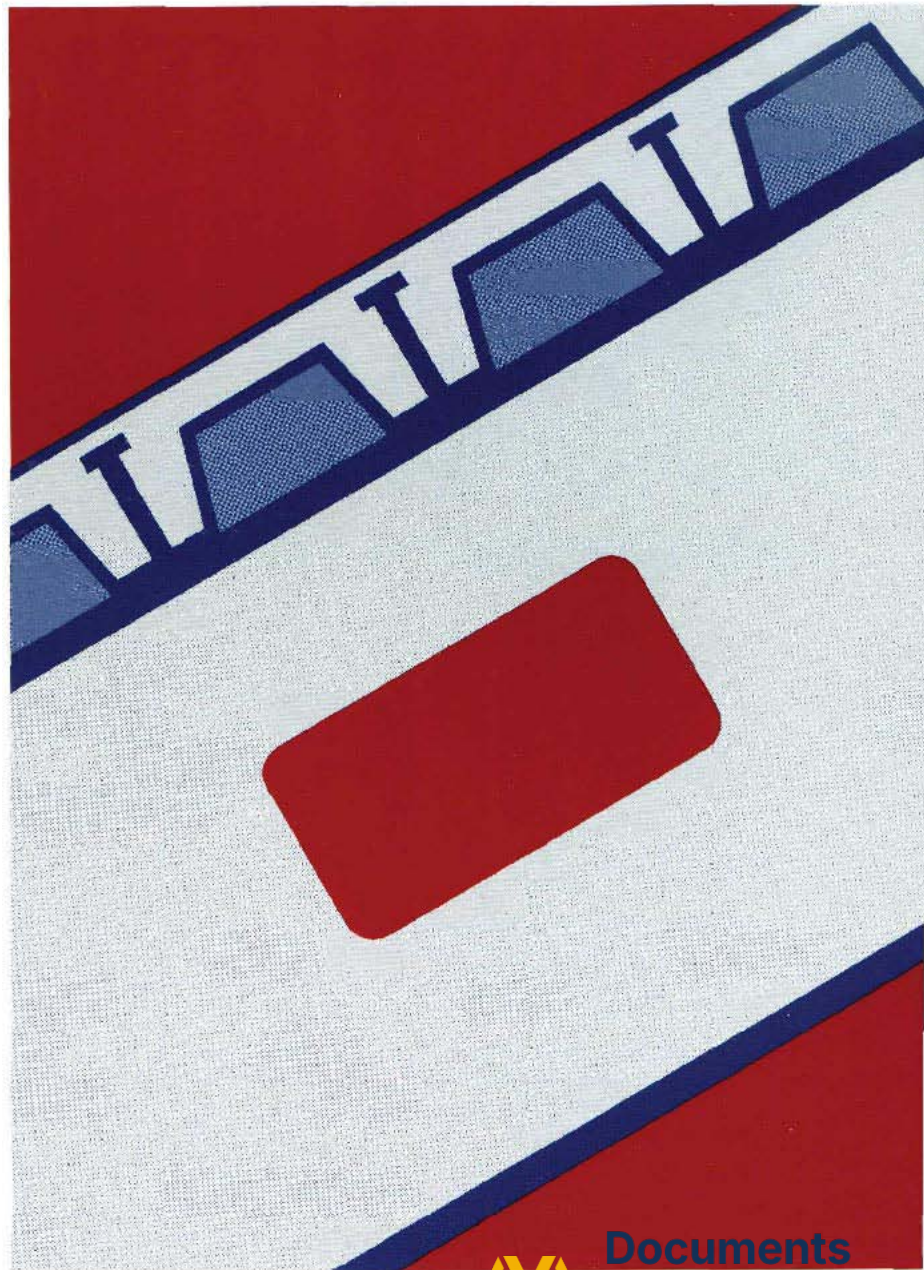




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Steel and Composite Beams with Web Openings



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Steel and Composite Beams with Web Openings

Design of Steel and Composite Beams with Web Openings

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A M E R I C A N I N S T I T U T E O F S T E E L C O N S T R U C T I O N

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PREFACE

This booklet was prepared under the direction of the Committee on Research of the American Institute of Steel Construction, Inc. as part of a series of publications on special topics related to fabricated structural steel. Its purpose is to serve as a supplemental reference to the AISC Manual of Steel Construction to assist practicing engineers engaged in building design.

The design guidelines suggested by the author that are outside the scope of the AISC Specifications or Code do not represent an official position of the Institute and are not intended to exclude other design methods and procedures. It is recognized that the design of structures is within the scope of expertise of a competent licensed structural engineer, architect or other licensed professional for the application of principles to a particular structure.

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Chapter 1

INTRODUCTION

Height limitations are often imposed on multistory buildings based on zoning regulations, economic requirements and esthetic considerations, including the need to match the floor heights of existing buildings. The ability to meet these restrictions is an important consideration in the selection of a framing system and is especially important when the framing system is structural steel. Web openings can be used to pass utilities through beams and, thus, help minimize story height. A decrease in building height reduces both the exterior surface and the interior volume of a building, which lowers operational and maintenance costs, as well as construction costs. On the negative side, web openings can significantly reduce the shear and bending capacity of steel or composite beams.

Web openings have been used for many years in structural steel beams, predating the development of straightforward design procedures, because of necessity and/or economic advantage. Openings were often reinforced, and composite beams were often treated as noncomposite members at web openings. Reinforcement schemes included the use of both horizontal and vertical bars, or bars completely around the periphery of the opening. As design procedures were developed, unreinforced and reinforced openings were often approached as distinct problems, as were composite and non-composite members.

In recent years, a great deal of progress has been made in the design of both steel and composite beams with web

openings. Much of the work is summarized in state-of-the-art reports (Darwin 1985, 1988 & Redwood 1983). Among the benefits of this progress has been the realization that the behavior of steel and composite beams is quite similar at web openings. It has also become clear that a single design approach can be used for both unreinforced and reinforced openings. If reinforcement is needed, horizontal bars above and below the opening are fully effective. Vertical bars or bars around the opening periphery are neither needed nor cost effective.

This guide presents a unified approach to the design of structural steel members with web openings. The approach is based on strength criteria rather than allowable stresses, because at working loads, locally high stresses around web openings have little connection with a member's deflection or strength.

The procedures presented in the following chapters are formulated to provide safe, economical designs in terms of both the completed structure and the designer's time. The design expressions are applicable to members with individual openings or multiple openings spaced far enough apart so that the openings do not interact. Castellated beams are not included. For practical reasons, opening depth is limited to 70 percent of member depth. Steel yield strength is limited to 65 ksi and sections must meet the AISC requirements for compact sections (AISC 1986).

Chapter 2

DEFINITIONS AND NOTATION

2.1 DEFINITIONS

The following terms apply to members with web openings.

bottom tee—region of a beam below an opening.

bridging—separation of the concrete slab from the steel section in composite beams. The separation occurs over an opening between the low moment end of the opening and a point outside the opening past the high moment end of the opening.

high moment end—the edge of an opening subjected to the greater primary bending moment. The secondary and primary bending moments act in the same direction.

low moment end—the edge of an opening subjected to the lower primary bending moment. The secondary and primary bending moments act in opposite directions.

opening parameter—quantity used to limit opening size and aspect ratio.

plastic neutral axis—position in steel section, or top or bottom tees, at which the stress changes abruptly from tension to compression.

primary bending moment—bending moment at any point in a beam caused by external loading.

reinforcement—longitudinal steel bars welded above and below an opening to increase section capacity.

reinforcement, slab—reinforcing steel within a concrete slab.

secondary bending moment—bending moment within a tee that is induced by the shear carried by the tee.

tee—region of a beam above or below an opening.

top tee—region of a beam above an opening.

unperforated member—section without an opening. Refers to properties of the member at the position of the opening.

