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STRUCTURAL AND EARTHQUAKE ENGINEERING SOFTWARE

ETABS® 2016
Integrated Building Design Software

Steel Connection Design Manual

AISC 360-10





Steel Connection Design Manual

AISC 360-10

For ETABS® 2016

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Chapter 1

Introduction

The design/check of steel connections is seamlessly integrated within the program. Initiation of the design process, along with control of various design parameters, is accomplished using the Design menu. Model and analysis data, such as material properties and member forces, are recovered directly from the model database, and are used in the design process in accordance with the user defined or default design settings. As with all design applications, the user should carefully review all of the user options and default settings to ensure that the design process is consistent with the user's expectations.

1.1 Load Combinations

The design is based on a set of user-specified loading combinations. However, the program provides default load combinations based on the design code. If the default load combinations are acceptable, no definition of additional load combinations is required.

1.2 Stress Check

Steel connection design consists of calculating the connection forces or stresses, and then comparing those calculated values with acceptable limits.

That comparison produces a demand/capacity ratio, which typically should not exceed a value of unity if code requirements are to be satisfied.

Program output can be presented graphically on the model or in calculation sheets prepared for each connection. For each presentation method, the output is in a format that allows the engineer to quickly study the stress conditions that exist in the connection, and in the event the connection is not adequate, aid the engineer in taking appropriate remedial measures.

This manual is dedicated to the use of the AISC 360-10 design code. This option covers the “ANSI/AISC 360-10 Specification for Structural Steel Buildings” (AISC 2010a, b).

The implementation covers loading and load combinations from “ASCE/SEI 7-10 Minimum Design Loads for Buildings and Other Structures” (ASCE 2010), and also special requirements from “IBC 2012 International Building Code” (IBC 2012). Only LRFD (Load and Resistance Factor Design) is currently included in this implementation. The strengths are calculated in the nominal levels. The phi (LRFD) factor is applied during calculation of demand/capacity ratios only. The design code is written in kip-inch units. All the associated equations and requirements have been implemented in the program in kip-in units. The program has been enabled with unit conversion capability. This allows the users to enjoy the flexibility of choosing any set of consistent units during creating and editing models, exporting and importing the model components, and reviewing the design results.

Chapter 2

Design Algorithms

This chapter provides an overview of the basic assumptions, design preconditions, and some of the design parameters that affect the design of steel connections.

2.1 Demand/Capacity Ratios

Determination of the controlling demand/capacity (D/C) ratios for each steel connection indicates the acceptability of the connection for the given loading conditions. The steps for calculating the D/C ratios are as follows:

- The factored forces are determined for each connection location.
- The nominal strengths and controlling material thicknesses are calculated for various criteria based on the equations provided later in this manual.
- Factored forces and material thicknesses are compared to nominal strengths and limiting thicknesses to determine D/C ratios. In either case, design codes typically require that the ratios not exceed a value of unity. A capacity ratio greater than unity indicates a connection that has exceeded a limit state.

2.2 Design Load Combinations

The design load combinations are the various combinations of the prescribed load cases for which the structure needs to be checked. The program creates a number of default design load combinations for steel frame design. Users can add their own design combinations as well as modify or delete the program default design load combinations. An unlimited number of design load combinations can be specified.

To define a design load combination, simply specify one or more load cases, each with its own scale factor. The scale factors are applied to the forces and moments from the load cases to form the factored design forces and moments for each design load combination.

For normal loading conditions involving static dead load (DL), live load (LL), roof live load (RL), snow load (SL), wind load (WL), earthquake load (EL), notional load (NL), and dynamic response spectrum load (EL), the program has built-in default design combinations for the design code. These are based on the code recommendations.

The default design combinations assume all load cases declared as dead or live to be additive. However, each load case declared as wind, earthquake, or response spectrum cases, is assumed to be non-additive with other loads and produces multiple lateral combinations. Also static wind, earthquake and notional load responses produce separate design combinations with the sense (positive or negative) reversed. The notional load patterns are added to load combinations involving gravity loads only. The user is free to modify the default design preferences to include the notional loads for combinations involving lateral loads.

