Steel Design
After College

Developed by:

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Chicago, IL

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This course was developed

by

Research and Development group at

Walter P. Moore

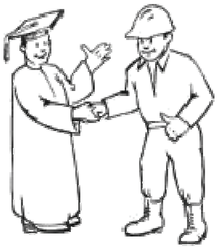
in Collaboration with

American Institute of Steel Construction

WALTER P MOORE
ENGINEERS + CONSULTANTS

Purpose

- Typical undergraduate takes 1 to 2 steel design courses
- These courses provide fundamentals needed to design steel structures
- The knowledge gained in these courses is prerequisite but not sufficient for designing steel structures
- Theoretical knowledge must be supplemented by practical experience
- The purpose of this course is to fill the gap



Scope

Seven Sessions

Design of Steel Flexural Members	1 Hr
Composite Beam Design	1 Hr
Lateral Design of Steel Buildings	1 Hr
Deck Design	½ Hr
Diaphragms	1 Hr
Base Plates and Anchor Rods	¾ Hr
Steel Trusses and Computer Analysis verification	¾ Hr

Strength Design of Steel Flexural Members

- Local buckling, LTB and yielding and what it means in practical applications
- Strategies for effective beam bracing
- Cantilevers
- Leveraging C_b for economical design
- Beams under uplift

Steel Composite Beam Design Concepts

- Review of composite beam design
- Serviceability issues and detailing
- Different ways to influence composite beam design

Lateral Analysis of Steel Buildings for Wind Loads

- **Serviceability, strength and stability limit states**
- **Analysis parameters and modeling assumptions**
- **Acceptability criteria**
- **Brace beam design**
- **Role of gravity columns**

Steel Deck Design

- **Types of steel decks and their application of use**
- **Designing for building insurance requirements**
- **Types of finishes and when to specify**
- **Special design and construction issues**

Role Of Diaphragm in Buildings and Its Design

- Types of diaphragms and design methods
- Diaphragm deflections
- Diaphragm modeling
- Potential diaphragm problems

Base Plates and Anchor Rod Design

- Material specifications
- Consideration for shear and tension
- Interface with concrete

Steel Trusses and Computer Analysis Verification

Trusses

- **Types of trusses**
- **Strategies in geometry**
- **Member selection**
- **Truss Connections**

Computer modeling issues

What is not covered?

AISC offers number of continuing education courses for steel design

- **Stability of Columns and Frames**
- **Bolting and Welding**
- **Field Fixes**
- **Seismic Design**
- **Blast and Progressive Collapse Mitigation**
- **Connection Design**
- **Erection related issues**
- **Each of these seminars covers the specific topic in detail**

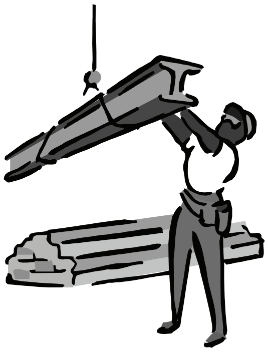
Relax and Enjoy

- Ask questions as we go
- Detail discussion if required at the end of each session
- No need to take notes. Handout has everything that is on the slides

Strength Design of Structural Steel Flexural Members (Non- Composite)

Scope

- Limit states
  - Strength
  - Serviceability (outside scope)
- Strength limit states
  - Local buckling
  - Lateral torsional buckling
  - Yielding
- Effective beam bracing
  - Compression flange lateral support
  - Torsional support
- Cantilevers
- Significance of C_b
  - AISC equations
  - Yura - downward loads
  - Yura - uplift
- Examples



Design of Structural Steel Flexural Members

(Non-composite)

- Limit states
  - Strength limit state (factored loads)
  - Serviceability limit states (service loads)
- Strength limit states
  - Local buckling (flange, web)
  - Lateral torsional buckling (C_b, l_b, E)
  - Yielding (F_y)
- Serviceability limit states (outside scope)
  - Deflection
  - Vibration
  - Refer to Design Guide 3: Serviceability Design Considerations for Steel Buildings, Second Edition/ West and Fisher (2003)

Strength Limit State for Local Buckling

- **Definitions**
  - λ = **Slenderness parameter**
    ; must be calculated for flange and web buckling
  - λ_P = **limiting slenderness parameter for compact element**
  - λ_R = **limiting slenderness parameter for non-compact element**

  - $\lambda \leq \lambda_P$ **: Section capable of developing fully plastic stress distribution**
  - $\lambda_p \leq \lambda \leq \lambda_r$ **: Section capable of developing yield stress before local buckling occurs; will buckle before fully plastic stress distribution can be achieved.**
  - $\lambda \geq \lambda_R$ **: Slender compression elements; will buckle elastically before yield stress is achieved.**

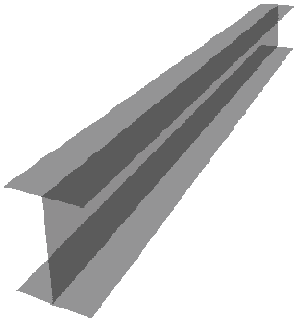
Strength Limit State for Local Buckling

- **Applies to major and minor axis bending**
- **Table B4.1 AISC LRFD Specifications**

Local Flange Buckling	**Local Web Buckling**
$\lambda = \frac{b_f}{2t_f}$ For I–Shapes	
$\lambda = \frac{b_f}{t_f}$ For Channels	$\lambda = \frac{h}{t_w}$
$\lambda_P = 0.38\sqrt{E/F_y}$	$\lambda_P = 3.76\sqrt{E/F_y}$
$\lambda_R = 1.0\sqrt{E/F_y}$	$\lambda_R = 5.70\sqrt{E/F_y}$

Note: λ_p limit assumes inelastic rotation capacity of 3.0. For structures in seismic design, a greater capacity may be required.

Local Compactness and Buckling



- **Local buckling of top flange of W6 X 15**
- **Top flange buckled before the lateral torsional buckling occurred or plastic moment is developed**
- **Many computer programs can find buckling load when section is properly meshed in a FEM (eigenvalue buckling analysis)**

Strength Limit States for Local Buckling

Example - Calculate $\phi_b M_n$ (Non compact section)

W6 X 15

F_y = 50.0 ksi

M_p = 10.8 x 50/12 = 45.0 k-ft

M_r = 0.7 x F_y x S_x

= 0.7 x 50 x 9.72/12 = 28.35 k-ft

Flange: $\lambda_P = 0.38\sqrt{29000 / 50} = 9.15$

$\lambda_R = 1.0\sqrt{29000 / 50} = 24.08$

Web: $\lambda_P = 3.76\sqrt{29000 / 50} = 90.31$

$\lambda_R = 5.70\sqrt{29000 / 50} = 137.27$

$\frac{b_f}{2t_f} = 11.50 > 9.15$ (Non−Compact Flange)

$\frac{h}{t_w} = 21.60 < 90.31$ (Compact Web)

$$\phi_b M_n = 0.9x\left[45-(45-28.35)\left[\frac{11.5-9.15}{24.08-9.15}\right]\right]$$

$$= 38.14\text{ k-ft } (94.18\%\text{ of } \phi_b M_p)$$

Strength Limit States for Local Buckling

$\lambda < \lambda_p$ **Section is compact**

$\phi_b M_n = \phi_b M_p = \phi ZxF_Y$ **Equation F 2-1**

$\lambda_p \le \lambda < \lambda_r$ **Section is non−compact**

$$\phi_b M_n = \phi_b \left[M_p - \left(M_p - 0.7F_y S_x \right) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \right] \le \phi_b M_p$$

Flange Equation F 3-1

$$\phi_b M_n = \phi_b \left[M_p - \left(M_p - F_y S_x \right) \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \right) \right] \le \phi_b M_p$$

Web Equation F4-9b

$\phi_b M_n$ is the smaller value computed using λ_p and λ_r corresponding to local flange and local web buckling

$\lambda > \lambda_r$ **Slender section, See detailed procedure Chapter F.**

Strength Limit States for Local Buckling

List of non-compact sections (W and C sections)

F_y	Beams	ϕM_n / ϕ M_p
50 ksi	W6 X 8.5	97.35 %
	W6 X 9	99.98 %
	W6x15	94.18 %
	W8 X 10	98.82 %
	W8 X 31	99.91 %
	W10 X 12	99.27 %
	W12 X 65	98.13 %
	W14 X 90	97.46 %
	W14 X 99	99.54 %
	W21 X 48	99.17 %
36 ksi	W6 X 15	98.51 %

- **Only one W-section is non-compact for F_y = 36 ksi**
- **Only ten W-sections are non-compact for F_y= 50 ksi**
- **All C-sections and MC- sections are compact for both F_y = 36 ksi and F_y = 50 ksi**
- **All WT sections are made from W-sections. When stem of WT is in tension due to flexure, this list also applies to WT made from W-sections**
- **When stem of WT is in compression due to flexure, all WT sections are non-compact or slender**

Limit States of Yielding and LTB

- LTB = Lateral-Torsional Buckling
- $\phi_b M_n$ is a function of:
  - Beam section properties (Z_x, r_y, x_1, x_2, S_x, G, J, A, C_w, I_y)
  - Unbraced length, L_b
  - L_p : Limiting L_b for full plastic bending capacity
  - L_r : Limiting L_b for inelastic LTB
  - F_y : Yield strength, ksi
  - E : Modulus of elasticity, ksi
- C_b: Bending coefficient depending on moment gradient
- Seminar considers only compact sections. See previous slide. (W6 X 15, 50 ksi outside scope)

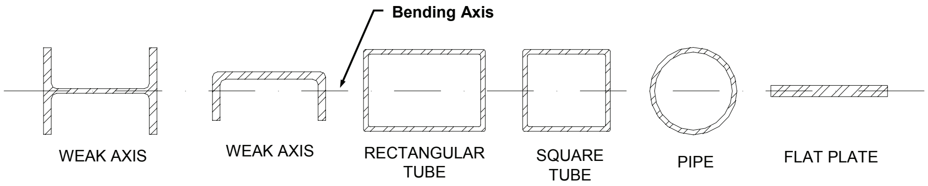
Strength Limit States for Local Buckling

Notes on non-compact sections

- Most W-sections and all C and MC-sections are compact for F_y = 50 ksi and F_y = 36 ksi
- With the exception of W6 X 15 (F_y = 50 ksi), the ratio of ϕM_n to ϕM_p for other non-compact sections is 97.35% or higher
- With the possible exception of W6 X 15 (F_y = 50 ksi), which is seldom used, local buckling effect on moment capacity may be safely ignored for all AISC standard W and C sections

Limit States of Yielding and LTB

- Lateral torsional buckling (LTB) cannot occur if the moment of inertia about the bending axis is equal to or less than the moment of inertia out of plane
- LTB is not applicable to W, C, and rectangular tube sections bent about the weak axis, and square tubes, circular pipes, or flat plates



No lateral - torsional buckling

- In these cases, yielding controls if the section is compact

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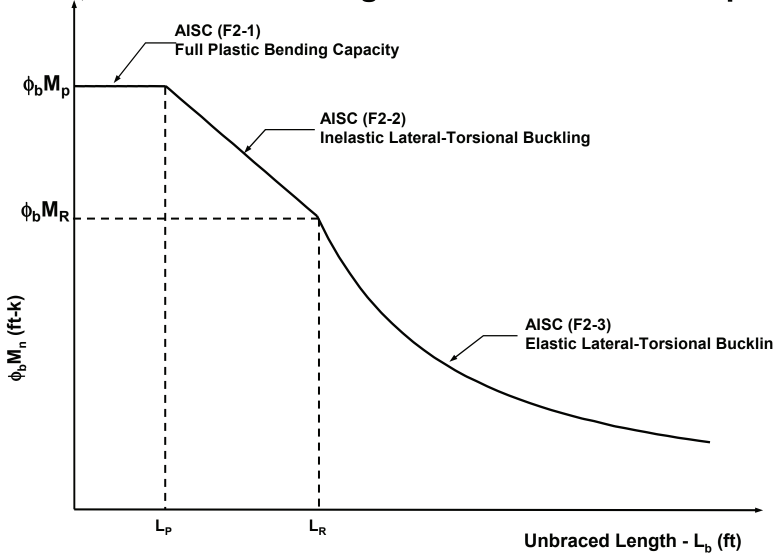
Limit States of Yielding and LTB

C_b = 1.0; uniform moment gradient between brace points



Limit States of Yielding and LTB

- **Definition of L_b (unbraced length)**

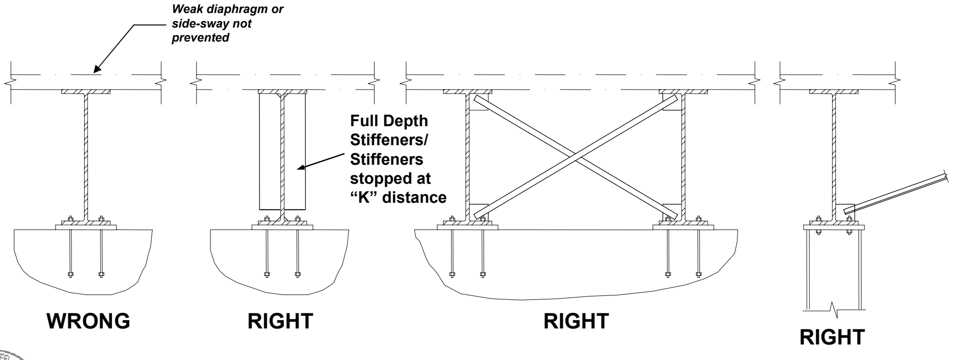
L_b = distance between points which are either braced against lateral displacement of compression flange <u>or</u> braced against twist of the cross section

*** Note that a point in which twist of the cross section is prevented is considered a brace point even if lateral displacement of the compression flange is permitted at that location**

Limit States of Yielding and LTB

Bracing at supports (scope of Chapter F)

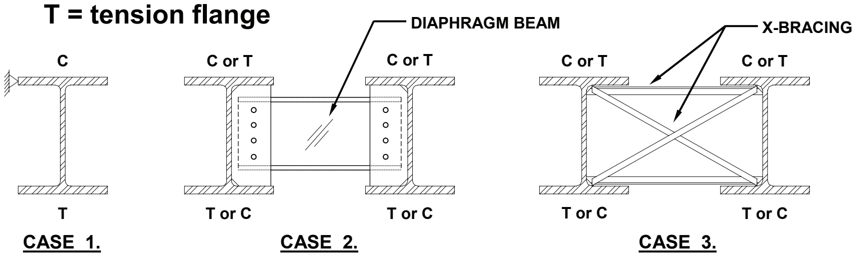
- **At points of support for beams, girders, and trusses, restraint against rotation about their longitudinal axis shall be provided.**