2.2 NOTATION

A	Gross transformed area of a tee
A_f	Area of flange
A_r	Cross-sectional area of reinforcement along top or bottom edge of opening
A_s	Cross-sectional area of steel in unperforated member
A_{sc}	Cross-sectional area of shear stud
A_{sn}	Net area of steel section with opening and reinforcement
A_{st}	Net steel area of top tee
A^T	Area of a steel tee
A_{vc}	Effective concrete shear area = $3t_s t_e$
A_y	Effective shear area of a steel tee
D_o	Diameter of circular opening

E	Modulus of elasticity of steel
E_c	Modulus of elasticity of concrete
F_{x_1}, F_{x_2}	Horizontal forces at ends of a beam element
F_y	Yield strength of steel
$\bar{F}_y$	Reduced axial yield strength of steel; see Eqs. 5-19 and 5-20
F_{y_1}, F_{y_2}	Vertical forces at ends of a beam element
F_{yr}	Yield strength of opening reinforcement
G	Shear modulus = $E/2(1 + \nu)$
I	Moment of inertia of a steel tee, with subscript b or t
I_b	Moment of inertia of bottom steel tee
I_e	Moment of inertia of unperforated steel beam or effective moment of inertia of unperforated composite beam
I_o	Moment of inertia of perforated beam
I_T	Moment of inertia of tee
I_t	Moment inertia of top steel tee
J	Torsional constant
K	Shape factor for shear
$K_{i,j}$	Elements of beam stiffness matrix, $i, j = 1, 6$
$[K]$	Stiffness matrix of a beam element
L	Length of a beam
L_b	Unbraced length of compression flange
M	Bending moment at center line of opening
M_{bh}, M_{bl}	Secondary bending moment at high and low moment ends of bottom tee, respectively.
M_m	Maximum nominal bending capacity at the location of an opening
M_n	Nominal bending capacity
M_p	Plastic bending capacity of an unperforated steel beam
M_{pc}	Plastic bending capacity of an unperforated composite beam
M_{th}, M_{tl}	Secondary bending moment at high and low moment ends of top tee, respectively
M_u	Factored bending moment
M_1, M_2	Moments at ends of a beam element
N	Number of shear connectors between the high moment end of an opening and the support
N_o	Number of shear connectors over an opening
P	Axial force in top or bottom tee
$\{P\}$	Force vector for a beam element
P_b	Axial force in bottom tee
P_c	Axial force in concrete for a section under pure bending

$P_{c \min}$	Minimum value of P_c for which Eq. 3-10 is accurate = $F_y(\frac{3}{4}t_w d - \Delta A_s)$	f'_c	Compressive (cylinder) strength of concrete
P_{ch}, P_{cl}	Axial force in concrete at high and low moment ends of opening, respectively, for a section at maximum shear capacity	h_o	Depth of opening
PNA	Plastic neutral axis	ℓ, ℓ_b, ℓ_t	Distance from center of gravity of unperforated beam to center of gravity of a tee section, bottom tee, and top tee, respectively.
P_r	Axial force in opening reinforcement	ℓ_1	Length of extension of reinforcement beyond edge of opening
P_t	Axial force in top tee	ℓ_o	Distance from high moment end of opening to adjacent support
Q_n	Individual shear connector capacity, including reduction factor for ribbed slabs	ℓ_1	Distance from low moment end of opening to adjacent support
R	Ratio of factored load to design capacity at an opening = $V_u/\phi V_n$ = $M_u/\phi M_n$	ℓ_2	Distance from support to point at which deflection is calculated
R_s	Strength reduction factor for shear studs in ribbed slabs	ℓ_3	Distance from high moment end of opening to point at which deflection is calculated
R_{wr}	Required strength of a weld		
S	Clear space between openings	p_o	Opening parameter = $\frac{a_o}{h_o} + \frac{6h_o}{d}$
T'	Tensile force in net steel section		
$\{U\}$	Displacement vector for a beam element	r_Δ	Ratio of midspan deflection of a beam with an opening to midspan deflection of a beam without an opening
V	Shear at opening	s, s_b, s_t	Depth of a tee, bottom tee and top tee, respectively
V_b	Shear in bottom tee	$\bar{s}, \bar{s}_b, \bar{s}_t$	Effective depth of a tee, bottom tee and top tee, respectively, to account for movement of PNA when an opening is reinforced; used <i>only for calculation of ν, when $\nu \geq \mu$</i>
$\bar{V}_c$	Calculated shear carried by concrete slab = $V_{pt}(\mu/\nu - 1) \geq 0$, or $V_{mt}(sh) - V_{pt}$, whichever is less	t	Thickness of flange or reinforcement
V_m	Maximum nominal shear capacity at the location of an opening	t_e	Effective thickness of concrete slab
V_{mb}, V_{mt}	Maximum nominal shear capacity of bottom and top tees, respectively	t_f	Thickness of flange
$V_{mt}(sh)$	Pure shear capacity of top tee	t_s	Total thickness of concrete slab
V_n	Nominal shear capacity	t'_s	Thickness of concrete slab above the rib
V_p	Plastic shear capacity of top or bottom tee	t_w	Thickness of web
$\bar{V}_p$	Plastic shear capacity of unperforated beam	u_1, u_2	Horizontal displacements at ends of a beam element
V_{pb}, V_{pt}	Plastic shear capacity of bottom and top tees, respectively	v_1, v_2	Vertical displacements at ends of a beam element
V_t	Shear in top tee	w	Uniform load
V_u	Factored shear	w_u	Factored uniform load
Z	Plastic section modulus	x	Distance from top of flange to plastic neutral axis in flange or web of a composite beam
a_o	Length of opening	z	Distance between points about which secondary bending moments are calculated
$\bar{a}$	Depth of concrete compressive block	$\alpha_t, \beta_t, \gamma_t$	Variables used to calculate V_{mt}
b	Projecting width of flange or reinforcement	α_v	Ratio of maximum nominal shear capacity to plastic shear capacity of a tee, $\alpha_{vb} = V_{mb}/V_{pb}$ and $\alpha_{vt} = V_{mt}/V_{pt}$
b_e	Effective width of concrete slab	β	Term in stiffness matrix for equivalent beam element at web opening; see Eq. 6-12
b_{em}	Sum of minimum rib widths for ribs that lie within b_e ; for composite beams with longitudinal ribs in slab	ΔA_s	Net reduction in area of steel section due to presence of an opening and reinforcement = $h_o t_w - 2A_r$
b_f	Width of flange		
d	Depth of steel section		
d_h, d_l	Distance from top of steel section to centroid of concrete force at high and low moment ends of opening, respectively.		
d_r	Distance from outside edge of flange to centroid of opening reinforcement; may have different values in top and bottom tees		
e	Eccentricity of opening; always positive for steel sections; positive up for composite sections		

Δ_b	Maximum deflection due to bending of a beam without an opening
Δ_m	Maximum deflection of a beam with an opening due to bending and shear
Δ_o	Deflection through an opening
Δ_{ob}	Bending deflection through an opening
Δ_{os}	Shear deflection through an opening
$\Delta_{p_1}, \Delta_{p_2}$	Components of deflection caused by presence of an opening at a point between high moment end of opening and support
Δ_s	Maximum deflection due to shear of a beam without an opening
Θ_1, Θ_2	Rotations of a beam at supports due to presence of an opening = $EI/(A_y G)$; see Eq. 6-12
θ_H, θ_T	Rotations used to calculate beam deflections due to presence of an opening; see Eq. 6-3
θ_1, θ_2	Rotations at ends of a beam element
λ	Constant used in linear approximation of von Mises yield criterion; recommended value = $\sqrt{2}$

μ Dimensionless ratio relating the secondary bending moment contributions of concrete and opening reinforcement to the product of the plastic shear capacity of a tee and the depth of the tee

$$= \frac{2P_r d_r + P_{ch} d_h - P_{cl} d_l}{V_p s}$$

ν, ν_b, ν_t Ratio of length to depth or length to effective depth for a tee, bottom tee or top tee, respectively = a_o/s or $a_o/\bar{s}$

ν Poisson's ratio

τ Average shear stress

ϕ Resistance factor

Subscripts:

b Bottom tee

m Maximum or mean

n Nominal

t Top tee

u Factored

Chapter 3

DESIGN OF MEMBERS WITH WEB OPENINGS

3.1 GENERAL

This chapter presents procedures to determine the strength of steel and composite beams with web openings. Composite members may have solid or ribbed slabs, and ribs may be parallel or perpendicular to the steel section. Openings may be reinforced or unreinforced. Fig. 3.1 illustrates the range of beam and opening configurations that can be handled using these procedures. The procedures are compatible with the LRFD procedures of the American Institute of Steel Construction, as presented in the *Load and Resistance Factor Design Manual of Steel Construction* (AISC 1986a). With minor modifications, the procedures may also be used with Allowable Stress Design techniques (see section 3.8).

