTIONS

(a) Methods

Joint connections and splices shall be made by attaching the members to one another by arc or resistance welding or other accredited methods.

(1) Welded Connections

a) Selected welds shall be inspected visually by the manufacturer. Prior to this inspection, weld slag shall be removed.

b) Cracks are not acceptable and shall be repaired.

c) Thorough fusion shall exist between layers of weld metal and between weld metal and base metal for the required design length of the weld; such fusion shall be verified by visual inspection.

d) Unfilled weld craters shall not be included in the design length of the weld.

e) Undercut shall not exceed 1/16 inch (2 millimeters) for welds oriented parallel to the principal stress.

f) The sum of surface (piping) porosity diameters shall not exceed 1/16 inch (2 millimeters) in any 1 inch (25 millimeters) of design weld length.

g) Weld spatter that does not interfere with paint coverage is acceptable.

(2) Welding Program

Manufacturers shall have a program for establishing weld procedures and operator qualification, and for weld sampling and testing.

(3) Weld Inspection by Outside Agencies (See Section 1004.10 of this specification).

The agency shall arrange for visual inspection to determine that welds meet the acceptance standards of Section 1003.5(a)(1). Ultrasonic, X-Ray, and magnetic particle testing are inappropriate for Joists Girders due to the configurations of the components and welds.

(b) Strength

(1) Joint Connections – Joint connections shall develop the maximum force due to any of the design loads, but not less than 50 percent of the strength of the member in tension or compression, whichever force is the controlling factor in the selection of the member.

(2) Shop Splices - Shop splices may occur at any point in chord or web members. Splices shall be designed for the member force but not less than 50 percent of the member strength. Members containing a butt weld splice shall develop an ultimate tensile force of at least 57 ksi (393 MPa) times the full design area of the chord or web. The term “member” shall be defined as all component parts comprising the chord or web, at the point of splice.

(c) Field Splices

Field Splices shall be designed by the manufacturer and may be either bolted or welded. Splices shall be designed for the member force, but not less than 50 percent of the member strength.

1003.6 CAMBER

Joist Girders shall have approximate cambers in accordance with the following:

TABLE 1003.6-1

Top Chord Length		Approximate Camber	
20'-0"	(6096 mm)	1/4"	(6 mm)
30'-0"	(9144 mm)	3/8"	(10 mm)
40'-0"	(12192 mm)	5/8"	(16 mm)
50'-0"	(15240 mm)	1"	(25 mm)
60'-0"	(18288 mm)	1 1/2"	(38 mm)
70'-0"	(21336 mm)	2"	(51 mm)
80'-0"	(24384 mm)	2 3/4"	(70 mm)
90'-0"	(27342 mm)	3 1/2"	(89 mm)
100'-0"	(30480 mm)	4 1/4"	(108 mm)
110'-0"	(33528 mm)	5"	(127 mm)
120'-0"	(36576 mm)	6"	(152 mm)

The specifying professional shall give consideration to coordinating Joist Girder camber with adjacent framing.

1003.7 VERIFICATION OF DESIGN AND MANUFACTURE

(a) Design Calculations

Companies manufacturing Joist Girders shall submit design data to the Steel Joist Institute (or an independent agency approved by the Steel Joist Institute) for verification of compliance with the SJI Specifications.

(b) In-Plant Inspections

Each manufacturer shall verify their ability to manufacture Joist Girders through periodic In-Plant Inspections. Inspections shall be performed by an independent agency approved by the Steel Joist Institute. The frequency, manner of inspection, and manner of reporting shall be determined by the Steel Joist Institute. The In-Plant Inspections are not a guarantee of the quality of any specific Joist Girder; this responsibility lies fully and solely with the individual manufacturer.

SECTION 1004.
APPLICATION

1004.1 USAGE

This specification shall apply to any type of structure where steel joists are to be supported directly by Joist Girders installed as hereinafter specified. Where Joist Girders are used other than on simple spans under equal concentrated gravity loading, as prescribed in Section 1003.1, they shall be investigated and modified if necessary to limit the unit stresses to those listed in Section 1003.2. The magnitude and location of all loads and forces, other than equal concentrated gravity loading, shall be provided on the structural drawings. The specifying professional shall design the supporting structure, including the design of columns, connections, and moment plates*. This design shall account for the stresses caused by lateral forces and the stresses due to connecting the bottom chord to the column or other support.