Limit States of Yielding and LTB

Effective beam bracing examples

- **Bracing is effective if it prevents twist of the cross section <u>and/or</u> lateral movement of compression flange**
- **C = compression flange**
  T = tension flange

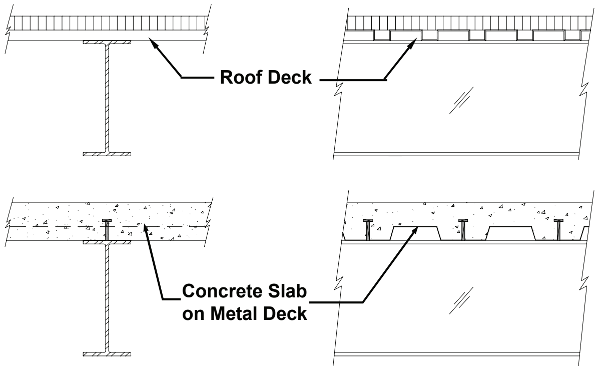


- **Case 1 is a brace point because lateral movement of compression flange is prevented**
- **Cases 2 and 3 are brace points because twist of the cross section is prevented.**
- **Stiffness and strength design of diaphragms and x-bracing type braces is outside of scope of this seminar**

Limit States of Yielding and LTB

Roof decks - slabs on metal deck

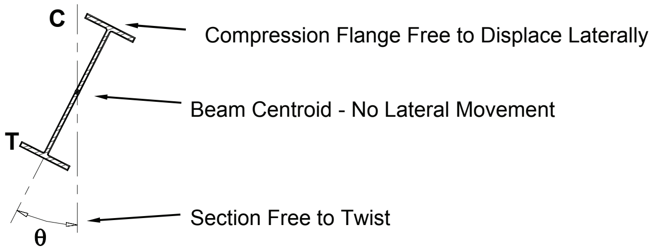
- **All concrete slabs-on-metal deck, and in most practical cases steel roof decks, can be considered to provide full bracing of compression flange**



Limit States of Yielding and LTB

- **FAQ = is the inflection point a brace point?**
- **Answer = no**
- **Reason = Twist of the cross section and/or lateral displacement of the compression flange is not prevented at inflection point location.**

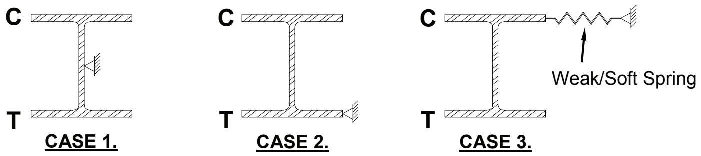
<u>Cross section buckled shape at inflection point:</u>



Limit States of Yielding and LTB

Ineffective beam bracing examples

- **Bracing is effective if it prevents twist of the cross section <u>and/or</u> lateral movement of the compression flange**
- **C = compression flange**
  T = tension flange



- **Cases 1 and 2 are not brace points because lateral movement of compression flange is not prevented and twist of cross-section is not prevented.**
- **Case 3 is not a brace point because "weak/soft spring" does not have enough stiffness or strength to adequately prevent translation of compression flange. Brace strength and stiffness requirements are outside the scope of this seminar.**

Limit States of Yielding and LTB

- **Compact sections**

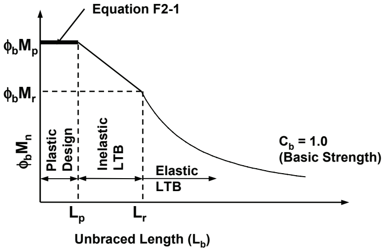
C_b = 1.0 (Uniform moment between braced points)

$L_b \le L_p$

$\phi_b M_n = \phi_b M_p = \phi_b Z F_y$ **: AISC (F2-1)**

$L_P = 1.76\, r_y \sqrt{E/F_y}$ **: AISC (F2-5)**

L_b = Unbraced Length



Limit States of Yielding and LTB

- **Compact sections: C_b = 1.0 (uniform moment between braced points)**

$L_p < L_b \le L_r$

$$\phi_b M_n = \phi_b C_b \left[M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le \phi_b M_p$$ **AISC (F2-2)**

- Linear interpolation between values of $\phi_b M_p$, $\phi_b(M_p\text{-}0.7F_yS_x)$, L_p,and L_r

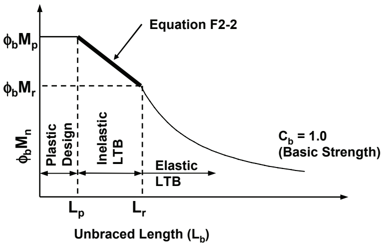
$$L_p = 1.76 r_y \sqrt{\frac{E}{Fy}}$$ **AISC F2-5**

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_x h_o}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\frac{S_x h_o}{Jc}\right)^2}}$$ **AISC F2-6**

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x}$$ **AISC F2-7**

For a doubly symmetric I-shape: c=1 **AISC F2-8a**

For a channel: $c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}}$ **AISC F2-8b**



Yielding and LTB

- **Compact and non-compact section**

  $C_b = 1.0$

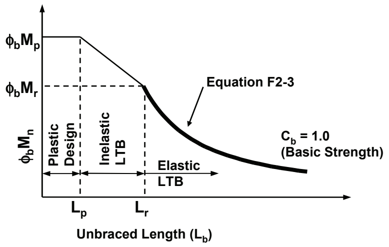
  $L_b > L_r$

$$\phi_b M_n = \phi_b F_{cr} S_x = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1+0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \le \phi_b M_P$$

AISC F2-3

Where: $r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x}$

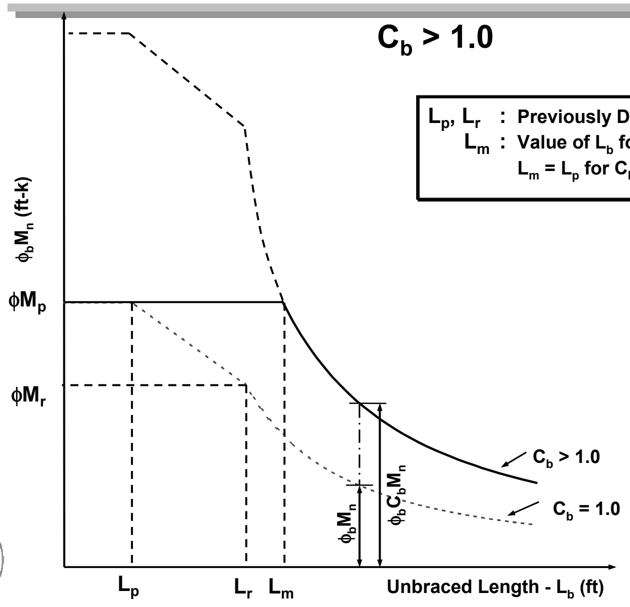
- **Elastic buckling**
- **$\phi_b M_n = \phi_b M_{cr}$ is not a function of F_y**



Limit States of Yielding and LTB

$C_b > 1.0$

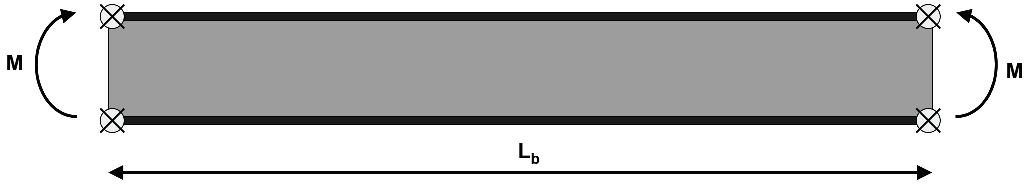
L_p, L_r : Previously Defined ($C_b = 1.0$)
L_m : Value of L_b for which $M_n = M_p$ for $C_b > 1.0$
$L_m = L_p$ for $C_b = 1.0$



Limit States of Yielding and LTB

Physical significance of C_b

- **AISC LRFD Eq. F2-4 is solution* to buckling differential equation of:**



- **Key Points:**
  - **Moment is uniform across L_b – constant compression in flange**
  - **Beam is braced for <u>twist</u> at ends of L_b**
  - **Basis is <u>elastic</u> buckling load**
  - **C_b = 1.0 for this case by definition**

See Timoshenko, *Thoery of Elastic Stability

Limit States of Yielding and LTB

Physical significance of C_b

- **What is C_b?**

C_b = (for a given beam size and L_b)

$$\frac{\text{[elastic buckling load of any arrangement of boundary conditions and loading diagram]}}{\text{[elastic buckling load of boundary conditions and loading shown on previous slide]}}$$

- Key points:
  - C_b is a linear multiplier
  - C_b is an “adjustment factor”
    - Simplifies calculation of LTB for various loading and boundary conditions
    - One equation (LRFD F2-3) can be used to calculate basic capacity
    - Published values of C_b represent pre-solved buckling solutions
    - All published C_b equations are approximations of actual D.E. solutions

Limit States of Yielding and LTB

Physical significance of C_b

- **You don't need a PhD to find C_b**
  - Differential equation solution with exact BC, loading (…ok, maybe need PhD)
  - Eigenvalue elastic buckling analysis

Example: W18x35 with L_b = 24' subject to uniform uplift load applied at top flange

AISC Eq. F2-3 (uniform moment): M_{cr} = 59.3 k-ft

Ends braced for twist, uniform moment (eigenvalue buckling analysis): M_{cr} = 59.1 k-ft

Ends braced for twist, top flange laterally braced, uplift load (eigenvalue buckling analysis): M_{cr} = 123.5 k-ft

C_b = 123.5 / 59.1 = 2.09

Limit States of Yielding and LTB

Notes on significance of C_b

- **For $C_b > 1.0$**
  - **$\phi_B M_n = C_b\ [\phi_b M_n \text{ for } C_b = 1.0\ ] \leq \phi_b M_p$**
  - **Moment capacity is directly proportional to C_b**
- **For $C_b = 2.0$**
  - **$\phi_B M_n(C_b = 2.0) = 2.0 \times \phi_b M_n(C_b = 1.0) \leq \phi_b M_p$**
- **Use of C_b in design can result in significant economies**

C_b Equations

- **AISC F1-1**
  - ◆ **Assumes beam top and bottom flanges are totally unsupported between brace points**
- **Yura**
  - ◆ **Downward loads**
  - ◆ **Top flange (compression flange) laterally supported**
- **Yura**
  - ◆ **Uplift loads**
  - ◆ **Top flange (tension flange) laterally supported**

Yielding and LTB

AISC equation

- **C_b is a factor to account for moment gradient between brace points**
- $$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} R_m \leq 3.0 \qquad \text{AISC F1-1}$$

M_{max} = maximum moment between brace points

M_A = at quarter point between brace points

M_B = at centerline between brace points

M_C = at three-quarter point between brace points

R_m = cross-section monosymmetry parameter

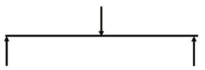
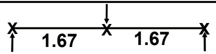
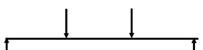
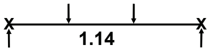
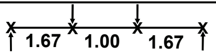
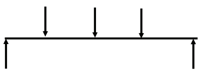
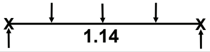
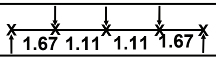
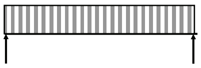
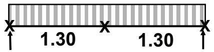
= 1.0, doubly symmetric members

= 1.0, singly symmetric members subjected to *single curvature* bending

- **M is the absolute value of a moment in the unbraced beam segment (between braced points which prevent translation of compression flange or twist of the cross section)**
- **This equation assumes that top and bottom flanges are totally unsupported between braced points**

AISC Table 3-1. Values of C_b

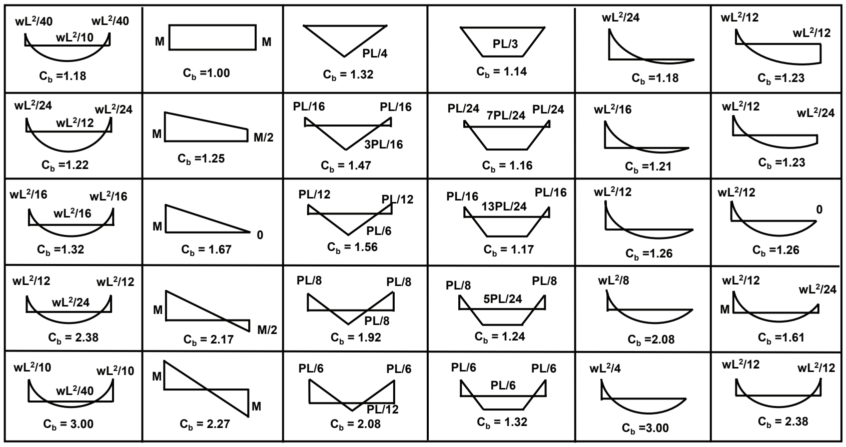
For simply supported beams

Load	Lateral Bracing Along Span	C_b	L_b
	None	 1.32	L
	At Load Points	 1.67 1.67	L/2
	None	 1.14	L
	At Load Points	 1.67 1.00 1.67	L/3
	None	 1.14	L
	At Load Points	 1.67 1.11 1.11 1.67	L/4
	None	 1.14	L
	At Centerline	 1.30 1.30	L/2

X = Brace Point.
Note That Beam Must Be Braced at Supports.