Design equations and design aids (Appendix A) based on these equations accurately represent member strength with a minimum of calculation. The derivation of these equations is explained in Chapter 5.

The design procedures presented in this chapter are limited to members with a yield strength $F_y \leq 65$ ksi meeting the AISC criteria for compact sections (AISC 1986b). Other limitations on section properties and guidelines for detailing are presented in section 3.7. Design examples are presented in Chapter 4.

3.2 LOAD AND RESISTANCE FACTORS

The load factors for structural steel members with web openings correspond to those used in the AISC Load and Resistance Factor Design Specifications for Structural Steel Buildings (AISC 1986b).

Resistance factors, $\phi = 0.90$ for steel members and 0.85 for composite members, should be applied to both moment and shear capacities at openings.

Members should be proportioned so that the factored loads are less than the design strengths in both bending and shear.

$$M_u \leq \phi M_n \quad (3-1)$$

$$V_u \leq \phi V_n \quad (3-2)$$

in which

M_u = factored bending moment

V_u = factored shear

M_n = nominal flexural strength

V_n = nominal shear strength

3.3 OVERVIEW OF DESIGN PROCEDURES

Many aspects of the design of steel and composite members with web openings are similar. At web openings, members may be subjected to both bending and shear. Under the combined loading, member strength is below the strength that can be obtained under either bending or shear alone. Design of web openings consists of first determining the maximum nominal bending and shear capacities at an opening, M_n and V_n , and then obtaining the nominal capacities, M_u and V_u , for the combinations of bending moment and shear that occur at the opening.

For steel members, the maximum nominal bending strength, M_n , is expressed in terms of the strength of the member without an opening. For composite sections, expressions for M_n are based on the location of the plastic neutral axis in the unperforated member. The maximum nomi-

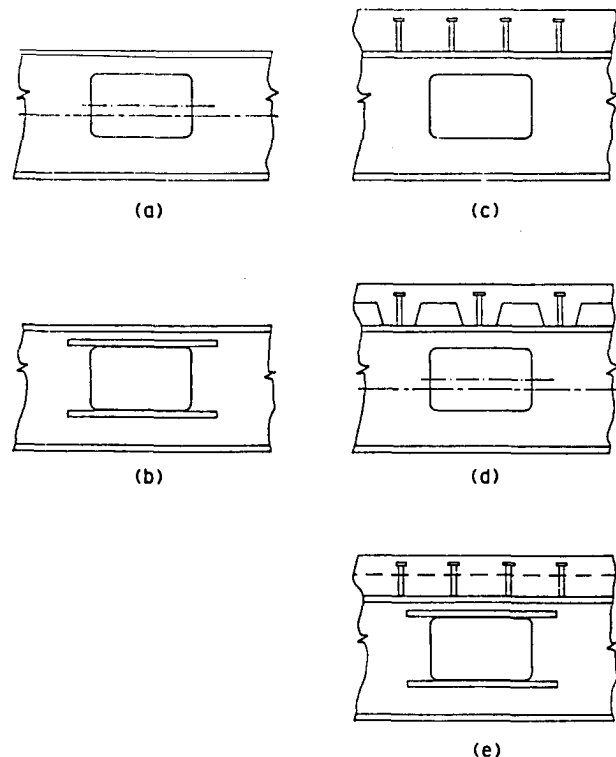


Fig.3.1. Beam and opening configurations, (a) Steel beam with unreinforced opening, (b) steel beam with reinforced opening, (c) composite beam, solid slab, (d) composite beam, ribbed slab with transverse ribs, (e) composite beam with reinforced opening, ribbed slab with longitudinal ribs.

nal shear capacity, V_m , is expressed as the sum of the shear capacities, V_{mt} and V_{mb} , for the regions above and below the opening (the top and bottom tees).

The design expressions for composite beams apply to openings located in positive moment regions. The expressions for steel beams should be used for openings placed in negative moment regions of composite members.

The next three sections present the moment-shear interaction curve and expressions for M_m and V_m used to design members with web openings. Guidelines for member proportions follow the presentation of the design equations.

3.4 MOMENT-SHEAR INTERACTION

Simultaneous bending and shear occur at most locations within beams. At a web opening, the two forces interact to produce lower strengths than are obtained under pure bending or pure shear alone. Fortunately at web openings, the interaction between bending and shear is weak, that is, neither the bending strength nor the shear strength drop off rapidly when openings are subjected to combined bending and shear.

The interaction between the design bending and shear strengths, ϕM_n and ϕV_n , is shown as the solid curve in Fig. 3.2 and expressed as

$$\left(\frac{\phi M_n}{\phi M_m}\right)^3 + \left(\frac{\phi V_n}{\phi V_m}\right)^3 = R^3, R = 1 \quad (3-3)$$

Additional curves are included in Fig. 3.2 with values of R ranging from 0.6 to 1.2. The factored loads at an opening,

V_u and M_u , are checked using the interaction curve by plotting the point $(V_u/\phi V_m, M_u/\phi M_m)$. If the point lies inside the $R = 1$ curve, the opening meets the requirements of Eqs. 3-1 and 3-2, and the design is satisfactory. If the point lies outside the curve, the design is not satisfactory. A large-scale version of Fig. 3.2, suitable for design, is presented in Fig. A.1 of Appendix A.

The value of R at the point $(V_u/\phi V_m, M_u/\phi M_m)$ allows ϕV_n and ϕM_n to be obtained from the applied loads.

$$\phi V_n = \frac{V_u}{R} \quad (3-4a)$$

$$\phi M_n = \frac{M_u}{R} \quad (3-4b)$$

Alternatively, ϕV_n and ϕM_n can be calculated directly.

$$\phi V_n = \phi V_m \left[\frac{\left(\frac{M_u}{\phi M_m}\right)^3}{\left(\frac{V_u}{\phi V_m}\right)^3} + 1 \right]^{-1/3} \quad (3-5a)$$

$$\phi M_n = \phi V_n \left(\frac{M_u}{V_u}\right) = \phi M_m \left[\frac{\left(\frac{V_u}{\phi V_m}\right)^3}{\left(\frac{M_u}{\phi M_m}\right)^3} + 1 \right]^{-1/3} \quad (3-5b)$$

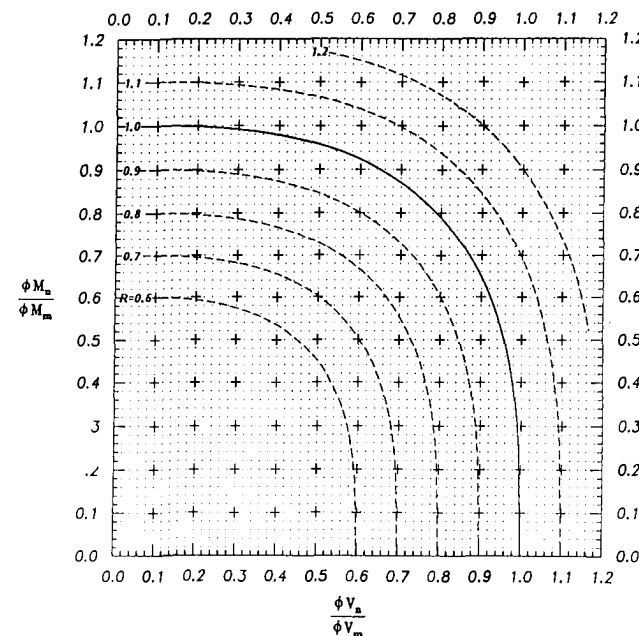


Fig 3.2. Design interaction curves. $R = V_u/\phi V_n = M_u/\phi M_n$.

3.5 EQUATIONS FOR MAXIMUM MOMENT CAPACITY, M_m

The equations presented in this section may be used to calculate the maximum moment capacity of steel (Fig. 3.3) and composite (Fig. 3.4) members constructed with compact steel sections. The equations are presented for rectangular openings. Guidelines are presented in section 3.7 to allow the expressions to be used for circular openings.

The openings are of length, a_o , height, h_o , and may have an eccentricity, e , which is measured from the center line of the steel section. For steel members, e is positive, whether the opening is above or below the center line. For composite members, e is positive in the upward direction.

The portion of the section above the opening (the top tee) has a depth s_t , while the bottom tee has a depth of s_b . If reinforcement is used, it takes the form of bars above and below the opening, welded to one or both sides of the web. The area of the reinforcement on each side of the opening is A_r .