For other loading conditions involving moving load, time history, pattern live load, separate consideration of roof live load, snow load, and the like, the user must define the design load combinations in lieu of or in addition to the default design load combinations. If notional loads are to be combined with other load combinations involving wind or earthquake loads, the design load combinations need to be defined in lieu of or in addition to the default design load combinations.

For multi-valued design combinations, such as those involving response spectrum, time history, moving loads and envelopes, where any correspondence be-

tween forces is lost, the program automatically produces sub-combinations using the maxima/minima values of the interacting forces. Separate combinations with negative factors for response spectrum load cases are not required because the program automatically takes the minima to be the negative of the maxima response when preparing the sub-combinations described previously.

The program allows live load reduction factors to be applied to the member forces of the reducible live load case on a member-by-member basis to reduce the contribution of the live load to the factored responses.

2.3 Supported Connection Types

The program currently handles steel base plates, as well as the following connection types.

Shear Connections

Beam to Column Flange (major axis)

Beam to Column Web (minor axis)

Beam to Beam (beam coped top)

Beam to Beam (beam coped top and bottom)

Moment Connections

Beam to Column Flange (major axis)

Beam to Column Web (minor axis)

2.4 Design Checks

The design checks performed for each of the connection types are listed below and described in detail in the next chapter.

Beam to Column Flange Moment (Major Axis)

Beam design flexural strength

Strength of bolt group

Beam to Column Flange Moment (Major Axis)

Shear yielding of web plate
Shear rupture of web plate
Block shear rupture strength of web plate
Design strength of weld
Web plate rupture strength at weld
Shear yielding of beam web
Shear rupture strength of beam web
Panel zone shear strength
Local flange bending
Local web yielding
Web crippling

Beam to Column Web Moment (Minor Axis)

Beam design flexural strength
Strength of bolt group
Shear yielding of web plate
Shear rupture of web plate
Block shear rupture strength of web plate
Design strength of weld
Weld strength at tension flange
Shear yielding of beam web

Beam to Column Flange Shear (Major Axis)

Bolt strength in single shear
Bolt bearing on web plate
Shear yielding of web plate
Shear rupture of web plate

Beam to Column Flange Shear (Major Axis)

Block shear rupture strength of web plate

Design strength of weld

Bolt bearing on beam web

Beam to Column Web Shear (Minor Axis)

Strength of bolt group

Maximum plate thickness for plate yielding before bolt shear/bearing

Shear yielding of web plate

Critical flexural stress

Shear rupture of web plate

Block shear rupture strength of web plate

Flexure rupture of plate

Local buckling of plate

Strength of column web at weld

Beam to Beam Coped at Top

Bolt strength in single shear

Bolt bearing on web plate

Shear yielding of web plate

Shear rupture of web plate

Block shear rupture strength of web plate

Design strength of weld

Bolt bearing on beam web

Block shear rupture strength of beam web

Flexural yielding of coped section

Local web buckling on coped section

Beam to Beam Coped at Top

Shear rupture of girder web

Beam to Beam Coped Top and Bottom

Bolt strength in single shear

Bolt bearing on web plate

Shear yielding of web plate

Shear rupture of web plate

Block shear rupture of strength of web plate

Design strength of weld

Bolt bearing on beam web

Shear yielding of beam web

Shear rupture of beam web

Block shear rupture strength of beam web

Flexural yielding of coped section

Local web buckling on coped section

Shear rupture of girder web

2.5 Choice of Units

English as well as SI and MKS metric units can be used for input. The codes are based on a specific system of units. All equations and descriptions presented in the subsequent chapters correspond to that specific system of units unless otherwise noted. However, any system of units can be used to define and design a structure in the program.

The Display Unit preferences allow the user to specify the units.