C_b Values for Different Load Cases

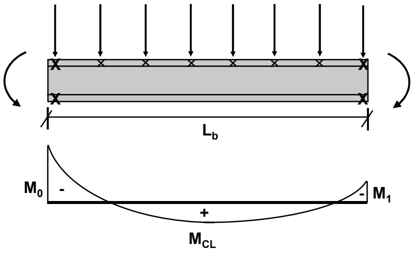
AISC Equation F1-1



$wL^2/40$ $wL^2/40$ $wL^2/10$ C_b =1.18	M M C_b =1.00	PL/4 C_b = 1.32	PL/3 C_b = 1.14	$wL^2/24$ C_b =1.18	$wL^2/12$ $wL^2/12$ C_b =1.23
$wL^2/24$ $wL^2/24$ $wL^2/12$ C_b =1.22	M M/2 C_b =1.25	PL/16 PL/16 3PL/16 C_b = 1.47	PL/24 7PL/24 PL/24 C_b = 1.16	$wL^2/16$ C_b =1.21	$wL^2/12$ $wL^2/24$ C_b =1.23
$wL^2/16$ $wL^2/16$ $wL^2/16$ C_b =1.32	M 0 C_b = 1.67	PL/12 PL/12 PL/6 C_b = 1.56	PL/16 13PL/24 PL/16 C_b = 1.17	$wL^2/12$ C_b =1.26	$wL^2/12$ 0 C_b =1.26
$wL^2/12$ $wL^2/12$ $wL^2/24$ C_b = 2.38	M M/2 C_b = 2.17	PL/8 PL/8 PL/8 C_b = 1.92	PL/8 5PL/24 PL/8 C_b = 1.24	$wL^2/8$ C_b =2.08	$wL^2/12$ $wL^2/24$ M C_b =1.61
$wL^2/10$ $wL^2/10$ $wL^2/40$ C_b = 3.00	M M C_b = 2.27	PL/6 PL/6 PL/12 C_b = 2.08	PL/6 PL/6 PL/6 C_b = 1.32	$wL^2/4$ C_b =3.00	$wL^2/12$ $wL^2/12$ C_b = 2.38

Yura's C_b Equation

(Compression flange continuously braced)



M_0 = End moment that gives the largest compression stress on the bottom flange

M_1 = Other end moment

M_{cl} = Moment at mid-span

$$C_b = 3.0 - \frac{2}{3}\left(\frac{M_1}{M_0}\right) - \frac{8}{3}\frac{M_{CL}}{(M_0 + M_1)^*}$$

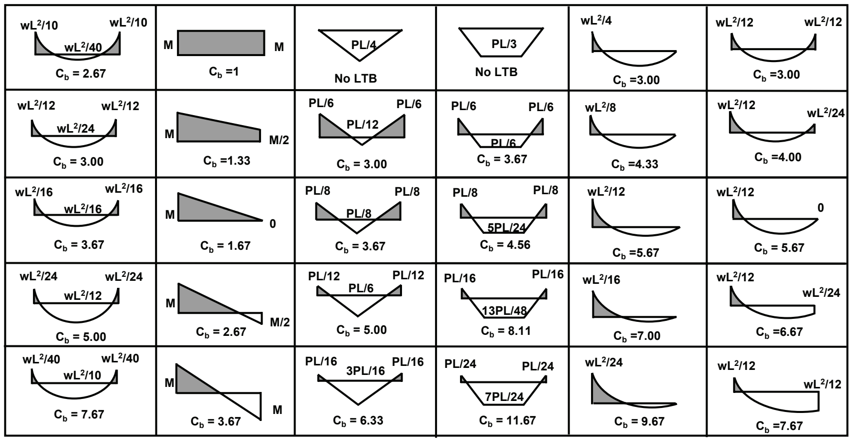
M_{max} = Max. of M_o, M_1, and M_{CL}

* Take $M_1 = 0$ in this term if M_1 is positive

1. If neither moment causes compression on the bottom flange there is no buckling
2. When one or both end moments cause compression on the bottom flanges use C_b with L_b
3. Use C_b with M_o to check buckling ($C_b \phi_b M_n > M_o$)
4. Use Mmax to check yielding (ϕ_bMp > Mmax)
5. X = Effective brace point

C_b Value for Different Load Cases

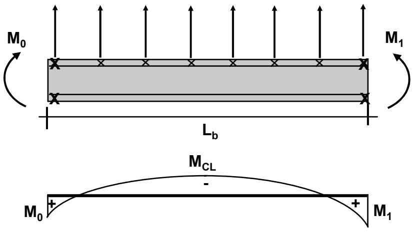
Yura's C_b Equation (compression flange continuously braced)



$wL^2/10$ $wL^2/40$ $wL^2/10$ $C_b = 2.67$	M M $C_b = 1$	PL/4 No LTB	PL/3 No LTB	$wL^2/4$ $C_b = 3.00$	$wL^2/12$ $wL^2/12$ $C_b = 3.00$
$wL^2/12$ $wL^2/24$ $wL^2/12$ $C_b = 3.00$	M M/2 $C_b = 1.33$	PL/6 PL/12 PL/6 $C_b = 3.00$	PL/6 PL/6 PL/6 $C_b = 3.67$	$wL^2/8$ $C_b = 4.33$	$wL^2/12$ $wL^2/24$ $C_b = 4.00$
$wL^2/16$ $wL^2/16$ $wL^2/16$ $C_b = 3.67$	M 0 $C_b = 1.67$	PL/8 PL/8 PL/8 $C_b = 3.67$	PL/8 5PL/24 PL/8 $C_b = 4.56$	$wL^2/12$ $C_b = 5.67$	$wL^2/12$ 0 $C_b = 5.67$
$wL^2/24$ $wL^2/12$ $wL^2/24$ $C_b = 5.00$	M M/2 $C_b = 2.67$	PL/12 PL/6 PL/12 $C_b = 5.00$	PL/16 13PL/48 PL/16 $C_b = 8.11$	$wL^2/16$ $C_b = 7.00$	$wL^2/12$ $wL^2/24$ $C_b = 6.67$
$wL^2/40$ $wL^2/10$ $wL^2/40$ $C_b = 7.67$	M M $C_b = 3.67$	PL/16 3PL/16 PL/16 $C_b = 6.33$	PL/24 7PL/24 PL/24 $C_b = 11.67$	$wL^2/24$ $C_b = 9.67$	$wL^2/12$ $wL^2/12$ $C_b = 7.67$

Yura's C_b Equation

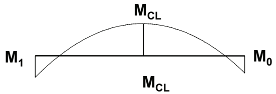
(Tension flange continuously braced)



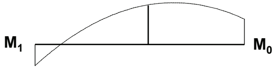
If the applied loading does not cause compression on the bottom flange, there is no buckling.

M_0 = End moment that produces the smallest tensile stress or the largest compression in the bottom flange.

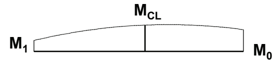
- <u>Three cases - three equations</u>
  - ◆ Case A - Both end moments are positive or zero:



  - ◆ Case B - M_0 is negative, M_1 is positive or zero:



  - ◆ Case C - Both end moments are negative:

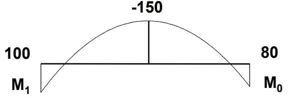


X = Effective brace point

Yura's C_b Equation (Uplift)

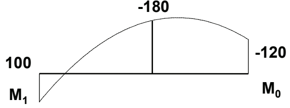
(Tension flange continuously braced)

- <u>Three cases</u>
  - ◆ Case A - Both end moments are positive or zero:



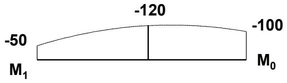
$$C_b = 2.0 - \frac{M_0 + 0.6M_1}{M_{CL}} \qquad C_b = 2.0 - \frac{(80 + 0.6x100)}{-150} = 2.93; \qquad C_bM_{CR} > 150$$

  - ◆ Case B - M_0 is negative, m_1 is positive or zero:



$$C_b = \frac{2M_1 - 2M_{CL} + 0.165M_0}{0.5M_1 - M_{CL}} \qquad C_b = \frac{2(100) - 2(-180) + 0.165(-120)}{0.5(100) - (-180)} = 2.35; \qquad C_bM_{CR} > 180$$

  - ◆ Case C - Both end moments are negative:

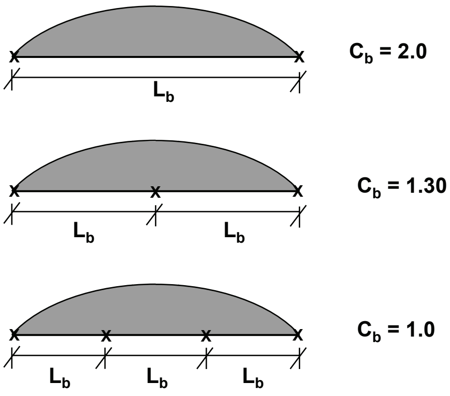


$$C_b = 2.0 - \frac{(M_0 + M_1)}{M_{CL}}\left[0.165 + \frac{1}{3}\frac{M_1}{M_0}\right] \qquad C_b = 2.0 - \frac{(-100 - 50)}{-120}\left[0.165 + \frac{1}{3}\left(\frac{-50}{-100}\right)\right] = 1.59;$$

$$C_bM_{CR} > 120$$

C_b Values (Uplift)

Yura's C_b Equation (compression flange continuously braced)



x = Bottom flange Brace Points (Compression flange displacement <u>or</u> twist prevented) at these points in addition to continuous tension flange bracing

---

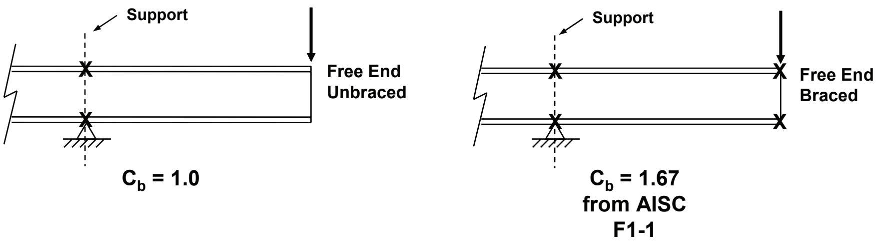
Limit States of Yielding and LTB

Cantilever beam design recommendations

- **AISC Spec. F1.2**

  For cantilevers or overhangs where the free end is unbraced, C_b = 1.0

- **Note that cantilever support shall always be braced.**



Cantilever Beams

Design recommendations

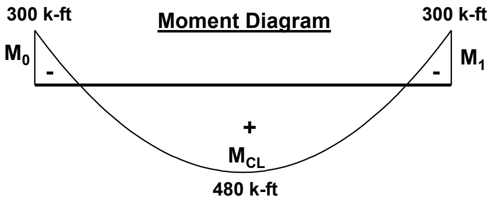
- Use AISC equation F1-1 with L_b = cantilever length and,
- Bracing method: (in order of preference)
  - Brace beam with full depth connection to cantilever at support and cantilever end
  - Or, full depth stiffeners and kicker at support and cantilever end
  - Or, full depth stiffeners at support and cantilever end

    (only if stiff flexural slab and if positive moment connection is provided between slab and beam. For example, slab on metal deck with shear connectors)

Example 1 - Beam Design

Downward load - top flange continuously braced

- W_u = 3.90 k/ft
- L = 40 ft; F_y = 50 ksi



M_o = -300 k-ft (causes compression on bottom flange)
M_1 = -300 k-ft (causes compression on bottom flange)
M_{CL} = +480 k-ft

$$C_b = 3.0 - \frac{2}{3}\left(\frac{-300}{-300}\right) - \frac{8}{3}\left(\frac{+480}{-300-300}\right)$$

Yura Equation

$$C_b = 4.47$$

Example 1 - Beam Design

Downward load - top flange continuously braced

- **Use C_b with M_O to check buckling ($C_b \phi_b M_n > M_O$)** **M_O = 300 k-ft**
- **Use M_{MAX} to check yielding ($\phi_b M_n > M_{MAX}$)** **M_{MAX} = 480 k-ft**

Try W18x50	**$\phi_b M_p$ = 378 k-ft < 480 k-ft**	**yielding NG**
	$C_b \phi_b M_n$ = 315 k-ft > 300 k-ft	**buckling OK**
Try W24x55	**$\phi_b M_p$ = 502 k-ft > 480 k-ft**	**yielding OK**
	$C_b \phi_b M_n$ = 270 k-ft < 300 k-ft	**buckling NG**
Try W21x57	**$\phi_b M_p$ = 483 k-ft > 480 k-ft**	**yielding OK**
	$C_b \phi_b M_n$ = 318 k-ft > 300 k-ft	**buckling OK**

Example 2 - Beam Design

Downward and uplift load - top flange continuously braced

- **Beam span = 38 ft** **DL = 16 psf** **WL = -30 psf**
- **Spacing = 10 ft** **LL = 50 psf**
- **Downward load**
  - **W_u = (1.2x16 + 1.6x50)(10) = 0.99 k/ft**
  - **$M_u = 0.99 \times 38^2/8$ = 179 k-ft**
  - **Beam compression flange fully braced by roof metal deck.**
- **Uplift**
  - **W_u = (-1.3x30 + 0.9x16)(10) = -0.25 k/ft**
  - **$M_u = -0.25 \times 38^2/8$ = -45 k-ft**

Example 2 - Beam Design (cont.)

Downward and uplift load - top flange continuously braced

- **For W16x31, 50 ksi,**

  $\phi_b M_p$ = 203 k-ft > 179 k-ft $\therefore$ OK

- **Uniform uplift load with top flange continuously braced:**

  C_b = 2.0

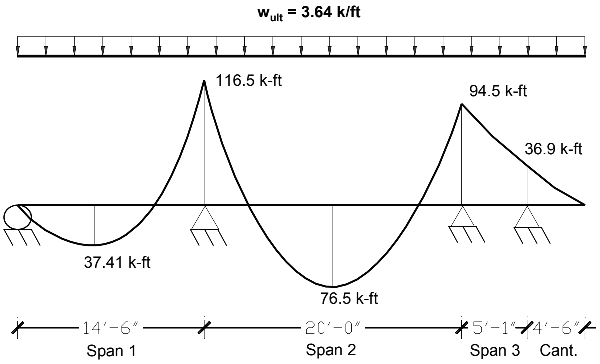
- **For W16x31, 50 ksi, L_b = 38 ft, and C_b = 2.0**

  $C_b \phi_b M_n$ = 48.7 k-ft > 45.0 k-ft OK
  No bracing of bottom flange necessary.