For composite sections, the slab is of total depth, t_s , with

a minimum depth of t'_s . Other dimensions are as shown in Figs. 3.3 and 3.4.

a. Steel beams

The nominal capacity of a steel member with a web opening, M_m , is expressed in terms of the capacity of the member without an opening, M_p .

Unreinforced openings

For members with unreinforced openings,

$$M_m = M_p \left[1 - \frac{\Delta A_s \left(\frac{h_o}{4} + e \right)}{Z} \right] \quad (3-6)$$

in which

- $M_p = F_y Z$
- $\Delta A_s = h_o t_w$
- h_o = depth of opening
- t_w = thickness of web
- e = eccentricity of opening = $|e|$
- Z = plastic section modulus of member without opening
- F_y = yield strength of steel

Reinforced openings

For members with reinforced openings,

$$M_m = M_p \left[1 - \frac{t_w \left(\frac{h_o^2}{4} + h_o e - e^2 \right) - A_r h_o}{Z} \right] \leq M_p \quad (3-7)$$

for $t_w e < A_r$

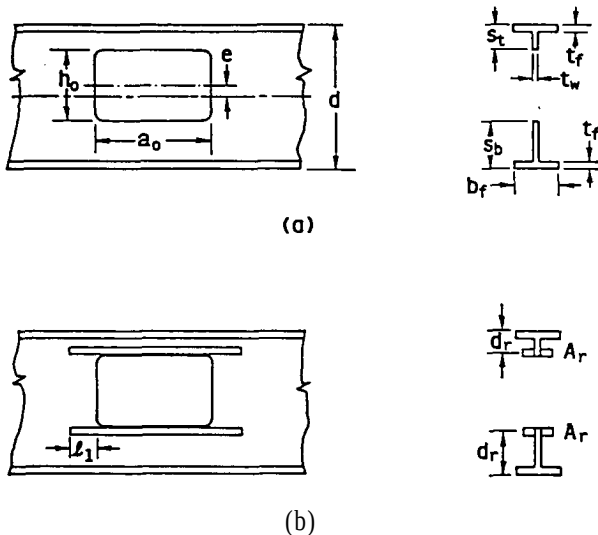


Fig. 3.3. Opening configurations for steel beams, (a) Unreinforced opening, (b) reinforced opening.

$$M_m = M_p \left[1 - \frac{\Delta A_s \left(\frac{h_o}{4} + e - \frac{A_r}{2t_w} \right)}{Z} \right] \leq M_p \quad (3-8)$$

for $t_w e \geq A_r$,

in which $\Delta A_s = h_o t_w - 2A_r$,

b. Composite beams

The expressions for the nominal capacity of a composite member with a web opening (Fig. 3.4) in pure bending, M_m , apply to members both with and without reinforcement.

Plastic neutral axis above top of flange

For beams in which the plastic neutral axis, PNA, in the unperforated member is located at or above the top of the flange,

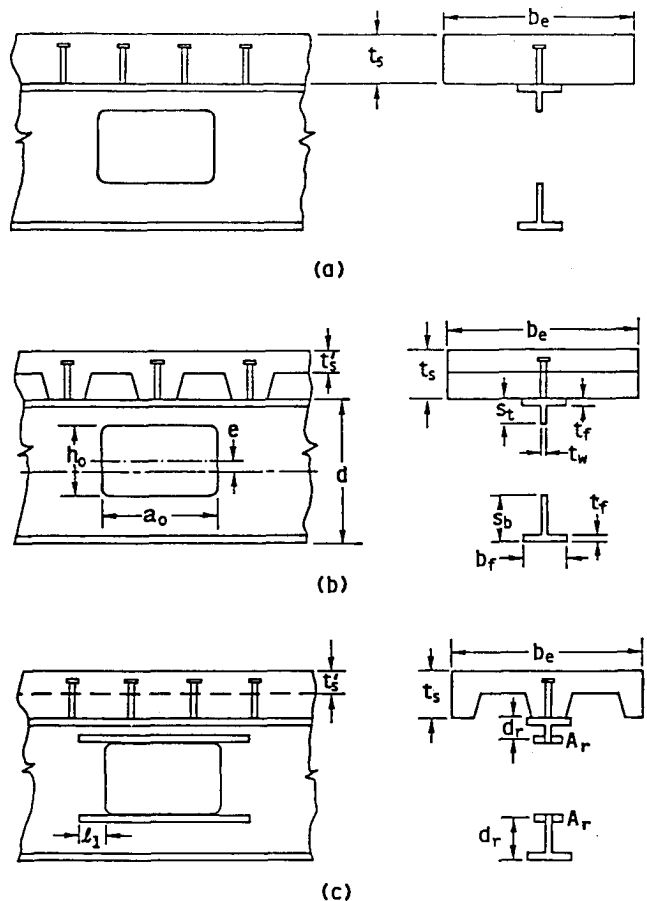


Fig. 3.4. Opening configurations for composite beams. (a) Unreinforced opening, solid slab, (b) unreinforced opening, ribbed slab with transverse ribs, (c) reinforced opening, ribbed slab with longitudinal ribs.

the value of M_m may be approximated in terms of the capacity of the unperforated section, M_{pc} .

$$M_m = M_{pc} \left[\frac{A_{sn}}{A_s} + \frac{F_y \Delta A_s e}{M_{pc}} \right] \leq M_{pc} \quad (3-9)$$

in which

M_{pc} = nominal capacity of the unperforated composite section, at the location of the opening

A_s = cross-sectional area of steel in the unperforated member

A_{sn} = net area of steel section with opening and reinforcement

$$A_s - h_o t_w + 2A_r = A_s - \Delta A_s$$

$$\Delta A_s = h_o t_w - 2A_r$$

e = eccentricity of opening, positive upward

Equation 3-9 is always conservative for $A_{sn} \leq A_s$. The values of M_{pc} can be conveniently obtained from Part 4 of the AISC Load and Resistance Factor Design Manual (AISC 1986a).

Plastic neutral axis below top of flange

For beams in which the PNA in the unperforated member is located below the top of the flange and $P_c \geq P_{c \min} = F_y(\frac{3}{4}t_w d - \Delta A_s)$, the value of M_m may be approximated using

$$M_m = F_y A_{sn} \frac{d}{2} + F_y \Delta A_s e + P_c \left(t_s - \frac{\bar{a}}{2} \right) \leq M_{pc} \quad (3-10)$$

in which

t_s = thickness of slab

$$\bar{a} = \text{depth of concrete stress block} = \frac{P_c}{0.85 f'_c b_e}$$

P_c = force in the concrete (Fig. 3.5)

P_c is limited by the concrete capacity, the stud capacity from the high moment end of the opening to the support, and the tensile capacity of the net steel section.

$$P_c \leq 0.85 f'_c b_e t_e \quad (3-11a)$$

$$P_c \leq N Q_n \quad (3-11b)$$

$$P_c \leq F_y A_{sn} \quad (3-11c)$$

in which

$t_e = t_s$ for solid slabs

$t_e = t'_s$ for ribbed slabs with transverse ribs

$t_e = (t_s + t'_s)/2$ for ribbed slabs with longitudinal ribs

N = number of shear connectors between the high moment end of the opening and the support

Q_n = individual shear connector capacity, including reduction factor for ribbed slabs (AISC 1986b)

b_e = effective width of concrete slab (AISC 1986b)

Equation 3-10 is also accurate for members with the PNA in the unperforated section located at or above the top of the flange.

If $P_c < F_y(\frac{3}{4}t_w d - \Delta A_s)$, the more accurate expressions given in section 5.5 should be used to calculate M_m .

3.6 EQUATIONS FOR MAXIMUM SHEAR CAPACITY, V_m

The equations presented in this section may be used to calculate the maximum shear strength of steel and composite members constructed with compact steel sections. The equations are presented for rectangular openings and used to develop design aids, which are presented at the end of this section and in Appendix A. Guidelines are presented in the next section to allow the expressions to be used for circular openings. Dimensions are as shown in Figs. 3.3 and 3.4.