Chapter 3

Design Using ANSI/AISC 360-10

This chapter provides a detailed description of the algorithms used by the program in the design/check of steel connections in accordance with “ANSI/AISC 360-10 — Specifications for Structural Steel Building” (AISC 2010a, b). The implementation covers load combinations from “ASCE/SEI 7-10,” which is described in the section “Design Loading Combinations” in this chapter. The loading based on “ASCE/SEI 7-10” has been described in a separate document entitled “CSI Lateral Load Manual” (CSI 2013). References also are made to IBC 2012 in this document.

For referring to pertinent sections of the corresponding code, a unique prefix is assigned for each code.

- Reference to the ANSI/AISC 360-10 code is identified with the prefix “**AISC.**”
- Reference to the ASCE/SEI 7-10 code is identified with the prefix “**ASCE.**”
- Reference to the IBC 2012 code is identified with the prefix “**IBC.**”

3.1 Notations

The various notations used in this chapter are described herein.

A	Cross-sectional area, in ²
A_b	Nominal bolt area, in ²
A_g	Gross area, in ²
A_{gv}	Gross area subject to shear, in ²
A_{nt}	Net area subject to tension, in ²
A_{nv}	Net area subject to shear, in ²
A_p	Plate cross-sectional area, in ²
D	Number of sixteenths-of-an-inch for a weld size, in
F_{EXX}	Classification strength of weld metal, ksi
F_{cr}	Critical stress, ksi
F_t	Nominal tensile strength of a bolt, ksi
F_u	Specified minimum tensile strength of steel, ksi
F_v	Nominal shear strength of a bolt, ksi
I_{xx}	Moment of inertia of a section about the x-axis, in ⁴
L	Height of a plate, in
L_{eh}	Horizontal edge distance, in
L_{ev}	Vertical edge distance, in
M_n	Nominal flexural strength, kip-in
Q	Full reduction factor for slender compression elements
R_n	Nominal resistance or strength, kips
S_{net}	Net elastic section modulus, in ³
S_x	Elastic section modulus about the x-axis, in ³

Z	Plastic section modulus, in ³
a	Distance from a bolt centerline to edge of the fitting, in
c	Cope length, in
d	Nominal fastener diameter, in
d	Overall member depth, in
d_c	Cope depth, in
d_{ct}	Top flange cope depth, in
d_{cb}	Bottom flange cope depth, in
d_b	Hole diameter, in
f_d	Adjustment factor for beams coped at both flanges
h_o	Remaining web depth of coped beam, in
k	Plate buckling coefficient for beams coped at top only
n	Number of bolts in a vertical row
s	Bolt spacing, in
t	Thickness, in
t_w	Web thickness, in
λ	Slenderness parameter
ϕ	Resistance factor
ϕF_{bc}	Design buckling stress for coped beams, ksi
ϕR_n	Design strength, kips

3.2 Design Loading Combinations

The structure is to be designed so that its design strength equals or exceeds the effects of factored loads stipulated by the applicable design code. The default design combinations are the various combinations of the already defined load cases, such as dead load (DL), live load (LL), roof live load (RL), snow load (SL), wind load (WL), and horizontal earthquake load (EL).

AISC 360-10 refers to the applicable building code for the loads and load combinations to be considered in the design, and to ASCE 7-10 in the absence of such a building code. Hence, the default design combinations used in the current version are the ones stipulated in ASCE 7-10:

For design in accordance with LRFD provisions:

1.4 DL	(ASCE 2.3.2-1)
1.2 DL + 1.6 LL + 0.5RL	(ASCE 2.3.2-2)
1.2 DL + 1.0 LL + 1.6RL	(ASCE 2.3.2-3)
1.2 DL + 1.6 LL + 0.5 SL	(ASCE 2.3.2-2)
1.2 DL + 1.0 LL + 1.6 SL	(ASCE 2.3.2-3)
0.9 DL ± 1.0WL	(ASCE 2.3.2-6)
1.2 DL + 1.6 RL ± 0.5WL	(ASCE 2.3.2-3)
1.2 DL + 1.0LL + 0.5RL ± 1.0WL	(ASCE 2.3.2-4)
1.2 DL + 1.6 SL ± 0.5 WL	(ASCE 2.3.2-3)
1.2 DL + 1.0LL + 0.5SL ± 1.0 WL	(ASCE 2.3.2-4)
0.9 DL ± 1.0 EL	(ASCE 2.3.2-7)
1.2 DL + 1.0 LL + 0.2SL ± 1.0EL	(ASCE 2.3.2-5)

The combinations described herein are the default loading combinations only. They can be deleted or edited as required by the design code or engineer-of-record.