- **Note W16x40 required for C_b = 1.0**

Example 3 - Load Check

Beam Size: W18x35, F_y = 50ksi



(Top flange continuously braced)

Example 3 - Load Check

Use of Yura's C_b equation
(Top flange continuously braced and downward load)

Span	L_b	M_O	M_{CL}	M_1	C_b	Check Yielding:			Check Buckling:				Beam
					(Yura)	$\phi_b M_P$	M_{MAX}	$\phi_b M_P > M_{MAX}$	$\phi_b M_n$	$C_b \phi_b M_n$	$\lvert M_O \rvert$	$C_b \phi_b M_n > M_O$	Design
1	14.5	-116.5	+37.4	0	3.19	249.4	116.5	OK	122.8	249.4	116.5	OK	OK
2	20	-116.5	+76.5	-94.5	3.43	249.4	116.5	OK	69.8	239.4	116.5	OK	OK
3	5.08	-94.5	-53.9	-36.9	1.48	249.4	94.5	OK	242.0	249.4	94.5	OK	OK

- **Check Span 2 with $C_b = 1.0$**
  W18x35, $L_b = 20$ ft, $C_b = 1.0$, $\phi_b M_n = 69.8$ k-ft < 116.5 k-ft NG!
- **Beam is not adequate with $C_b = 1.0$**

Summary - Steel Flexural Members

- **Local buckling is not an issue for most beams for A992 beams**
- **Effective bracing must be provided at ends to restrain rotation about the longitudinal axis**
- **Effective brace reduces Lb and therefore economy; bracing is effective if it prevents twist of the cross section and/or lateral movement of the compression flange**
- **C_b is important in economical design when plastic moment is not be developed**

The Composite Beam Top 10: Decisions that Affect Composite Beam Sizes

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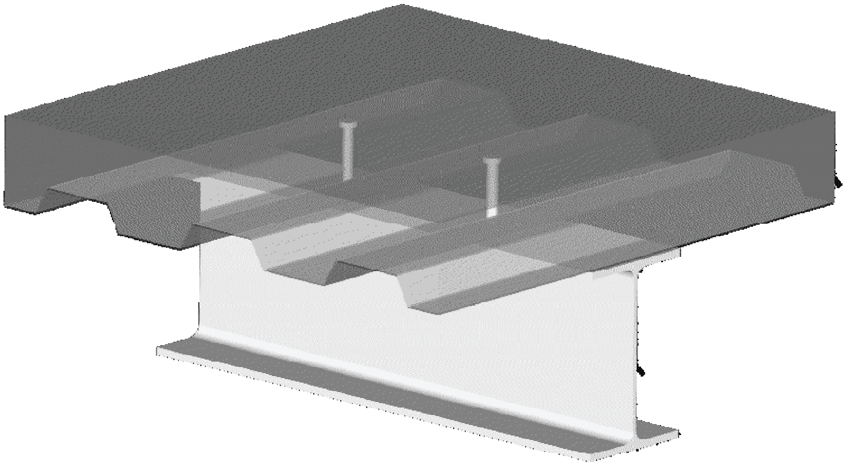
Composite Beam Design

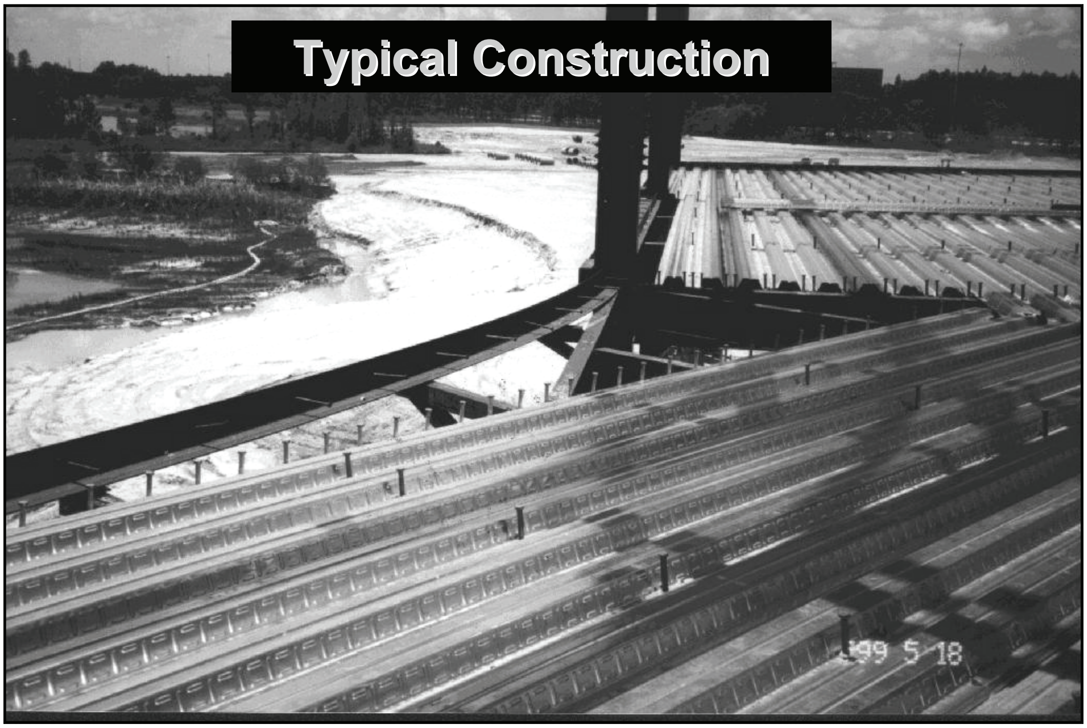
Overview:

1. **Materials considerations**
2. **Fire resistance issues**
3. **Deck and slab considerations**
4. **Strength design topics**
5. **Camber**
6. **Serviceability considerations**
7. **Design and detailing of studs**
8. **Reactions and connections**
9. **Composite beams in lateral load systems**
10. **Strengthening of existing composite beams**

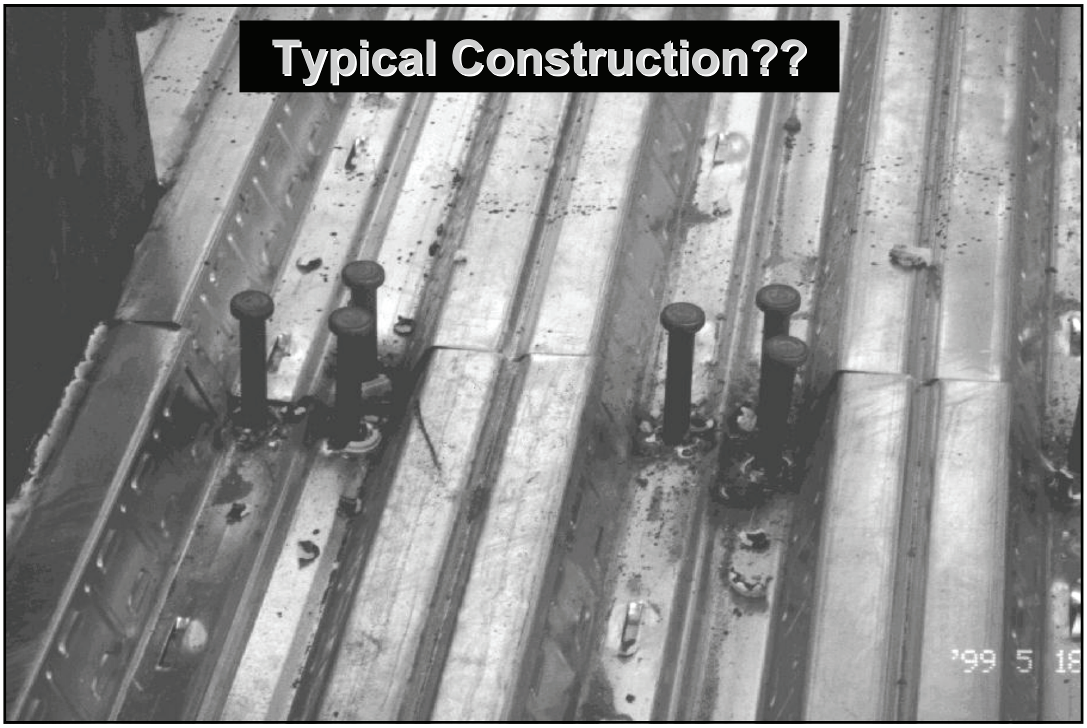
Typical Anatomy



Typical Construction



Typical Construction??



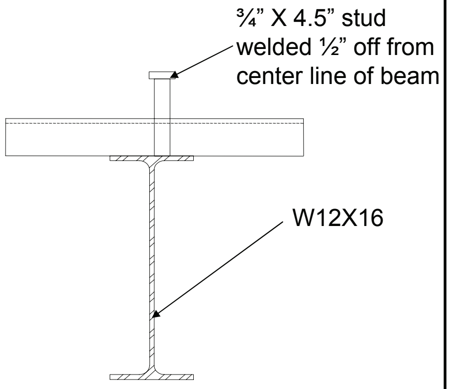
Materials Considerations

- Structural steel – per AISC specification section A3
  - A992 is most common material in use today for WF sections
  - AISC provisions apply also to HSS, pipes, built-up shapes
- Shear studs
  - Commonly specified as ASTM A108 (F_u = 60 ksi)
  - ¾” diameter studs typically used in building construction
- Slab reinforcing
  - Welded wire reinforcing
  - Reinforcing bars
  - Steel fiber reinforced (ASTM C1116)
    - For crack control only
    - Not all manufacturer’s steel fibers are UL rated (check with UL directory)

Materials – Beam Size

- Unless located over web, the diameter of stud shall be less than or equal to 2.5 times flange thickness
- It is very difficult to consistently weld studs right on center of the flange, therefore the minimum flange thickness should be equal to or more than 1/2.5 times stud diameter
- Stud welded on thin flange very likely will fail the hammer test
- Do not use beams with t_f < 0.3” when ¾” diameter studs are used

  W6X9 W6X12 W6X15
  W8X10 W8X13
  W10X12 W10X15
  W12X14 W12X16



Materials Considerations

- Slab concrete
  - Normal-weight concrete: 3 ksi < f'_c < 10 ksi
  - Light-weight concrete: 3 ksi < f'_c < 6 ksi
  - Higher strengths may be counted on for stiffness only
- Normal-weight concrete – 145 pcf including reinforcing
- Sand light-weight / Light-weight concrete
  - Consider wet weight of light-weight concrete
  - Actual weight varies regionally
  - 115 pcf recommended for long term weight and stiffness calculations
  - Beware of vendor catalogs that use 110 pcf

Materials Considerations

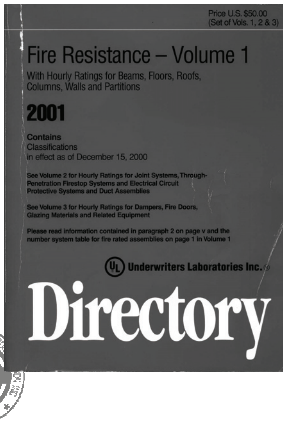
- Deck slab generally cracks over purlins or girders
  - Potential serviceability issue
  - Slab designed as simple span therefore strength of slab is not an issue
  - Potential crack is parallel to the beam and therefore is not a beam strength issue
- Deck serves as positive reinforcing
- Minimum temperature and shrinkage reinforcing is provided in slab
- W6X6-W1.4xW1.4 or W6X6-W2.1xW2.1 is generally provided
- Dynamic loads (parking garages, forklift traffic) can over time interfere with the mechanical bond between concrete and deck. Deck should not be used as reinforcing under such conditions.

Fire Resistance Issues

- Commercial construction typically Type I-A (Fire-Rated) per IBC
- 2-hour fire rating required for floor beams
- 3-hour fire rating required for "Structural Frame"
  - Columns
  - Lateral load system (bracing)
  - Horizontal framing with direct connection to columns
- Underwriters Laboratories Directory used to select fire rated assemblies
- Generally architect selects the design number and structural engineer satisfies the requirements of the UL design
- Check with Architect for fire rating requirements. Small buildings may not have 2 or 3 hour fire rating requirements
- Refer to AISC Design Guide 19 for detailed discussion

Fire Resistance Issues

- D916, D902 and D925 are typical UL construction
- According to ASTM E119, steel beams welded or bolted to framing members are considered "restrained"
- Visit http://www.ul.com/onlinetools.html for more information



Price U.S. $50.00
(Set of Vols. 1, 2 & 3)

Fire Resistance – Volume 1
With Hourly Ratings for Beams, Floors, Roofs, Columns, Walls and Partitions

2001

Contains
Classifications
in effect as of December 15, 2000

See Volume 2 for Hourly Ratings for Joint Systems, Through-Penetration Firestop Systems and Electrical Circuit Protective Systems and Duct Assemblies

See Volume 3 for Hourly Ratings for Dampers, Fire Doors, Glazing Materials and Related Equipment

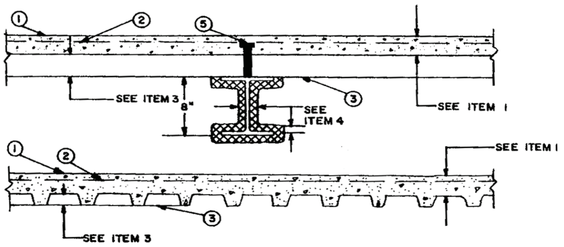
Please read information contained in paragraph 2 on page v and the number system table for fire rated assemblies on page 1 in Volume 1

Underwriters Laboratories Inc.

Directory

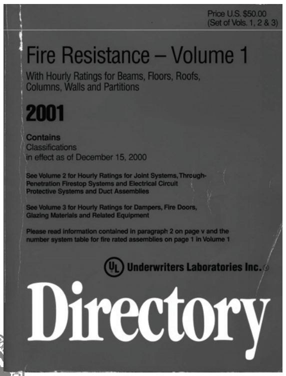
Design No. D916

Restrained Assembly Ratings—3/4, 1, 1-1/2, 2 or 3 Hr.
(See Items 1, 6, 7, 8 and 11)
Unrestrained Assembly Rating—0 Hr. (See Items 3, 4 and 4A)
Unrestrained Beam Ratings—1, 1-1/2, 2 and 3 Hr.
(See Items 4, 4A, 7 and 11)



Supports—W8x28 min size steel beams. Or steel joists or joist girders (not shown), composite or noncomposite. Welded or bolted to end supports.