The maximum nominal shear capacity at a web opening, V_m , is the sum of the capacities of the bottom and top tees.

$$V_m = V_{mb} + V_{mt} \quad (3-12)$$

a. General equation

α_v , the ratio of nominal shear capacity of a tee, V_{mb}

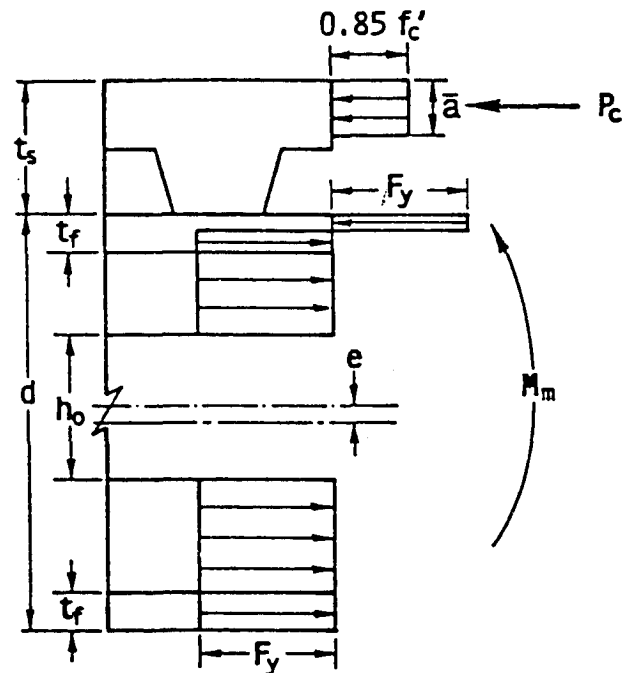


Fig. 3.5. Region at web opening at maximum moment, composite beam.

or V_{mt} , to the plastic shear capacity of the web of the tee, V_{pb} or V_{pt} , is calculated as

$$\alpha_v = \frac{V_{mb}}{V_{pb}} \text{ or } \frac{V_{mt}}{V_{pt}} = \frac{\sqrt{6} + \mu}{\nu + \sqrt{3}} \leq 1 \quad (3-13)$$

in which V_{pb} or $V_{pt} = \frac{F_y t_w s}{\sqrt{3}}$

ν = aspect ratio of tee = $\frac{a_o}{s}$, use $\nu = \frac{a_o}{\bar{s}}$
when reinforcement is used

s = depth of tee, s_b or s_t

$\bar{s} = s - \frac{A_r}{2b_f}$, used to calculate ν
when reinforcement is used

b_f = width of flange

a_o = length of opening

Subscripts "b" and "t" indicate the bottom and top tees, respectively.

$$\mu = \frac{2P_r d_r + P_{ch} d_h - P_{cl} d_l}{V_p s} \quad (3-14)$$

in which (see Fig. 3.5)

V_p = V_{pb} or V_{pt}
 P_r = force in reinforcement along edge of opening

$$F_y A_r \leq \frac{F_y t_w a_o}{2\sqrt{3}}$$

d_r = distance from outside edge of flange to centroid of reinforcement

P_{ch} and P_{cl} = concrete forces at high and low moment ends of opening, respectively. For top tee in composite sections only. See Eqs. 3-15a through 3-16.

d_h and d_l = distances from outside edge of top flange to centroid of concrete force at high and low moment ends of opening, respectively. For top tee in composite sections only. See Eqs. 3-17 through 3-18b.

For reinforced openings, s should be replaced by $\bar{s}$ in the calculation of ν only.

For tees without concrete, $\mu = 2P_r d_r / (V_p s)$. For tees without concrete or reinforcement, $\mu = 0$. For eccentric openings, $s_b \neq s_t$ and $\nu_b \neq \nu_t$.

Equations 3-13 and 3-14 are sufficient for all types of construction, with the exception of top tees in composite beams which are covered next.

b. Composite beams

The following expressions apply to the *top tee of composite members*. They are used in conjunction with Eqs. 3-13 and 3-4,

P_{ch} , the concrete force at the high moment end of the opening (Eq. 3-14, Fig. 3.6), is

$$P_{ch} \leq 0.85 f'_c b_e t_e \quad (3-15a)$$

$$P_{ch} \leq N Q_n \quad (3-15b)$$

$$P_{ch} \leq F_y A_{st} \quad (3-15c)$$

in which A_{st} = net steel area of top tee

P_{cl} , the concrete force at the low moment end of the opening (Fig. 3.6), is

$$P_{cl} = P_{ch} - N_o Q_n \geq 0 \quad (3-16)$$

in which N_o = number of shear connectors over the opening.

N in Eq. 3-15b and N_o in Eq. 3-16 include only connectors completely within the defined range. For example, studs on the edges of an opening are not included.

d_h and d_l , the distances from the top of the flange to the centroid of the concrete force at the high and the low moment ends of the opening, respectively, are

$$d_h = t_s - \frac{P_{ch}}{1.7f'_c b_e} \quad (3-17)$$

$$d_l = \frac{P_{cl}}{1.7f'_c b_e} \text{ for solid slabs} \quad (3-18a)$$

$$d_l = t_s - t'_s + \frac{P_{cl}}{1.7f'_c b_e} \text{ for ribbed slabs with transverse ribs} \quad (3-18b)$$

For ribbed slabs with longitudinal ribs, d_l is based on the centroid of the compressive force in the concrete considering all ribs that lie within the effective width b_e (Fig. 3.4). In this case, d_l can be conservatively obtained using Eq. 3-18a, replacing b_e by b_{em} , the sum of the minimum rib widths for the ribs that lie within b_e .

If the ratio of V_{mt} to V_{pt} in Eq. 3-13 exceeds 1, then an alternate expression must be used.

$$\alpha_v = \frac{V_{mt}}{V_{pt}} = \frac{\mu}{\nu} \geq 1 \quad (3-19)$$

in which $\nu = a_o/s$ for both reinforced and unreinforced openings.

To evaluate μ in Eq. 3-19, the value of P_{ch} in Eq. 3-15 must be compared with the tensile force in the flange and reinforcement, since the web has fully yielded in shear.

$$P_{ch} \leq F_y [t_f (b_f - t_w) + A_r] \quad (3-20)$$

in which

b_d = width of flange

t_f = thickness of flange

Equation 3-20 takes the place of Eq. 3-15c.

If Eq. 3-20 governs P_{ch} instead of Eq. 3-15, P_{cl} , d_h , d_l , and μ must also be recalculated using Eqs. 3-16, 3-17, 3-18, and 3-14, respectively.

Finally, V_{mt} must not be greater than the pure shear capacity of the top tee, $V_{mt}(sh)$.

$$V_{mt}(sh) = V_{pt} + 0.11 f_c^{r/2} A_{vc}, \text{ kips} \quad (3-21)$$

in which f_c^r and $f_c^{r/2}$ are in ksi

A_{vc} = effective concrete shear area = $3t_s t_e$.

c. Design aids

A design aid representing α_v from Eq. 3-13 is presented in Figs. 3.7 and A.2 for values of ν ranging from 0 to 12 and values of μ ranging from 0 to 11. This design aid is applicable to unreinforced and reinforced tees without concrete, as well as top tees in composite members, with α_v (V_{mb}/V_{pb} or V_{mt}/V_{pt}) less than or equal to 1.

A design aid for α_v from Eq. 3-19 for the top tee in composite members with $\alpha_v \geq 1$ is presented in Figs. 3.8 and A.3. This design aid is applicable for values of ν from 0 to 12 and values of μ from 0.5 to 23. If $\alpha_v > 1$, μ must be recalculated if Eq. 3-20 controls P_{ch} , and a separate check must be made for $V_{mt}(sh)$ using Eq. 3-21.

The reader will note an offset at $\alpha_v = 1$ between Figs. A.2 and A.3 (Figs. 3.7 and 3.8). This offset is the result of a discontinuity between Eqs. 3-13 and 3-19 at $\nu = \mu$. If α_v appears to be ≥ 1 on Fig. A.2 and ≤ 1 on Fig. A.3, use $\alpha_v = 1$.

3.7 GUIDELINES FOR PROPORTIONING AND DETAILING BEAMS WITH WEB OPENINGS

To ensure that the strength provided by a beam at a web opening is consistent with the design equations presented in sections 3.4-3.6, a number of guidelines must be followed. Unless otherwise stated, these guidelines apply to unreinforced and reinforced web openings in both steel and composite beams. All requirements of the AISC Specifications (AISC 1986b) should be applied. The steel sections should meet the AISC requirements for compact sections in both composite and non-composite members. $F_y \leq 65$ ksi.

a. Stability considerations

To ensure that local instabilities do not occur, consideration must be given to local buckling of the compression flange, web buckling, buckling of the tee-shaped compression zone above or below the opening, and lateral buckling of the compression flange.