The program allows live load reduction factors to be applied to the member forces of the reducible live load case on a member-by-member basis to reduce the contribution of the live load to the factored responses.

3.3 Design Check Calculations

The following sections provide details of the calculations performed for each of the design checks performed for the various connection types.

3.3.1 Strength of Bolt Group

The strength of the bolt group is determined as the minimum of the bolt strength in single shear and bolt bearing strength.

3.3.1.1 Bolt Strength in Single Shear

$$\phi R_n = \phi n F_{nv} A_b \quad (\text{AISC J3-1})$$

$$\phi = 0.75$$

The nominal shear strength of the bolt, F_{nv} , is taken from Table J3.2. A_b is the bolt area and n is the total number of bolts.

3.3.1.2 Bolt Bearing Strength

The bolt bearing on the web plate is calculated as follows.

$$\phi R_n = \phi [\min(r_{n1}, r_{n(\max)}) + (n - 1) \min(r_{n2}, r_{n(\max)})]$$

where,

$$\phi = 0.75$$

$$r_{n1} = 1.2 l_{c1} t F_u$$

$$r_{n2} = 1.2 l_c t F_u$$

$$r_{n(\max)} = 2.4 d t F_u$$

$$l_{c1} = L_{ev} - \frac{d_h}{2}$$

$$l_c = s - d_h$$

3.3.2 Shear Yield Strength

The shear yielding of the material is calculated as follows.

$$\phi R_n = \phi 0.6 F_y A_{gv} \quad (\text{AISC J4-3})$$

$$\phi = 1$$

$$A_{gv} = Lt$$

3.3.3 Shear Rupture Strength

The shear rupture strength is calculated as follows.

$$\phi R_n = \phi 0.6 F_y A_{nv} \quad (\text{AISC J4-4})$$

$$\phi = 0.75$$

$$A_{nv} = \left[L - n \left(d_h + \frac{1}{16} \right) \right] t$$

3.3.4 Block Shear Rupture Strength

The block shear rupture strength is calculated as follows.

$$\phi R_n = \phi [F_u A_{nt} + \min(0.6 F_y A_{gv}, 0.6 F_u A_{nv})] \quad (\text{AISC J4-5})$$

$$\phi = 0.75$$

$$A_{nt} = \left[L_{eh} - \frac{1}{2} \left(d_h + \frac{1}{16} \right) \right] t$$

$$A_{nv} = \left[\{(n-1)s + L_{ev}\} - \left\{ \frac{2n-1}{2} \left(d_h + \frac{1}{16} \right) \right\} \right] t$$

$$A_{gv} = \{(n-1)s + L_{ev}\} t$$

3.3.5 Design Weld Strength

The design strength of a double-sided fillet weld on a shear plate is calculated as follows.

$$\phi R_n = \frac{\phi 0.6 F_{EXX} D 2L}{22.627} \quad (\text{AISC Manual pg. 8-8})$$

$$\phi = 0.75$$

3.3.6 Web Plate Rupture Strength

The web plate rupture strength is calculated by determining the minimum base metal thickness that will match the available shear rupture strength of the base metal, to the available shear rupture strength of the weld(s). The minimum base metal thickness is calculated as follows.

$$t_{min} = \frac{F_{EXX}D}{22.62F_u} \quad (\text{AISC Manual pg. 9-5})$$

3.3.7 Critical Flexural Stress

The critical flexural stress is calculated as:

$$\phi M_n = \phi F_{cr} Z_x$$

where,

$$\phi F_{cr} = \sqrt{(\phi F_y)^2 - 3f_v^2}$$

$$\phi = 0.9$$

$$f_v = \frac{V_u}{A_p}$$

$$A_p = tL$$

$$Z_x = \frac{tL^2}{4}$$

3.3.8 Flexural Rupture Strength

The flexural rupture strength of a section is calculated as follows.