Fire Resistance Issues



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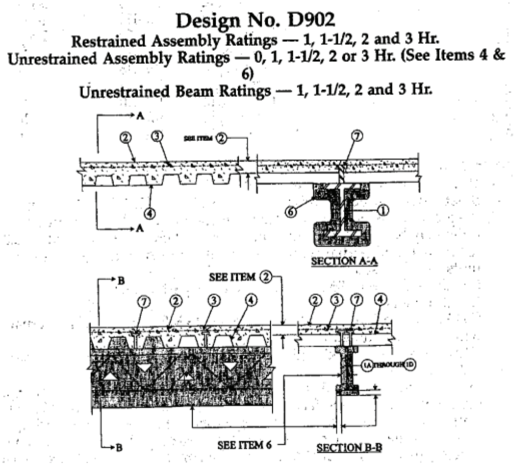
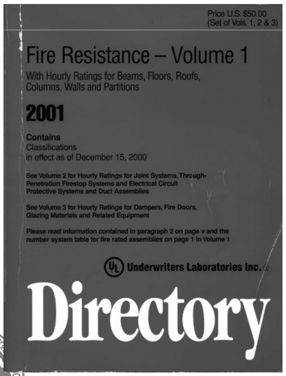
Directory



Design No. D902
Restrained Assembly Ratings — 1, 1-1/2, 2 and 3 Hr.
Unrestrained Assembly Ratings — 0, 1, 1-1/2, 2 or 3 Hr. (See Items 4 & 6)
Unrestrained Beam Ratings — 1, 1-1/2, 2 and 3 Hr.

Fire Resistance Issues



Price U.S. $50.00
(Set of Vols. 1, 2 & 3)

Fire Resistance – Volume 1
With Hourly Ratings for Beams, Floors, Roofs, Columns, Walls and Partitions

2001

Contains
Classifications
in effect as of December 15, 2000

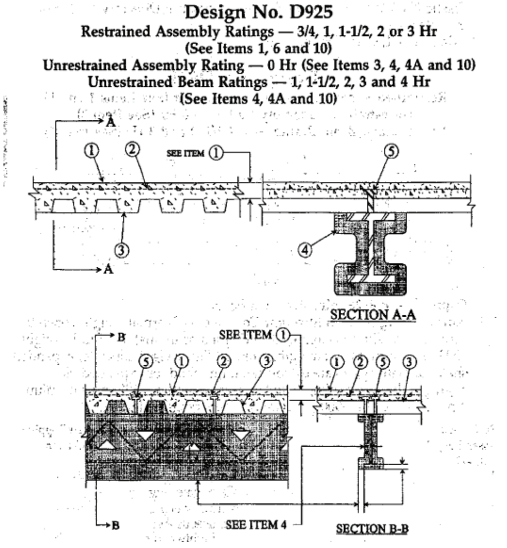
See Volume 2 for Hourly Ratings for Joint Systems, Through-Penetration Firestop Systems and Electrical Circuit Protective Systems and Duct Assemblies

See Volume 3 for Hourly Ratings for Dampers, Fire Doors, Glazing Materials and Related Equipment

Please read information contained in paragraph 2 on page v and the number system table for fire rated assemblies on page 1 in Volume 1

Underwriters Laboratories Inc.

Directory



Design No. D925
Restrained Assembly Ratings — 3/4, 1, 1-1/2, 2 or 3 Hr
(See Items 1, 6 and 10)
Unrestrained Assembly Rating — 0 Hr (See Items 3, 4, 4A and 10)
Unrestrained Beam Ratings — 1, 1-1/2, 2, 3 and 4 Hr
(See Items 4, 4A and 10)

Fire Resistance Issues

- Important aspects of D916
  - Universally used for composite floor construction
  - Minimum beam size is W8x28
  - Normal-weight concrete slab on metal deck
    - Minimum concrete strength = 3500 psi
    - Concrete thickness (over deck flutes) = 4 1/2” for 2-Hour Rating
  - Light-weight concrete
    - Minimum concrete strength = 3000 psi
    - Concrete thickness (over deck flutes) = 3 1/2” for 2-Hour Rating
    - Requires 4 to 7% air entrainment
  - Deck does not need to be sprayed
  - Beam fireproofing thickness = 1 1/16” for 2-Hour rating
  - Shear connector studs: optional
  - Weld spacing at supports: 12” o.c. for 12, 24, and 36” wide units
  - Weld spacing for adjacent units: 36” o.c. along side joints

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Fire Resistance Issues

- In some cases, engineer will need to provide a beam of a different size than that indicated in the UL design. Engineer may also need to use a channel instead of a W-section
- The UL Fire Resistance Directory states “Various size steel beams may be substituted … provided the beams are of the same shape and the thickness of spray applied fire resistive material is adjusted in accordance with the following equation:”

$$T_1 = \frac{\left(\frac{W_2}{D_2} + 0.6\right)}{\left(\frac{W_1}{D_1} + 0.6\right)} \times T_2$$

$$\frac{W}{D} \geq 0.37$$

$$T_1 \geq 3/8"$$

Note: this formula is the same as Eqn 7-17 in IBC 2000

- Where:
  - T = Thickness (in.) of spray applied material
  - W = Weight of beam (lb/ft)
  - D = Perimeter of protection (in.)
  - Subscript 1 = beam to be used
  - Subscript 2 = UL listed beam

Note: this is for information only EOR is not responsible for Specifying thickness of spray on Material thickness

Deck and Slab Selection

- **Slab thickness from fire rating:**

Deck Depth (in)	Fire Rating	Concrete Density	Minimum concrete thickness above flutes (in)	Stud Length (in)		
				Minimum	Recommended*	
				After Welding	Before Welding	After Welding
2	2-Hour	LWC	3.5	3 1/2	4 3/16	4
3			3.5	4 1/2	5 3/16	5
2		NWC	4.5	3 1/2	5 3/16	5
3			4.5	4 1/2	6 3/16	6
2	1-Hour	LWC	2.75	3 1/2	3 7/8	3 11/16
3			2.75	4 1/2	4 7/8	4 11/16
2		NWC	3.5	3 1/2	4 3/16	4
3			3.5	4 1/2	5 3/16	5

Notes:

- Studs must be 1½” (min.) above metal deck (LRFD I3.5a)
- For sand-lightweight concrete, 3½” thickness over flutes required for 2-hour fire rating
- Thinner slabs require sprayed on fire protection

SDI Maximum Spans

- Deck depth and gage chosen to set beam spacing
- Use 18 gage and thinner deck for greatest economy in deck system
- Greater deck span allows economy in steel framing (reduction in piece count, connections, etc.)

2-Hr Fire Rat-ing	*3.5” LWC Above Flutes*	Deck Gage						
	Deck Depth	22	21	20	19	18	17	16
	2	*8’-5”*	*9’-0”*	*9’-6”*	*10’-6”*	*11’-4”*	*12’-1”*	*12’-8”*
	3	*9’-1”*	*10’-9”*	*11’-7”*	*12’-10”*	*13’-8”*	*14’-7”*	*15’-3”*
	4.5” NWC Above Flutes	Deck Gage						
	Deck Depth	22	21	20	19	18	17	16
	2	*6’-2”*	*7’-6”*	*8’-1”*	*8’-11”*	*9’-8”*	*10’-3”*	*10’-9”*
	3	*6’-6”*	*7’-9”*	*8’-10”*	*10’-10”*	*11’-7”*	*12’-4”*	*13’-0”*
1-Hr Fire Rat-ing	*2.75” LWC Above Flutes*	Deck Gage						
	Deck Depth	22	21	20	19	18	17	16
	2	*8’-10”*	*9’-6”*	*10’-1”*	*11’-2”*	*12’-0”*	*12’-10”*	*13’-5”*
	3	*10’-1”*	*11’-7”*	*12’-3”*	*13’-6”*	*14’-5”*	*15’-4”*	*16’-1”*
	3.5” NWC Above Flutes	Deck Gage						
	Deck Depth	22	21	20	19	18	17	16
	2	*7’-2”*	*8’-2”*	*8’-8”*	*9’-7”*	*10’-4”*	*11’-0”*	*11’-7”*
	3	*7’-5”*	*8’-10”*	*10’-1”*	*11’-7”*	*12’-5”*	*13’-3”*	*13’-11”*

Deck and Slab Selection

- Base on 2 span condition
  - Watch out for single span conditions
- Concrete ponding
  - Implications of slab pour method
  - ½” equivalent added slab thickness for “flat” slab pour due to deck deflection
  - Consider adding a note on contract documents so that contractor considers additional concrete in a bid
- Consider wet weight of lightweight concrete
  - Field-reported wet weight can be as much as 125 pcf.
  - Spans from SDI’s “Composite Deck Design Handbook” allow wet unit weight to be as much as 130 pcf with a reasonable margin of safety for strength. Ponding of additional concrete may be an issue.
  - Vendor catalogs often show wet unit weight of lightweight concrete as 110 pcf.
    - If the catalog uses an ASD method, the maximum spans listed can be used with typical lightweight concrete specifications.
    - If the catalog uses an LRFD method, the maximum spans should be taken from the SDI publication with w = 115 pcf.

Strength Design

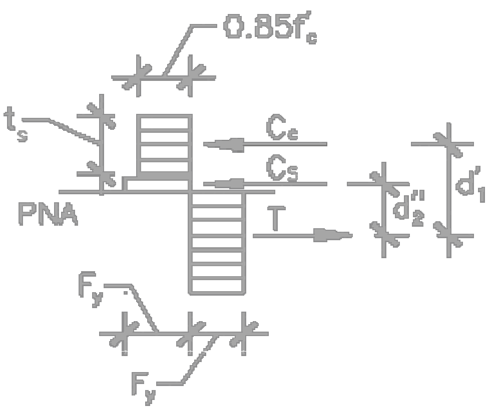
- 1999 LRFD
  - Moment capacity from plastic stress distribution
  - Loading sequence unimportant
  - Separate design for pre-composite condition
  - No minimum composite action (25% recommended)
  - Generally more economical than ASD method

Strength Design

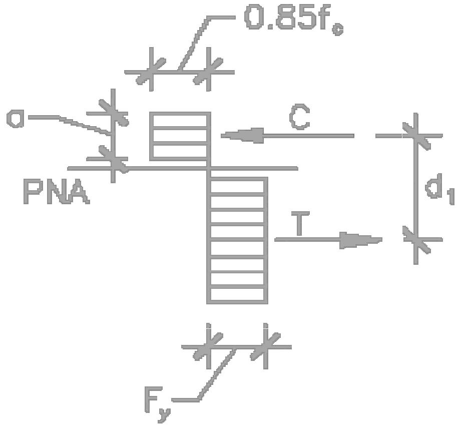
2005 Combined Specification:

- Compact web: $h / t_w \leq 3.76 (E / F_y)^{1/2}$
  - Design by Plastic Stress Distribution
- Non-compact web: $h / t_w > 3.76 (E / F_y)^{1/2}$
  - Design by superposition of elastic stresses
  - Primarily for built-up sections
- No minimum composite action (min 25% recommended)
- Maximum practical spacing 36” or 8 times slab thickness
- Design for strength during construction
- New provisions for stud capacity

Plastic Stress Distribution



PNA in Beam Flange



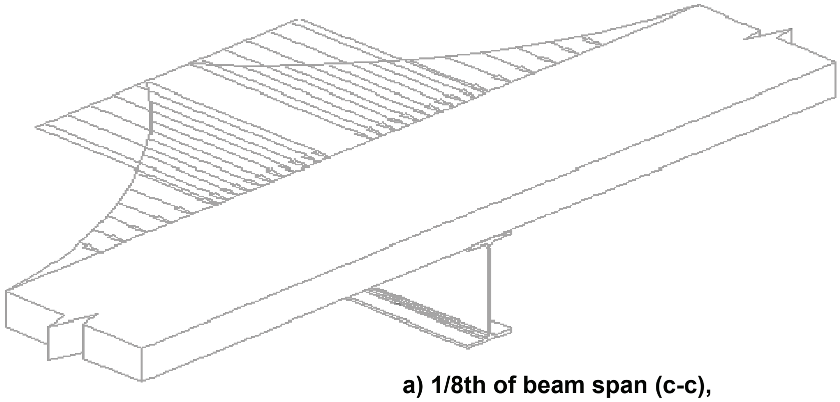
PNA in Slab

* Familiarity with detailed capacity calculations is assumed – not covered here

Definition of Percent Composite

- Economical design often achieved with less than full composite action in beam
- Provide just enough shear connectors to develop moment
- Percent Composite Connection (PCC):
  - Maximum Slab Force C_f is smaller of
    - $A_{sb} F_y$
    - $0.85 f'_c A_c$
  - ΣQ_n = sum of shear connector capacity provided between point of zero moment and point of maximum moment
  - PCC = $\Sigma Q_n / C_f$

Slab Effective Width



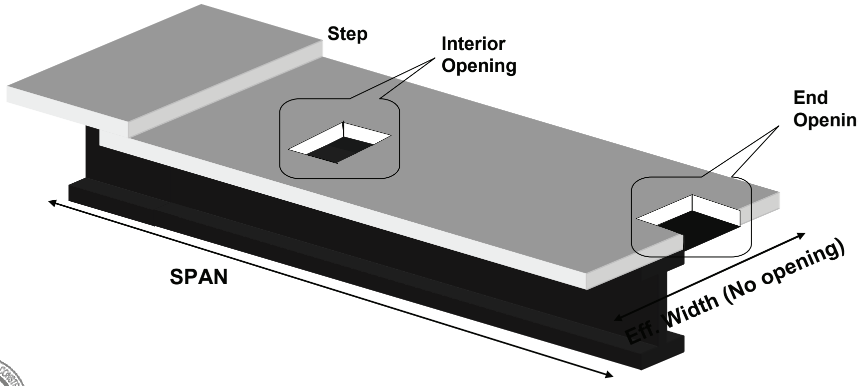
a) 1/8th of beam span (c-c),

b) Half the distance to CL of adj. beam, or

c) Distance to edge of slab

Slab Effective Width

- Watch out for slab openings, penetrations, steps



Strength During Construction

- Design as bare beam per Chapter F provisions
- Deck braces top flange only when perpendicular
- For girder unbraced length (Lb) during construction is equal to purlin spacing
  - provided purlin to girder connection is adequate to brace girder
- Capacity of braced beam is ϕM_p
  - Can beam yield under construction loads?

$$0.9\ F_y\ Z \approx 0.9\ F_y\ (1.1\ S) = F_y\ S$$

- Construction Loads
  - Use ASCE 37-02; Design loads on structures during construction
  - 20 psf minimum construction live load (1.6 load factor)
  - Treat concrete as dead load
    - 1.2 load factor in combination with live load
    - 1.4 load factor alone should account for any overpouring

The Do Not Camber List

1. Beams with moment connections
   - Difficult/expensive fit up with curved beam
2. Beams that are single or double cantilevers
   - Difficult/expensive for non-uniform or double curvature
3. Beams with welded cover plates
   - Heat from welding will alter camber
4. Non-prismatic beams
   - Cannot be cold bent with standard equipment
5. Beams that are part of inverted “V” (Chevron) bracing
   - Brace connection fit up problems
   - Influence of brace gusset plate on beam stiffness
6. Spandrel beams
7. Beams less than 20’ long
8. Very large and very small beams
   - Cold camber has limited capacity for large beams
   - Cold cambering may cripple small beams
   - Heat cambering is the alternative but very expensive

Camber

Methods to achieve floor levelness

1. Design for beam deflection
   - Design floor system for additional concrete that ponds due to beam deflection
2. Design for limiting stiffness
   - Floor is stiff enough to prevent significant accumulation of concrete
3. Design for shoring (Not recommended: $$$ and cracking)
4. Design for camber – preferred
5. Refer to AISC tool to evaluate whether camber is economical or not

   “Parametric Bay Studies” at www.aisc.org/steeltools

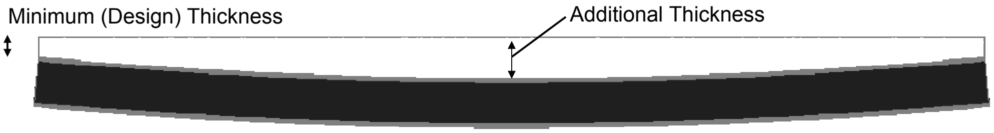
Camber

If beam is cambered, remember following:

- ◆ Method of pour
- ◆ Amount of camber
- ◆ Reasons for not over cambering
- ◆ Where to not camber?