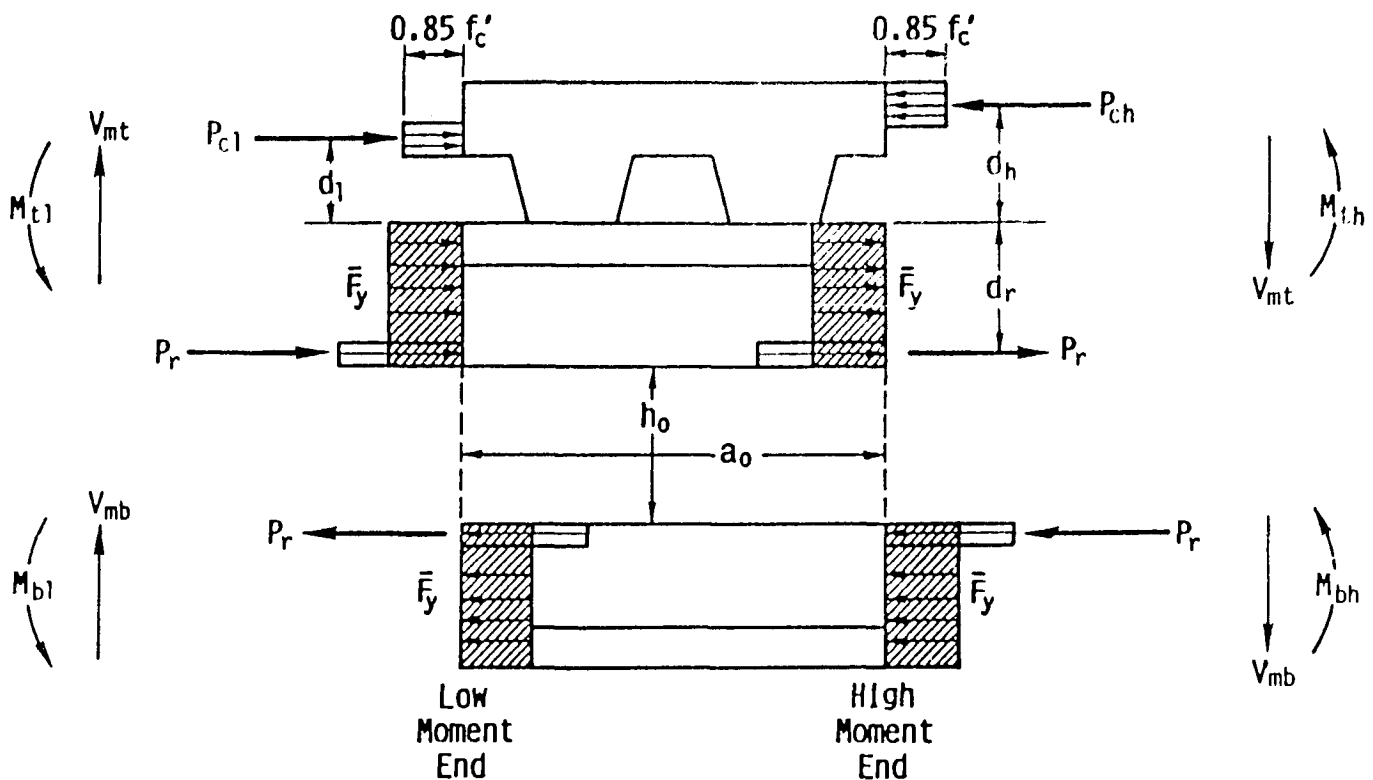


Fig. 3.6. Region at web opening under maximum shear.

1. Local buckling of compression flange or reinforcement

To ensure that local buckling does not occur, the AISC (AISC 1986b) criteria for compact sections applies. The width to thickness ratios of the compression flange or web reinforcement are limited by

$$\frac{b}{t} \leq \frac{65}{\sqrt{F_y}} \quad (3-22)$$

in which

b = projecting width of flange or reinforcement

t = thickness of flange or reinforcement

F_y = yield strength in ksi

For a flange of width, b_f , and thickness, t_f , Eq. 3-22 becomes

$$\frac{b_f}{2t_f} \leq \frac{65}{\sqrt{F_y}} \quad (3-23)$$

2. Web buckling

To prevent buckling of the web, two criteria should be met:

(a) The opening parameter, p_o , should be limited to a maximum value of 5.6 for steel sections and 6.0 for composite sections.

$$p_o = \frac{a_o}{h_o} + \frac{6h_o}{d} \quad (3-24)$$

in which a_o and h_o = length and width of opening, respectively, d = depth of steel section

(b) The web width-thickness ratio should be limited as follows

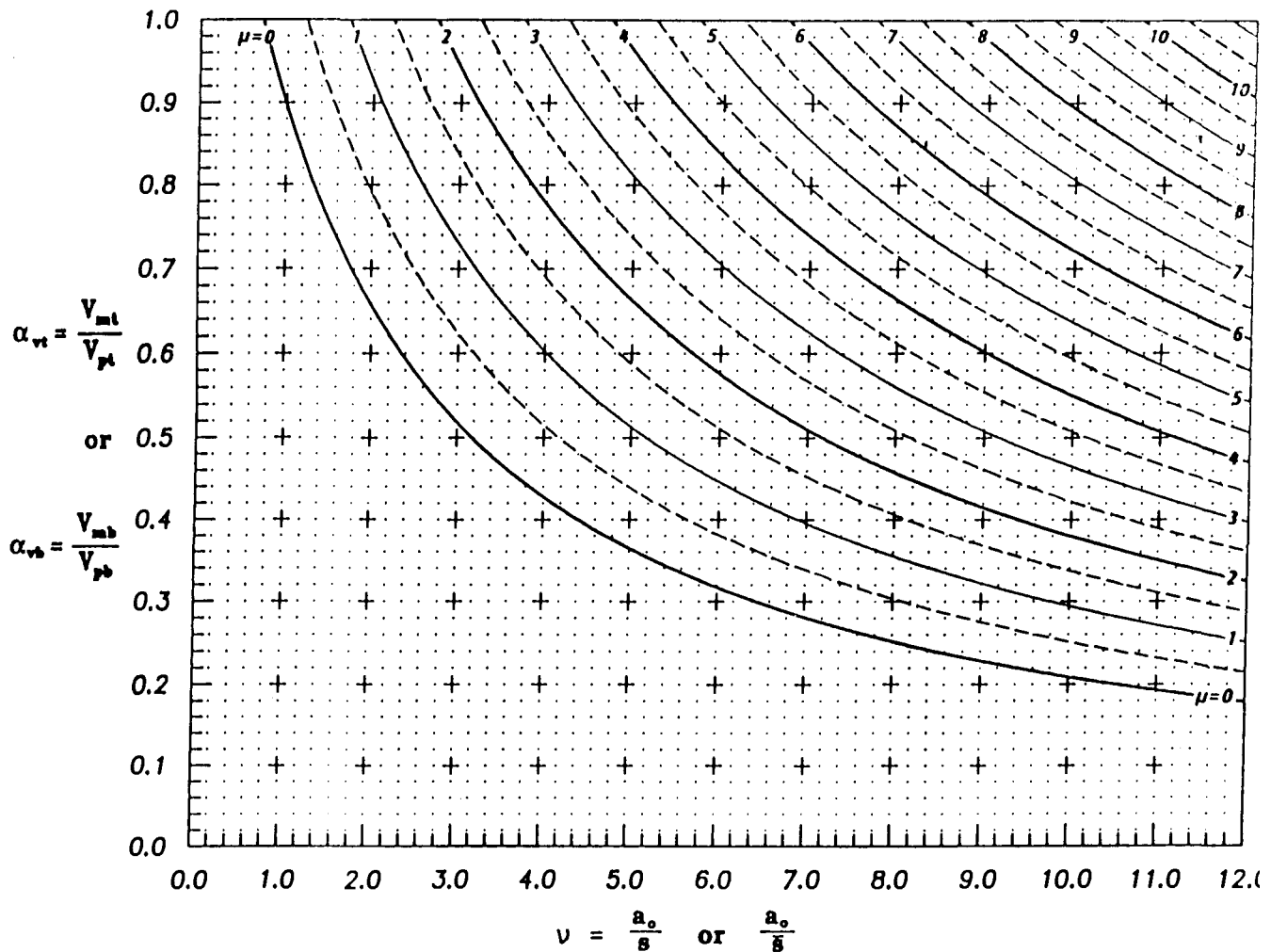


Fig. 3.7. Design aid relating α_v , the ratio of the nominal maximum shear strength to the plastic shear strength of a tee, to v , the ratio of length to depth or effective length to depth of a tee. V_{mb}/V_{pb} or $V_{mt}/V_{pt} \leq 1$.