The designed detail of a rigid type connection and moment plates shall be shown on the structural drawings by the specifying professional. The moment plates shall be furnished by other than the joist manufacturer.

* For further reference, refer to Steel Joist Institute Technical Digest #11, “Design of Joist-Girder Frames”

1004.2 SPAN

The span of a Joist Girder shall not exceed 24 times its depth.

1004.3 DEPTH

Joist Girders may have either parallel top chords or a top chord slope of 1/8 inch per foot (1:96). The nominal depth of sloping chord Joist Girders shall be the depth at mid-span.

1004.4 END SUPPORTS

(a) Masonry and Concrete

Joist Girders supported by masonry or concrete are to bear on steel bearing plates and shall be designed as steel bearing. Due consideration of the end reactions and all other vertical and lateral forces shall be taken by the specifying professional in the design of the steel bearing plate and the masonry or concrete. The ends of Joist Girders shall extend a distance of not less than 6 inches (152 millimeters) over the masonry or concrete support and be anchored to the steel bearing plate. The plate shall be located not more than 1/2 inch (13 millimeters) from the face of the wall and shall be not less than 9 inches (229 millimeters) wide perpendicular to the length of the girder. The plate is to be designed by the specifying professional and shall be furnished by other than the joist manufacturer.

Where it is deemed necessary to bear less than 6 inches (152 millimeters) over the masonry or concrete support, special consideration is to be given to the design of the steel bearing plate and the masonry or concrete by the specifying professional. The girders must bear a minimum of 4 inches (102 millimeters) on the steel bearing plate.

(b) Steel

Due consideration of the end reactions and all other vertical and lateral forces shall be taken by the specifying professional in the design of the steel support. The ends of Joist Girders shall extend a distance of not less than 4 inches (102 millimeters) over the steel supports and shall have positive attachment to the support, either by bolting or welding.

1004.5 BRACING

Joist Girders shall be proportioned such that they can be erected without bridging (See Section 1004.9 for bracing required for uplift forces). Therefore, the following requirements must be met:

a) The ends of the bottom chord are restrained from lateral movement to brace the girder from overturning. For Joist Girders at columns in steel frames, restraint shall be provided by a stabilizer plate on the column.

b) No other loads shall be placed on the Joist Girder until the steel joists bearing on the girder are in place and welded to the girder.

1004.6 END ANCHORAGE

(a) Masonry and Concrete

Ends of Joist Girders resting on steel bearing plates on masonry or structural concrete shall be attached thereto with a minimum of two 1/4 inch (6 millimeters) fillet welds 2 inches (51 millimeters) long, or with two 3/4 inch (19 millimeters) bolts, or the equivalent.

(b) Steel

Ends of Joist Girders resting on steel supports shall be attached thereto with a minimum of two 1/4 inch (6 millimeters) fillet welds 2 inches (51 millimeters) long, or with two 3/4 inch (19 millimeters) bolts, or the equivalent. In steel frames, bearing seats for Joist Girders shall be fabricated to allow for field bolting.

(c) Uplift

Where uplift forces are a design consideration, roof Joist Girders shall be anchored to resist such forces (Refer to Section 1004.9).

1004.7 DEFLECTION

The deflections due to the design live load shall not exceed the following:

Floors: 1/360 of span.

Roofs: 1/360 of span where a plaster ceiling is attached or suspended.

1/240 of span for all other cases.

The specifying professional shall give consideration to the effects of deflection and vibration* in the selection of Joist Girders.

* For further reference, refer to Steel Joist Institute Technical Digest #5, "Vibration of Steel Joist-Concrete Slab Floors" and the Institute's Computer Vibration Program.

1004.8 PONDING*

The ponding investigation shall be performed by the specifying professional.

* For further reference, refer to Steel Joist Institute Technical Digest #3, "Structural Design of Steel Joist Roofs to Resist Ponding Loads" and AISC Specifications.