Camber

- Camber should consider method of slab pour
- Slab pour options:



Pour to Constant Elevation

Pour to Constant Thickness

Camber Amount

- **Recommendation:**
  Camber beams for 80% of self-weight deflection
- **Loads to calculate camber**
  - **Self-weight only unless significant superimposed DL (Example: Heavy pavers)**
  - **If cambering for more than self-weight use constant thickness slab pour or design beam for greater stiffness**
- **AISC Code of Standard Practice 6.4.4**
  - **Beam < 50' – Camber tolerance - 0 / + ½"**
  - **Beam > 50' – Camber tolerance - 0 / + ½" + 1/8" (Length – 50')**
- **Beam simple end connections provide some restraint**

Camber Amount

<u>DO NOT OVER CAMBER!!!</u>

over cambered beam, flat slab pour

If beam is over cambered and slab poured flat, slab thickness is reduced, therefore:

- **Reduced fire rating**
- **Reduced stud / reinforcement cover**
- **Badly overcambered – protruding studs**
- **Reduced effective depth for strength**
- **Problems for cladding attachment**

Camber Amount

Under cambered beam, flat slab pour

- Minimum thickness at ends
- Additional thickness at middle
- Additional capacity typically accounts for additional concrete weight
- If camber is reasonably close additional concrete may be neglected

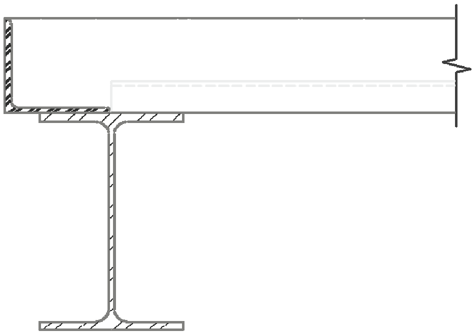
---

Recommended Camber Criteria

- Camber for 80% of construction dead load
  - Connection restraint
  - Over cambering of beam by fabricator
- Minimum camber
  - ¾” is a reasonable minimum
  - Design beam for additional stiffness below ¾” camber
- Camber increment
  - Provide camber in ¼” increments
  - Always round down
  - When using computer programs verify method
- Rule of thumb – L/300 is reasonable camber
- Maximum camber
  - Cambers > L/180 should be investigated further
  - LL deflection and vibration criteria likely to control
- Refer to AISC design guide 5; Low and Medium Rise Steel Buildings

Camber – Additional Stiffness

- Spandrel Beams
  - Edge angles or cladding frames can significantly increase beam stiffness
    Example:
    A L6x4x1/8 bent plate will increase the stiffness of a W18x35 by 25%
  - Include edge angles in moment of inertia calculations when cambering spandrels
  - Consider no spandrel camber and design for stiffness



Serviceability Considerations

- Serviceability considerations for composite floors:
  - Long-term deflections due to superimposed dead load
  - Short-term deflections due to live load
  - Vibration
    - Beyond the scope of this seminar
    - Extensively covered elsewhere – see AISC Design Guide 11
  - Performance of slab system
    - Cracking (if exposed)
    - Vapor emissions for adhered floor coverings
    - Shrinkage and temperature changes on large floor areas

Calculation of Deflections

- **Loads**
  - **Always evaluate at service load levels**
  - **Consider ASCE 7-02, Appendix B load combinations**
  - **Live load level**
    - **Design LL (reduced as applicable)**
    - **Realistic LL**
    - **Intermediate: 50% LL**
- **Section properties**
  - **Transform slab to equivalent steel section**
  - **Fully composite versus partially composite sections**
  - **Long-term loading**
- **Structural system**
  - **Consider continuity where influence is significant**
  - **Typical framing with simple connections not taken as continuous**

Calculation of Deflections

Method 1: Transformed Section Method (AISC Ch. I Commentary)

- **Fully composite (short-term loading):**

  $I_{eff} = 0.75\ I_{tr}$

- **Partially composite (short-term loading):**

  $I_{eff} = 0.75\ [\ I_s + [\ (\ \Sigma Q_n\ /\ C_f\ )\ ]^{1/2}\ (\ I_{tr} - I_s\ )\ ]$

- **Long term loading:**
  - **No established method**
  - **Simplified design procedure:**
    Calculate I_{tr} with $E_{effective} = 0.5 \text{ X } E_c = 0.5\ w_c^{1.5}\ (\ f_c'\ )^{1/2}$
  - **Critical cases should be evaluated with exact concrete properties**

Calculation of Deflections

Method 2: Lower Bound Moment of Inertia (AISC Ch. I Commentary)

- **Lower Bound Moment of Inertia**
  - **Fully composite**
  - **Partial composite**
- **Not applicable for long term loading**

$$I_{lb} = I_{eff} = I_x + A_s (Y_{ENA} - d_3)^2 + (\Sigma Q_n / F_y)(2d_3 + d_1 - Y_{ENA})^2$$

where:

I_x = moment of inertia of steel shape alone

A_s = area of steel shape alone

Y_{ENA} = distance from bottom of beam to ENA

d_1 = distance from the centroid of the slab compression force to the top of the steel section.

d_3 = distance from the centroid of the beam alone to the top of the steel section.

Calculation of Deflections

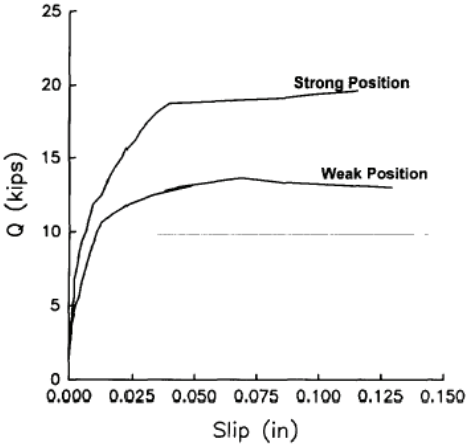
- **Total Beam deflection calculation:**
  - **Δ_1 = pre-composite deflection of steel beam alone**
  - **Δ_2 = deflection of composite section due to superimposed dead loads and realistic long-term live loads using long term I_{eff}**
  - **Δ_3 = deflection of composite section due to transient live load using short-term I_{eff}**
  - **Total Beam Deflection $\Delta_{tot} = \Delta_1 + \Delta_2 + \Delta_3$ - Camber**

Deflection Limits

- Typical floor deflection limits:
  - Under total load, Δ_{tot} < L/240
  - Under live load, Δ_3 < L/360 (< 1 1/2” for lease space)
- Beams supporting CMU or brick wall or cladding
  - Evaluate limits of cladding material (see AISC Design Guide 3)
  - Recommended limit at spandrels supporting cladding:
    - Under live, cladding and 50% SDL, Δ < 3/8”
    - Limit state for sizing exterior wall joints – not required for interior walls
  - Under live, cladding/wall and 66% SDL, Δ < L/600

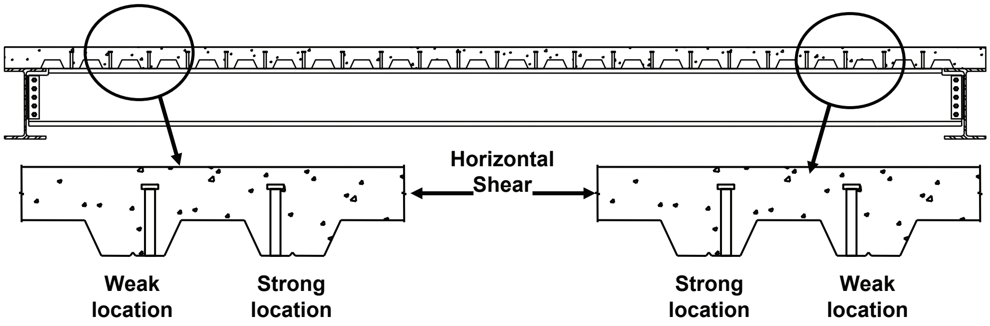
Shear Connectors

- Capacity for stud shear connectors is affected by several parameters
  - Position of Stud
  - Deck direction and number of studs in a row
- Note that when single stud is placed in a rib oriented perpendicular to the beam, 2001 LRFD imposes a minimum reduction factor of 0.75 to the nominal strength of a stud
- In addition to applying the required cap of 0.75 on the reduction factor for a single stud in rib perpendicular to the beam, 2001 LRFD recommends to avoid situations where all studs are in weak positions
- Additional reduction factors for position, number of studs in a row and deck directions are given in 2005 AISC specifications



Shear Connectors

- What is weak and strong position of studs?
- Reference: Easterling, Gibbings and Murray



Shear Connectors

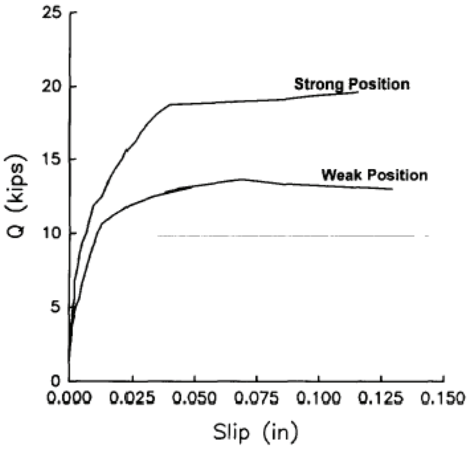
- New capacity for stud shear connectors in 2005 spec:

$Q_n = 0.5 A_{sc} (f_c' E_c)^{1/2} \le R_g R_p A_{sc} F_u$

R_g = stud geometry adjustment factor

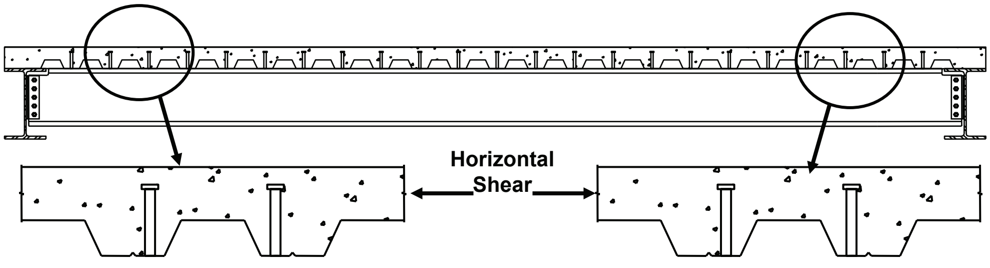
R_p = stud position factor

Note: $R_g = R_p = 1.0$ in 1999 LRFD code with adjustments for deck type



Shear Connectors

- Weak and strong position of studs
- Reference: Easterling, Gibbings and Murray paper in EJ



Weak location $R_p = 0.6$

Strong location $R_p = 0.75$

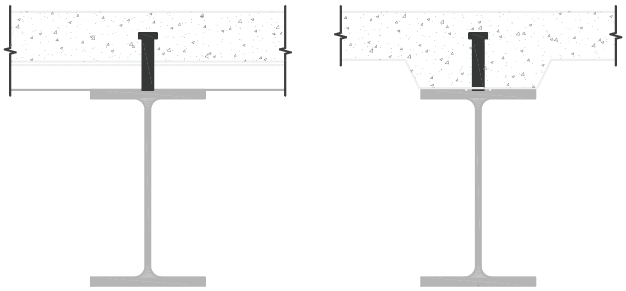
Strong location

Weak location

$R_p = 1.0$ for stud welded directly to steel (no deck)

Shear Connectors

$R_g = 1.0$



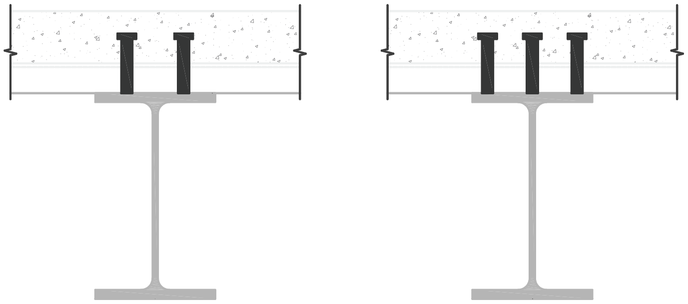
Single Stud in Rib Deck Perpendicular to Beam

Deck Parallel to Beam (typical rib widths)