$$\frac{d - 2t_f}{t_w} \leq \frac{520}{\sqrt{F_y}} \quad (3-25)$$

in which t_w = thickness of web

If $(d - 2t_f)/t_w \leq 420/\sqrt{F_y}$, the web qualifies as stocky. In this case, the upper limit on a_o/h_o is 3.0 and the upper limit on V_m (maximum nominal shear capacity) for non-composite sections is $\frac{2}{3}\bar{V}_p$, in which $\bar{V}_p = F_y t_w d/\sqrt{3}$, the plastic shear capacity of the unperforated web. For composite sections, this upper limit may be increased by $\bar{V}_c$ which equals $V_{pt}(\mu/\nu - 1) \geq 0$, or $V_{mt}(sh) - V_{pt}$, whichever is less. All standard rolled W shapes (AISC 1986a) qualify as stocky members.

If $420/\sqrt{F_y} < (d - 2t_f)/t_w \leq 520/\sqrt{F_y}$, then a_o/h_o should be limited to 2.2, and V_m should be limited to $0.45 \bar{V}_p$ for both composite and non-composite members.

The limits on opening dimensions to prevent web buck-

ling, along with an additional criterion from section 3.7bl, are summarized in Fig. 3.9.

3. Buckling of tee-shaped compression zone

For steel beams only: The tee which is in compression should be investigated as an axially loaded column following the procedures of AISC (1986b). For unreinforced members this is not required when the aspect ratio of the tee ($\nu = a_o/s$) is less than or equal to 4. For reinforced openings, this check is only required for large openings in regions of high moment.

4. Lateral buckling

For steel beams only: In members subject to lateral buckling of the compression flange, strength should not be governed by strength at the opening (calculated without regard to lateral buckling).

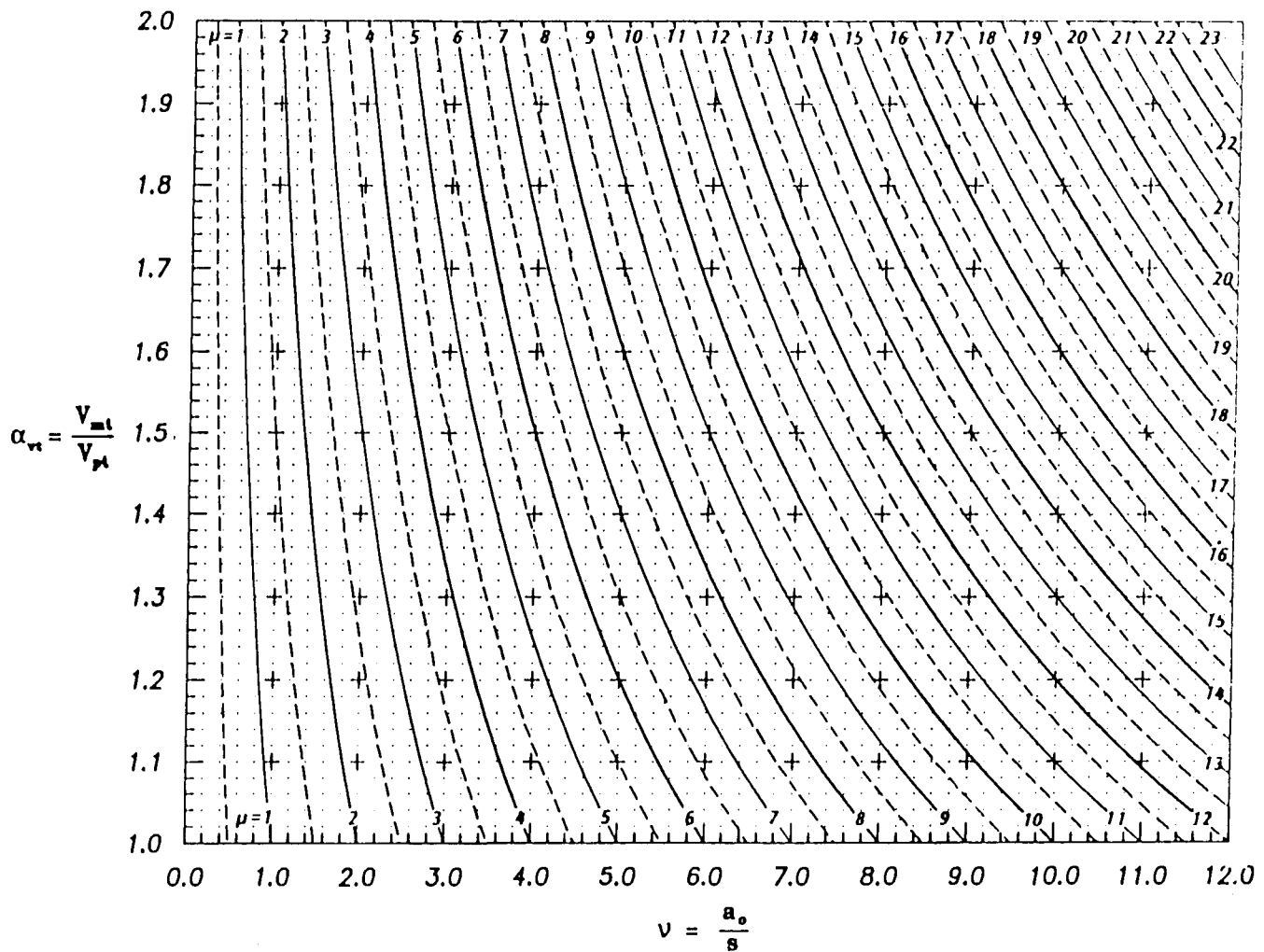


Fig. 3.8. Design aid relating α_v , the ratio of the nominal maximum shear strength to the plastic shear strength of the top tee, to ν , the length-to-depth ratio of the tee.

$1 \leq V_{mt}/V_{pt} \leq 2$ composite members only.

In members with unreinforced openings or reinforced openings with the reinforcement placed on both sides of the web, the torsional constant, J , should be multiplied by

$$\left[1 - \left(\frac{a_o}{L_b} \right) \frac{\Delta A_s}{t_w(d + 2b_f)} \right]^2 \leq 1 \quad (3-26)$$

in which L_b = unbraced length of compression flange

$$\Delta A_s = h_o t_w - 2A_r$$

In members reinforced on only one side of the web, $A_r = 0$ for the calculation of ΔA_s in Eq. 3-26. Members reinforced on one side of the web should not be used for long laterally unsupported spans. For shorter spans the lateral bracing closest to the opening should be designed for an additional load equal to 2 percent of the force in the compression flange.

b. Other considerations

1. Opening and tee dimensions

Opening dimensions are restricted based on the criteria in section 3.7a. Additional criteria also apply.

The opening depth should not exceed 70 percent of the section depth ($h_o \leq 0.7d$). The depth of the top tee should not be less than 15 percent of the depth of the steel section ($s_t \geq 0.15d$). The depth of the bottom tee, s_b , should not be less than $0.15d$ for steel sections or $0.12d$ for composite sections. The aspect ratios of the tees ($v = a_o/s$) should not be greater than 12 ($a_o/s_b \leq 12$, $a_o/s_t \leq 12$).

2. Corner radii

The corners of the opening should have minimum radii at least 2 times the thickness of the web, $2t_w$, or $\frac{3}{8}$ in., whichever is greater.

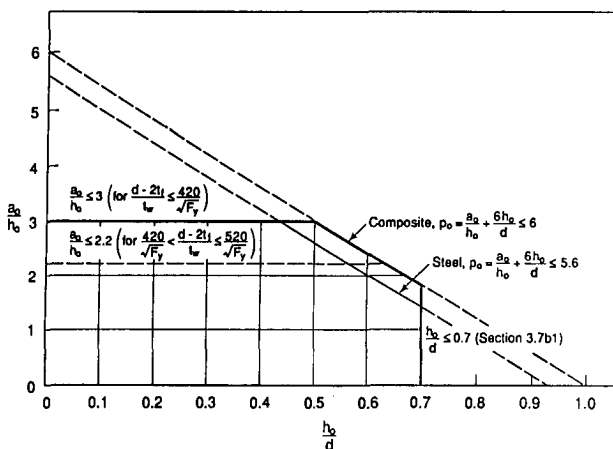


Fig. 3.9. Limits on opening dimensions.