$$\phi M_n = \phi F_u Z_{net} \quad (\text{AISC Manual pg. 9-6})$$

$$\phi = 0.75$$

$$Z_{net} = \frac{tL^2}{4} \left[1 - \frac{d_h + \frac{1}{16}}{3} \right]$$

3.3.9 Local Buckling of Plate

The local buckling of a plate is checked by calculating the capacity of the section as follows.

$$\phi M_n = \phi F_{cr} S_{net} \quad (\text{AISC Manual pg. 9-7})$$

where,

$$\phi = 0.9$$

$$S_{net} = \frac{tL^2}{6}$$

$$F_{cr} = F_y Q$$

The value of Q depends on the value of λ , where

$$Q = 1 \text{ for } \lambda \leq 0.7$$

$$Q = 1.34 - 0.486\lambda$$

$$\lambda = \frac{(L\sqrt{F_y})}{10t\sqrt{475+280\left(\frac{L}{a}\right)^2}}$$

3.3.10 Flexural Yielding of Coped Section

Flexure of the coped section is checked by calculating the flexural capacity as follows.

$$\phi M_n = \phi F_y S_{net} \quad (\text{AISC Manual Part 9})$$

$$\phi = 0.9$$

where, S_{net} is the net elastic section modulus of the section. For beams coped at the top and bottom,

$$S_{net} = \frac{t_w h_o^2}{6}$$

For beams coped at the top only,

$$S_{net} = \frac{I_{xx}}{\gamma_x}$$

where,

$$y_x = h_o - \left(\frac{h_o^2 t_w + t_f^2 (b_f - t_w)}{2A} \right)$$

$$I_{xx} = \left(\frac{1}{3} \right) \left[t_w y_x^3 + b_f (h_o - y_x)^3 - (b_f - t_w) (h_o - y_x - t_f)^3 \right]$$

3.3.11 Local Web Buckling of Coped Section

The local web buckling strength of coped sections is calculated as follows.

$$\phi R_n = \frac{\phi F_{bc} S_{net}}{e} \quad (\text{AISC Manual Part 9})$$

where S_{net} is calculated as in 3.3.10 and ϕF_{bc} is calculated as follows. For a beam section coped top and bottom:

$$\phi F_{bc} = \frac{50840 t_w^2 f_d}{c h_o}$$

$$f_d = 3.5 - \frac{7.5 d_c}{d}$$

For a beam section coped at the top only:

$$\phi F_{bc} = \phi 26,210 \left(\frac{t_w}{h_o} \right)^2 f k$$

$$\phi = 0.9$$

where f and k are determined as follows.

$$f = 2 \left(\frac{c}{d} \right) \text{ for } \frac{c}{d} \leq 1.0$$

$$f = 1 + \left(\frac{c}{d} \right) \text{ for } \frac{c}{d} > 1.0$$

$$k = 2.2 \left(\frac{h_o}{c} \right) \text{ for } \frac{c}{h_o} \leq 1.0$$

$$k = 2.2 \left(\frac{h_o}{c} \right) \text{ for } \frac{c}{h_o} > 1.0$$

3.3.12 Beam Design Flexural Strength

The beam design flexural capacity is calculated as follows.

$$\phi M_n = \phi F_u S_x \quad (\text{AISC F13-1})$$

3.3.13 Slip Resistance

The serviceability limit state of slip is checked using the slip resistance calculated as follows.

$$\phi R_n = \phi \mu D_u h_f T_b n_s \quad (\text{AISC J3-8b})$$

where,

$$\phi = 1 \text{ for Standard hole}$$

$$0.85 \text{ for Oversize hole}$$

$$D_u = 1.13$$

$$h_f = 1$$

The value of μ is taken from Appendix A, Table 1 and the value of T_b is taken from Appendix A, Table 2.

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