Shear Connectors

Deck perpendicular to beam

$R_g = 0.85$ $R_g = 0.7$



2 studs per rib

3 or more studs per rib

Shear Connectors

Q_n Values (kips) – 2005 AISC Specification

f'c (ksi)		Girders (deck parallel)	Beams (deck perpendicular)					
			Studs per Rib - Strong			Studs per Rib - Weak		
			1	2	3	1	2	3
NWC	3	21.0	19.9	16.9	13.9	15.9	13.5	11.1
	3.5	23.6	19.9	16.9	13.9	15.9	13.5	11.1
	4	26.1	19.9	16.9	13.9	15.9	13.5	11.1
	4.5	26.5	19.9	16.9	13.9	15.9	13.5	11.1
	5	26.5	19.9	16.9	13.9	15.9	13.5	11.1
LWC	3	17.7	17.7	16.9	13.9	15.9	13.5	11.1
	3.5	19.8	19.8	16.9	13.9	15.9	13.5	11.1
	4	21.9	19.9	16.9	13.9	15.9	13.5	11.1
	4.5	24.0	19.9	16.9	13.9	15.9	13.5	11.1
	5	25.9	19.9	16.9	13.9	15.9	13.5	11.1

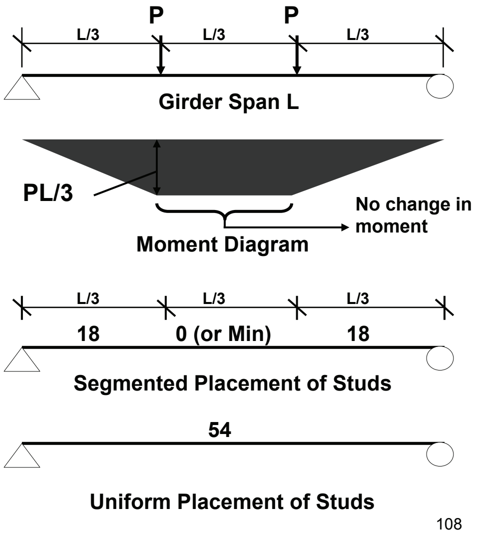
Shear Connectors

- New stud capacity a result of recent research
  - Capacity now consistent with other codes worldwide
  - Previous stud values provided lower factory of safety than implied by code
- Are all of my old designs in danger?
  - No failures or poor performance have been reported
  - Large change in shear stud strength does not result in a proportional decrease in flexural strength
  - Alternate shear transfer mechanisms not considered
  - Biggest effect is for beams with low PCC (<50%)

Shear Connector Placement

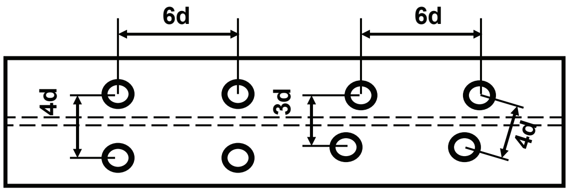
Uniform vs. Segmented Placement of Shear Connectors

- Most girders supports several point loads from purlins
- For zone with zero moment gradient, no shear connectors are needed
- Only minimum number of studs are generally provided in this zone
- For uniform layout, significant additional studs are needed
- Additional studs do not add any value (no increase in strength for the given load pattern)
- Most computer programs offer an option for segmented stud layout



Stud Criteria

- Stud Spacing Criteria (AISC Fig C-I3.6)



Plan view beam top flange

Note: d = diameter of stud

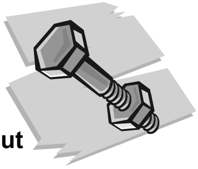
- 3 rows of studs maximum for strength calculations
  - 5½” minimum flange width for 2 rows of studs (for ¾” stud)
  - 8½” minimum flange width for 3 rows of studs (for ¾” stud)

Shear Connectors

- Should number of shear connectors be increased over calculated value?
  - Provides increased capacity (maybe) for future change in occupancy
  - Should increase when using computer programs with pre-2005 spec capacity routines
- Beams used as drag or collector elements
  - Studs required for drag forces should be added to studs required for composite beam action
  - Consider load path

Connections and Reactions

- Consider design method per AISC COSP
  - Delegate design to fabricator
  - EOR designed connections
- Delegated design is most common for simple connections
  - Must show reactions on drawings
  - Allows fabricator to use preferred connection types
- EOR Designed Connections
  - AISC developing design aids to simplify EOR design
  - Can be quicker method with early fabricator/erector input
  - Advantages in review time and avoiding disputes



Specifying Reactions

- Minimum strength
  - 10 kips factored LRFD
  - 6 kips service ASD
- Basis of Reaction
  - Uniform load method can be both excessive and unconservative
  - Provide actual reaction whenever possible
  - Specify ASD or LRFD design and show consistent reactions
  - Show axial loads (and through forces) for collector, drag, or brace beams

Specifying Reactions

- Should calculated reactions be increased?
- Pros
  - Tributary area is half tributary area of beam; live load reduction may differ
  - Allowance for future load increase due to occupancy change
  - Additional factor of safety and additional toughness
- Cons
  - May result in impractical or unbuildable connections
  - Adds cost to job that may not have value (consider client needs)
- If increasing, limit to a practical value (~15% max)

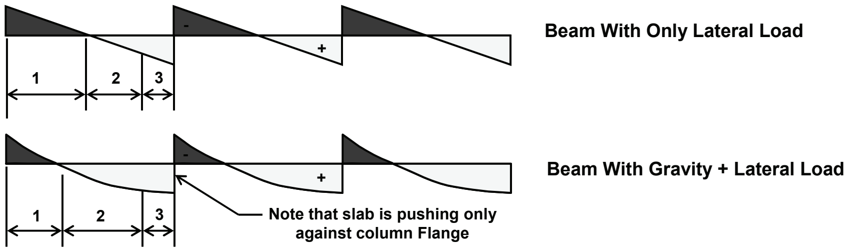
Beam Stiffness

- Composite beam stiffness for lateral analysis model can be ignored for simplicity (typical practice in engineering design office)
- Ignoring Composite action may be uneconomical for moment frames;
  - Steel moment frames should be avoided as much as possible (braced frames are very effective lateral load resisting system for steel buildings and should be used whenever architectural layout permits)

Or

- The stiffness of composite beams that are part of the lateral load-resisting frame can be based on the effective moment of inertia
- Effective moment inertia must reflect different moment of inertia in the positive moment regions (near the column & away from column) and negative moment region

Beam Stiffness



Beam With Only Lateral Load

Beam With Gravity + Lateral Load

Note that slab is pushing only against column Flange

Region 1: I_1 = Steel section moment of inertia I_{neg} ;L_1 = Conservatively = 0.5 L

Region 2: I_2 = Lower bound moment of inertia ; L_2 = Conservatively = 0.5 L – L_3

Q_n should reflect long term modulus of elasticity of concrete ($E_c/1+\beta_d$)

Region 3: I_3 = Effective moment of inertia with effective slab width equal to flange width of column; Conservatively take steel section moment of inertia

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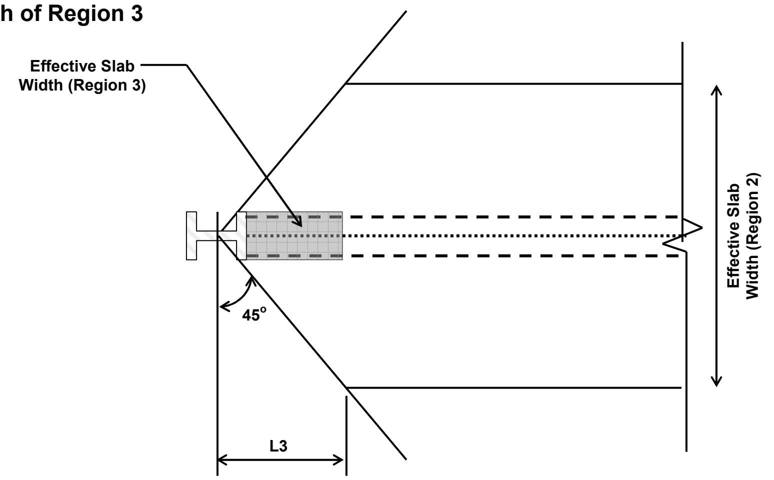
Beam Stiffness

Length of Region 3

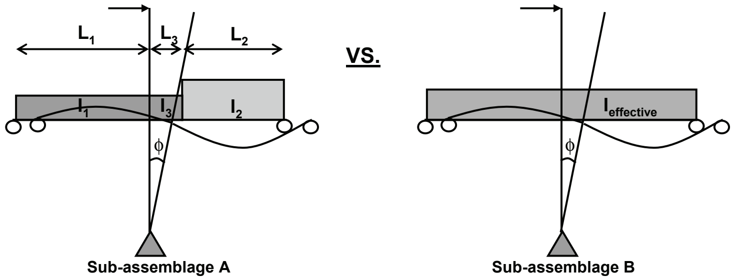


Reference: Multistory rigid frames with composite girders under gravity and lateral forces by: Robert J. Schaffhausen and Anton W. Wegmuller

Beam Stiffness

Model Beam as non prismatic with three segments (Not Practical)

Model Beam as prismatic with Ieffective



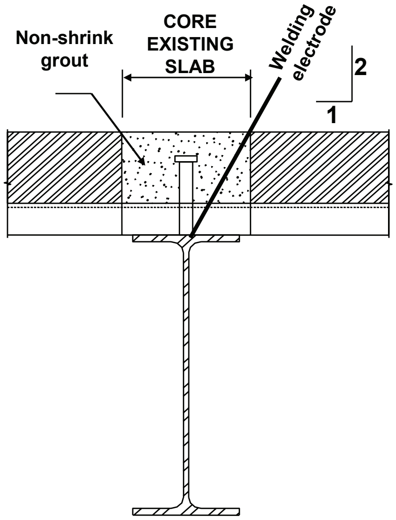
- **Objective is to Find I_{eff} such that rotation ϕ for a given load is same in both sub-assemblages**
- **10% to 30% increase in effective moment of inertia can be obtained over bare steel beam**
- **This will reduce 10 to 30% of drift component due to beam flexure**
- **Most engineers ignore the beneficial effect of composite action in beams of the moment frame**

Strengthening Existing Beams

- **Need for upgrade of the existing beams**
  - **Additional high density file loading**
  - **Change in occupancy**
- **Several options are available**
  - **Make non composite beams composite**
  - **Strengthen composite beams for flexural capacity**
  - **Strengthen composite beams that can work non-compositely**
- **Considerations for connections**

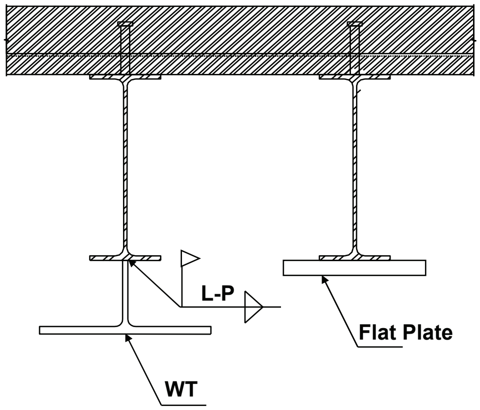
Strengthening Existing Beams

- Make non composite beams composite
- Existing beam must work under existing imposed services loads
- Composite action is used for ultimate design loads
- Requires work from floor above and therefore convenient
- Careful coordination required for drilling cores for shear connector installation
- Determine welding process
- Reference:
  - AISC EJ paper by David T Ricker



Strengthening Existing Beams

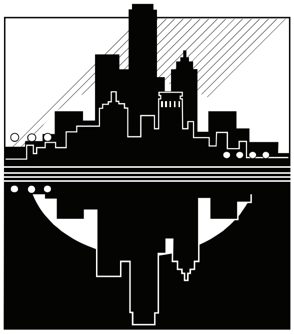
- Upgrade composite beams by welding bottom chord reinforcement (WT or Plate)
- Field welding is required between existing beam and reinforcement
- Due to length limitations, field splice also may be required for reinforcement
- Requires work from floor below (MEP interference)
- Strength gain limited by ΣQ_n (number of existing studs)
- Reinforcing must be developed at critical section
- Reference:
  - AISC EJ paper by John P. Miller



Strengthening Existing Beams

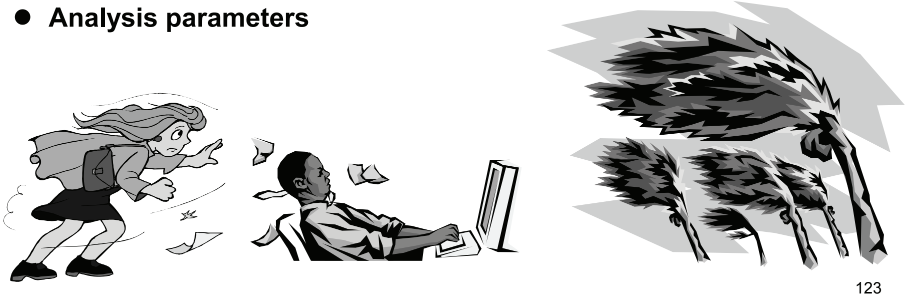
- In all upgrades critical strength limit may be the connection strength
- Upgrading of connection is rather labor intensive
- Strength of bearing bolted connections cannot be supplemented by field welding; Welding must be capable of resisting entire ultimate reaction
- Slip critical bolted connections can be upgraded by welding for deficiency only
- Consider changing bolts from A307 and A325 to A490
- Avoid developing excessive restraint at both ends

Lateral Analysis Of Steel Buildings For Wind Loads



Overview

- General issues
- Limit states
  - Serviceability
  - Strength
  - Stability - under factored gravity loads
- Analysis parameters



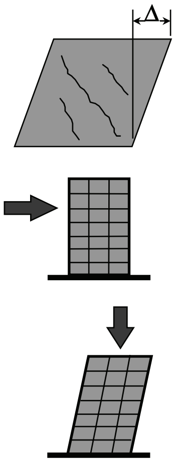
General Issues

- Identify the vertical lateral load resisting (VLLR) system
- Determine the extent of the model
- Gravity system may or may not be modeled with LLR system
- When gravity system is not modeled
  - Consider the loads on leaning columns
  - Consider gravity loads on LLR system properly
  - Consider diaphragm related forces in gravity system design
  - Wind loads on proper building geometry
- When gravity system is modeled
  - Use proper boundary conditions
  - Optimize gravity system before incorporating in lateral analysis model