3. Concentrated loads

No concentrated loads should be placed above an opening.

Unless needed otherwise, bearing stiffeners are not required to prevent web crippling in the vicinity of an opening due to a concentrated load if

$$(1) \frac{d - 2t_f}{t_w} \leq \frac{420}{\sqrt{F_y}} \quad (3-27a)$$

$$\frac{b}{t} \leq \frac{54}{\sqrt{F_y}} \quad (3-27b)$$

and the load is placed at least $d/2$ from the edge of the opening,

$$(2) \frac{d - 2t_f}{t_w} \leq \frac{520}{\sqrt{F_y}} \quad (3-28a)$$

$$\frac{b}{t} \leq \frac{65}{\sqrt{F_y}} \quad (3-28b)$$

and the load is placed at least d from the edge of the opening.

In any case, the edge of an opening should not be closer than a distance d to a support.

4. Circular openings

Circular openings may be designed using the expressions in sections 3.5 and 3.6 by using the following substitutions for h_o and a_o .

Unreinforced web openings:

$$h_o = D_o \text{ for bending} \quad (3-29a)$$

$$h_o = 0.9 D_o \text{ for shear} \quad (3-29b)$$

$$a_o = 0.45 D_o \quad (3-29c)$$

in which D_o = diameter of circular opening.

Reinforced web openings:

$$h_o = D_o \text{ for bending and shear} \quad (3-30a)$$

$$a_o = 0.45 D_o \quad (3-30b)$$

5. Reinforcement

Reinforcement should be placed as close to an opening as possible, leaving adequate room for fillet welds, if required on both sides of the reinforcement. Continuous welds should be used to attach the reinforcement bars. A fillet weld may be used on one or both sides of the bar within the length of the opening. However, fillet welds should be used on both sides of the reinforcement on extensions past the opening. The required strength of the weld within the length of the opening is,

$$R_{wr} = \phi 2P_r \quad (3-31)$$

in which

R_{wr} = required strength of the weld

$\phi = 0.90$ for steel beams and 0.85 for composite beams

$$P_r = F_y A_r \leq \frac{F_y t_w a_o}{2\sqrt{3}}$$

A_r = cross-sectional area of reinforcement above or below the opening.

The reinforcement should be extended beyond the opening by a distance $\ell_1 = a_o/4$ or $A_r \sqrt{3}/(2t_w)$, whichever is greater, on each side of the opening (Figs 3.3 and 3.4). Within each extension, the required strength of the weld is

$$R_{wr} = \phi F_y A_r \quad (3-32)$$

If reinforcing bars are used on only one side of the web, the section should meet the following additional requirements.

$$A_r \leq \frac{A_f}{3} \quad (3-33)$$

$$\frac{a_o}{h_o} \leq 2.5 \quad (3-34)$$

$$\frac{s_t}{t_w} \text{ or } \frac{s_b}{t_w} \leq \frac{140}{\sqrt{F_y}} \quad (3-35)$$

$$\frac{M_u}{V_u d} \leq 20 \quad (3-36)$$

in which A_f = area of flange

M_u and V_u = factored moment and shear at centerline of opening, respectively.

6. Spacing of openings

Openings should be spaced in accordance with the following criteria to avoid interaction between openings.

Rectangular openings: $S \geq h_o$ (3-37a)

$$S \geq a_o \left(\frac{V_u / \phi \bar{V}_p}{1 - V_u / \phi \bar{V}_p} \right) \quad (3-37b)$$

Circular openings: $S \geq 1.5 D_o$ (3-38a)

$$S \geq D_o \left(\frac{V_u / \phi \bar{V}_p}{1 - V_u / \phi \bar{V}_p} \right) \quad (3-38b)$$

in which S = clear space between openings.

In addition to the requirements in Eqs. 3-37 and 3-38, openings in composite beams should be spaced so that

$$S \geq a_o \quad (3-39a)$$

$$S \geq 2.0 d \quad (3-39b)$$

c. Additional criteria for composite beams

In addition to the guidelines presented above, composite members should meet the following criteria.

1. Slab reinforcement

Transverse and longitudinal slab reinforcement ratios should be a minimum of 0.0025 , based on the *gross* area of the slab, within a distance d or a_o , whichever is greater, of the opening. For beams with longitudinal ribs, the transverse reinforcement should be below the heads of the shear connectors.

2. Shear connectors

In addition to the shear connectors used between the high moment end of the opening and the support, a minimum of two studs per foot should be used for a distance d or a_o , whichever is greater, from the high moment end of the opening toward the direction of *increasing* moment.

3. Construction loads

If a composite beam is to be constructed without shoring, the section at the web opening should be checked for adequate strength as a *non-composite* member under factored dead and construction loads.

3.8 ALLOWABLE STRESS DESIGN

The safe and accurate design of members with web openings requires that an ultimate strength approach be used. To accommodate members designed using ASD, the expressions presented in this chapter should be used with $\phi = 1.00$ and a load factor of 1.7 for both dead and live loads. These factors are in accord with the Plastic Design Provisions of the AISC ASD Specification (1978).

Chapter 4

DESIGN SUMMARIES AND EXAMPLE PROBLEMS

4.1 GENERAL

Equations for maximum bending capacity and details of opening design depend on the presence or absence of a composite slab and opening reinforcement. However, the overall approach, the basic shear strength expressions, and the procedures for handling the interaction of bending and shear are identical for all combinations of beam type and opening configuration. Thus, techniques that are applied in the design of one type of opening can be applied to the design of all.

Tables 4.1 through 4.4 summarize the design sequence, design equations and design aids that apply to steel beams with unreinforced openings, steel beams with reinforced openings, composite beams with unreinforced openings, and composite beams with reinforced openings, respectively. Table 4.5

summarizes proportioning and detailing guidelines that apply to all beams.

Sections 4.2 through 4.6 present design examples. The examples in sections 4.2, 4.4, 4.5, and 4.6 follow the LRFD approach. In section 4.3, the example in section 4.2 is resolved using the ASD approach presented in section 3.8.

A typical design sequence involves cataloging the properties of the section, calculating appropriate properties of the opening and the tees, and checking these properties as described in sections 3.7a and b. The strength of a section is determined by calculating the maximum moment and shear capacities and then using the interaction curve (Fig. A.1) to determine the strength at the opening under the combined effects of bending and shear.

Designs are completed by checking for conformance with additional criteria in sections 3.7b and c.

Table 4.1
Design of Steel Beams with Unreinforced Web Openings

See sections 3.7a1-3.7b1 or Table 4.5 a1-b1 for proportioning guidelines.

Calculate maximum moment capacity: Use Eq. 3-6.

$$M_m = M_p \left[1 - \frac{\Delta A_s \left(\frac{h_o}{4} + e \right)}{Z} \right] \quad (3-6)$$

in which $\Delta A_s = h_w t_w$ and $M_p = F_y Z$.

Calculate maximum shear capacity: Use Fig. A.2 or Eq. 3-13 to obtain α_v . For the bottom tee, use $\nu = a_o/s_b$ and $\mu = 0$. For the top tee, use $\nu = a_o/s_t$ and $\mu = 0$.

$$V_{mb} = V_{pb} \alpha_{vb} \text{ and } V_{mt} = V_{pt} \alpha_{vt}$$

$$\alpha_v = \frac{\sqrt{6} + \mu}{\nu + \sqrt{3}} = \frac{\sqrt{6}}{\nu + \sqrt{3}} \leq 1 \quad (3-13)$$

in which $V_{pb} = \frac{F_y t_w s_b}{\sqrt{3}}$ and $V_{pt} = \frac{F_y t_w s_t}{\sqrt{3}}$

$$V_m = V_{mb} + V_{mt} \leq \frac{2}{3} \bar{V}_p \quad (3-12)$$

Check moment-shear interaction: Use Fig. A.1 with $\phi = 0.9$. $R \leq 1.0$.

See sections 3.7b2-3.7b4 and 3.7b6 or Table 4.5b2-b4 and b6 for other guidelines.