1004.9 UPLIFT

Where uplift forces due to wind are a design requirement, these forces must be indicated on the contract drawings in terms of NET uplift in pounds per square foot (Pascals). The contract drawings must indicate if the net uplift is based on ASD or LRFD. When these forces are specified, they must be considered in the design of Joist Girders and/or bracing. If the ends of the bottom chord are not strutted, <u>bracing must be provided near the first bottom chord panel points</u> whenever uplift due to wind forces is a design consideration.*

* For further reference, refer to Steel Joist Institute Technical Digest #6, "Structural Design of Steel Joist Roofs to Resist Uplift Loads".

1004.10 INSPECTION

Joist Girders shall be inspected by the manufacturer before shipment to verify compliance of materials and workmanship with the requirements of this specification. If the purchaser wishes an inspection of the Joist Girders by someone other than the manufacturer's own inspectors, they may reserve the right to do so in their "Invitation to Bid" or the accompanying "Job Specifications". Arrangements shall be made with the manufacturer for such inspection of the Joist Girders at the manufacturing shop by the purchaser's inspectors at purchaser's expense.

SECTION 1005.* HANDLING AND ERECTION

Particular attention should be paid to the erection of Joist Girders.

Care shall be exercised at all times to avoid damage through careless handling during unloading, storing and erecting. Dropping of Joist Girders shall not be permitted.

In steel framing, where Joist Girders are utilized at column lines, the Joist Girder shall be field-bolted at the column. Before hoisting cables are released and before an employee is allowed on the Joist Girder the following conditions must be met:

a) The seat at each end of the Joist Girder is attached in accordance with Section 1004.6.

When a bolted seat connection is used for erection purposes, as a minimum, the bolts must be snug tightened. The snug tight condition is defined as the tightness that exists when all plies of a joint are in firm contact. This may be attained by a few impacts of an impact wrench or the full effort of an employee using an ordinary spud wrench.

b) Where stabilizer plates are required the Joist Girder bottom chord must engage the stabilizer plate.

During the construction period, the contractor shall provide means for the adequate distribution of loads so that the carrying capacity of any Joist Girder is not exceeded.

Joist Girders shall not be used as anchorage points for a fall arrest system unless written direction to do so is obtained from a "qualified person".(1)

Field welding shall not damage the Joist Girder. The total length of weld at any one cross-section on cold-formed members whose yield strength has been attained by cold working and whose as-formed strength is used in the design, shall not exceed 50 percent of the overall developed width of the cold-formed section.

* For a thorough coverage of this topic, refer to SJI Technical Digest #9, "Handling and Erection of Steel Joists and Joist Girders".

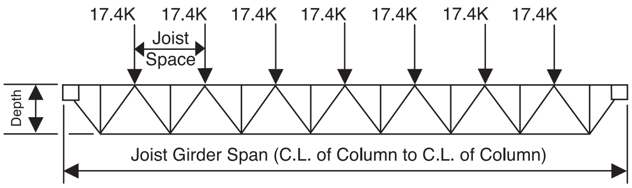
(1) See Appendix E for OSHA definition of "qualified person".

SECTION 1006. HOW TO SPECIFY JOIST GIRDERS

For a given Joist Girder span, the specifying professional first determines the number of joist spaces. Then the panel point loads are calculated and a depth is selected. The following tables give the Joist Girder weight in pounds per linear foot (kiloNewtons per meter) for various depths and loads.

1. The purpose of the Joist Girder Design Guide Weight Table is to assist the specifying professional in the selection of a roof or floor support system.
2. It is not necessary to use only the depths, spans, or loads shown in the tables.
3. Holes in chord elements present special problems which must be considered by both the specifying professional and the Joist Girder Manufacturer. The sizes and locations of such holes shall be clearly indicated on the structural drawings.

Example using *Load and Resistance Factor Design (LRFD)* and U. S. Customary units:



STANDARD DESIGNATION

44G	8N	17.4F
Depth in Inches	Number of Joist Spaces	Factored Load in Kips at Each Panel Point

Given 42'-0" x 50'-0" bay. Joists spaced on 5'-3" centers

Live Load = 30 psf x 1.6

Dead Load = 15 psf x 1.2
(includes the approximate Joist Girder weight)

Total Load = 66 psf (factored)

Note: Web configuration may vary from that shown. Contact Joist Girder manufacturer if exact layout must be known.

1. Determine number of actual joist spaces (N).
   In this example, N = 8
2. Compute total factored load:

   Total load = 5.25 x 66 psf = 346.5 plf
3. Joist Girder Section: (Interior)

   a) Compute the factored concentrated load at top chord panel points

   P = 346.5 x 50 = 17,325 lbs = 17.4 kips
   (use 18K for depth selection).

   b) Select Joist Girder depth:

   Refer to the LRFD Joist Girder Design Guide Weight Table for the 42'-0" span, 8 panel, 18.0K Joist Girder. The rule of about one inch of depth for each foot of span is a good compromise of limited depth and economy. Therefore, select a depth of 44 inches.

   c) The Joist Girder will then be designated 44G8N17.4F. Note that the letter "F" is included at the end of the designation to clearly indicate that this is a factored load.

   d) The LRFD Joist Girder Design Guide Weight Table shows the weight for a 44G8N17.4K as 49 pounds per linear foot. The designer should verify that the weight is not greater than the weight assumed in the Dead Load above.

   e) Check live load deflection:

   Live load = 30 psf x 50 ft = 1500 plf

   Approximate Joist Girder moment of inertia

   = 0.018 NPLd

   = 0.018 x 8 x 17.4 x 42 x 44 = 4630 in.^{4}

Allowable deflection for plastered ceilings

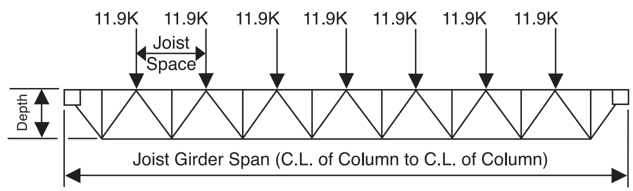
$$= L/360 = \frac{42(12)}{360} = 1.40 \text{ in.}$$

$$\text{Deflection} = 1.15\left[\frac{5wL^4}{384EI}\right] = \frac{1.15(5)(1.500/12)(42x12)^4}{384(29000)(4630)}$$

= 0.90 in. < 1.40 in., Okay

Live load deflection rarely governs because of the relatively small span-depth ratios of Joist Girders.

Example using *Allowable Strength Design (ASD)* and U. S. Customary units:



STANDARD DESIGNATION

44G	8N	11.9K
Depth in Inches	Number of Joist Spaces	Load in Kips at Each Panel Point

Given 42'-0" x 50'-0" bay. Joists spaced on 5'-3" centers.

Live Load = 30 psf

Dead Load = 15 psf
(includes the approximate Joist Girder weight)

Total Load = 45 psf

Note: Web configuration may vary from that shown. Contact Joist Girder manufacturer if exact layout must be known.

1. Determine number of actual joist spaces (N).

   In this example, N = 8
2. Compute total load:

   Total load = 5.25 x 45 psf = 236.25 plf
3. Joist Girder Section: (Interior)

   a) Compute the concentrated load at top chord panel points

   P = 236.25 x 50 = 11,813 lbs = 11.9 kips
   (use 12K for depth selection).

   b) Select Joist Girder depth:

   Refer to the ASD Joist Girder Design Guide Weight Table for the 42'-0" span, 8 panel, 12.0K Joist Girder.

The rule of about one inch of depth for each foot of span is a good compromise of limited depth and economy. Therefore, select a depth of 44 inches.

c) The Joist Girder will then be designated 44G8N11.9K.

d) The ASD Joist Girder Design Guide Weight Table shows the weight for a 44G8N12K as 49 pounds per linear foot. The designer should verify that the weight is not greater than the weight assumed in the Dead Load above.

e) Check live load deflection:

Live load = 30 psf x 50 ft = 1500 plf.

Approximate Joist Girder moment of inertia

$$= 0.027\ NPLd$$

$$= 0.027 \times 8 \times 11.9 \times 42 \times 44 = 4750\ \text{in.}^4$$

Allowable deflection for plastered ceilings

$$= L/360 = \frac{42(12)}{360} = 1.40\ \text{in.}$$

$$\text{Deflection} = 1.15 \left[\frac{5wL^4}{384EI}\right] = \frac{1.15(5)(1.500/12)(42\text{x}12)^4}{384(29000)(4750)}$$

$$= 0.88\ \text{in.} < 1.40\ \text{in., Okay}$$

Live load deflection rarely governs because of the relatively small span-depth ratios of Joist Girders.