General Issues

- Perform preliminary hand calculations before starting full analytical model
- Steel moment frame design is generally drift controlled
- When frames are drift controlled, perform optimization for optimum use of material
  - Many computer programs facilitate this process
  - Simple tools can be developed to find out the contribution of each drift component
- Check foundation loads (net uplift) under braced frame columns

Limit States



- Serviceability
  - Service loads
  - Damage to non–structural elements
  - Interstory drift
  - Perception to motion
- Strength-lateral loads
  - Factored loads (1.60 wind)
  - Ultimate strength design (LRFD)
  - P–Δ effects
- Stability—sustained gravity loads
  - Covered since 1999 LRFD
  - No lateral loads

Three Analyses Required

- Analysis I
  - Serviceability
- Analysis II
  - Strength design—gravity + lateral loads
- Analysis III
  - Stability—under factored gravity loads
  - Stability Index $Q \leq 0.375$: Interstory stability index $Q = P\Delta /vh$

Analysis Parameters

- Lateral load intensity and application
- Building mass (P–Δ effect)
- Member stiffness
- Beam-column joint modeling
- Diaphragm modeling and brace beam axial stiffness
- Foundation flexibility
- Acceptability criteria

Lateral Load Intensity

- **Serviceability**
  - **10 to 25 year wind**
  - **ASCE 7–02 commentary**
- **Strength**
  - **50 year wind (approximately) + Importance factor**
  - **Building code or wind tunnel**
- **Stability**
  - **Any lateral load**

Lateral Load Intensity

- **Serviceability**
  - **10 Year wind**

	V = 85-100 Mph	V > 100 Mph (Hurricane)	Alaska
Conversion factors for 10 year wind *velocity* (from 50 year)	**0.84**	**0.74**	**0.87**
Conversion factors for 10 year wind *loads* (from 50 year)	**0.71**	**0.55**	**0.76**

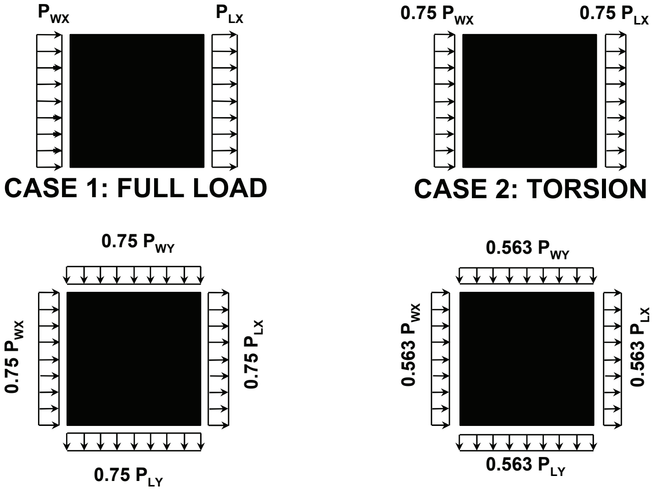
Notes:

1: Wind velocity is based on 3 second gust (ASCE 7-02)

2: Wind pressure conversion factors assume that pressure is proportional to the square of wind velocity. For tall flexible buildings, this may not be the case.

3: If wind tunnel study is performed, please check wind tunnel report for conversion factors.

Wind Load Application—ASCE 7-05



CASE 1: FULL LOAD

CASE 2: TORSION

$$M_T = 0.75(P_{WX} + P_{LX})B_X e_X$$
$$(e_X = \pm 0.15B_X)$$

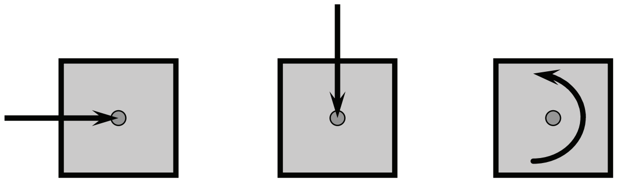
CASE 3: DIAGONAL

CASE 4: TORSION

$$M_T = 0.563\,(P_{WX} + P_{LX})B_X e_X$$
$$+\ 0.563\,(P_{WY} + P_{LY})B_Y e_Y$$
$$(e_X = \pm 0.15B_X,\ e_Y = \pm 0.15B_Y)$$

Lateral Load Application

Stability Under Factored Gravity Load



F_x = 100 Kip
Case 1

F_y = 100 Kip
Case 2

M_z = 100 Kip-in.
Case 3

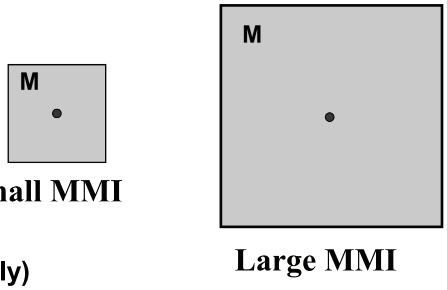
- May control lateral stiffness requirements. Especially for heavy buildings.
- Check torsional instability.
- Must be checked for all stories.

Torsional Stability

- Similar to translational stability
- Translational stability
    - Story mass
    - Story translational stiffness (kip/in.)
- Torsional stability
    - Story mass moment of inertia
    - Story torsional stiffness (kip-in./rad.)
    - May be a problem for heavy buildings with high mass moments of inertia in which lateral load resisting elements are located near the center of mass.

Mass Moment of Inertia—MMI

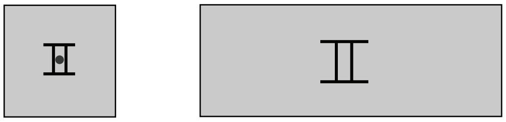
- Measure of mass distribution
- How far mass is from center of mass



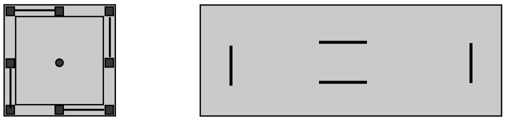
- MMI = M/A (Ix + Iy)
    - M = mass (w/g)
    - A = area
    - Ix = area moment of inertia about x–axis
    - Iy = area moment of inertia about y–axis

Torsional Stiffness

- Function of distance between center of stiffness and lateral load resisting elements



Poor torsional behavior

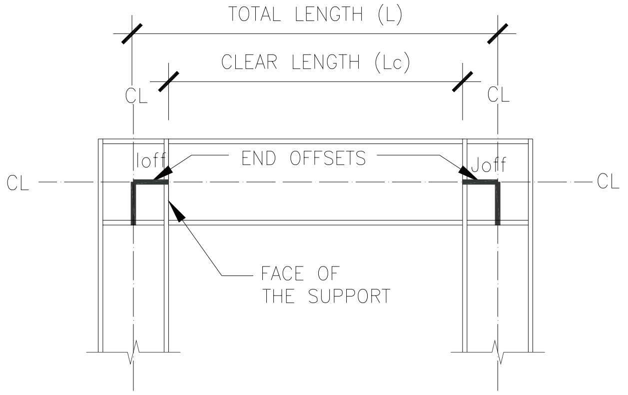


Good torsional behavior

Building Mass for P–Δ Purposes

- Analysis I — serviceability
  - Sustained building weight
  - Best estimate of actual weight (Self weight + portion of S.D.L and L.L.)
- Analysis II — strength
  - *1.2DL + 0.5LL*
  - Same weight as column design with wind loads
- Analysis III — stability
  - *1.2DL + 1.6LL*
  - Same weight as column design without wind loads

Beam-Column Joint Modeling



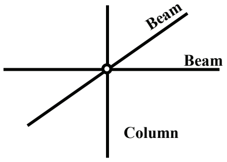
Flexible length = L_f = [L - rigid (Ioff + Joff)]
Rigid = 1 for fully rigid panel zone
Rigid = 0 for center line modeling

Beam-Column Joint Modeling

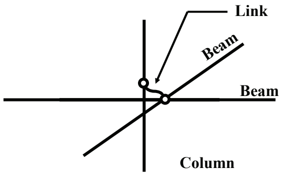
- **Most modern computer analysis program offers modeling of beam column joint**
- **Deformations in beam and column are allowed only outside of the rigid zone portion of beam column joint; i.e. clear length**
- **No flexural or shear deformations are captured within the rigid zone**
- **Studies have shown that significant shear deformations occur in panel zone area**
- **Using center line dimensions indirectly and approximately accounts for drift produced by deformations in the beam column joint**
- **Consider using center line modeling (no rigid zone) for steel moment frames**

Beam-Column Joint Modeling

- Advanced modeling option is available some commercial programs
- Panel zone can be assigned to beam-column joint
- Panel zone properties can be specified by user
- Complete control of properties is achieved by assigning link property to panel zone



Beam column connection



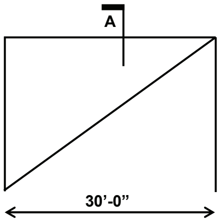
Panel zone representation

Design of Brace Beams

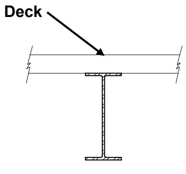
- Braced beams are subjected to flexure and axial loads
- When floor and roof diaphragm is modeled properly computer programs/post processors can be used to design these beams
- However, most commercial design programs do not check torsional buckling
- The following series of slides provide specific recommendations on appropriate values of KL_{major}, KL_{minor}, KL_z and L_b to be used for the design of brace beams
- KL_{major} is effective length of a compression member for buckling about strong axis
- KL_{minor} is effective length of a compression member for buckling about weak axis
- KL_z is effective length of a compression member for torsional buckling about longitudinal axis
- L_b is unbraced length of flexural member

Design of Brace Beams

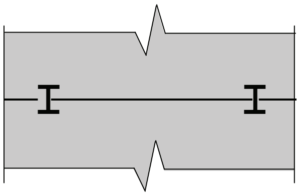
(Diagonal concentric brace)



Elevation



Detail A

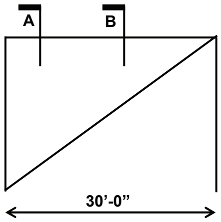


PLAN

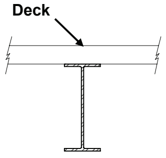
- Unbraced lengths for axial Loading
  - KL_{major} = 30'
  - KL_{minor} = 0'
  - KL_z = 30' (with lateral brace at distance = ½ beam depth + ½ deck depth)
  - By taking $KL_{minor} = KL_z$, torsional buckling need not be checked
- Unbraced lengths for bending
  - L_b = unbraced length = 0'
    (Net downward load)
  - L_b = unbraced length = 30'
    (Net upward load, C_b =2)

Design of Brace Beams

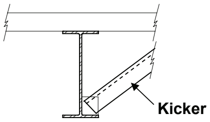
(Diagonal concentric brace with center kicker)



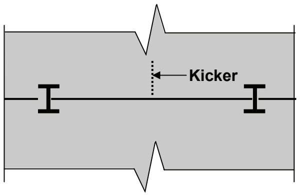
Elevation



Detail A



Detail B

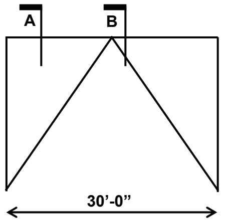


PLAN

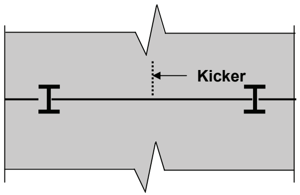
- Unbraced lengths for axial Loading
  - KL_{major} = 30'
  - KL_{minor} = 0'
  - KL_z = 15' (with lateral brace at distance = ½ beam depth + ½ deck depth)
  - By taking $KL_{minor} = KL_z$, torsional buckling need not be checked
- Unbraced lengths for bending
  - L_b = unbraced Length = 0'
    (Net downward load)
  - L_b = unbraced Length = 15'
    (With appropriate C_b for Net upward load.)
  - See previous slide for downward loads

Design of Brace Beams

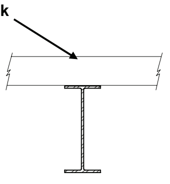
(Inverted “V” or chevron brace with center kicker)



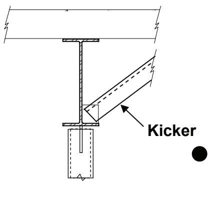
Elevation



PLAN



Detail A



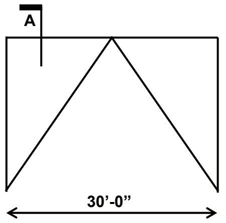
Detail B

- Unbraced lengths for axial Loading
  - KL_{major} = 15’
  - KL_{minor} = 0’
  - KL_z = 15’
    - Note by taking $KL_{minor} = KL_z$, torsional buckling need not be checked
  - It is recommended to always place a bottom flange brace (kicker) at mid span for Chevron or Inverted “V” brace
- Unbraced lengths for bending
  - L_b = unbraced length = 15’

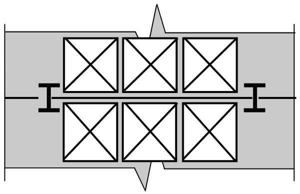
    (With appropriate C_b)

Design of Brace Beams

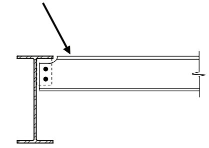
(Inverted “V” or chevron brace at openings)



Elevation



PLAN

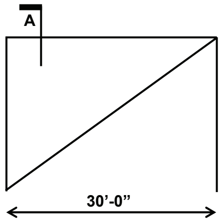


Detail A

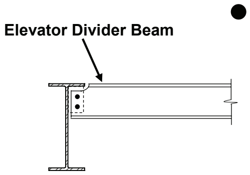
- Unbraced lengths for axial Loading
  - KL_{major} = 15’
  - KL_{minor} = 30’
  - KL_z = 30’
    - Note that possible bracing provided by elevator divider beam is not considered. Elevator divider beams are not fire proofed and therefore can not be considered as bracing member for primary load resisting elements
- Unbraced lengths for bending
  - L_b = unbraced length = 30’

Design of Brace Beams

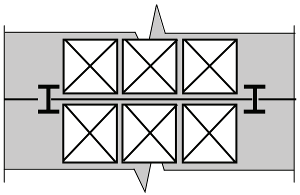
(Diagonal concentric brace at openings)



Elevation



Detail A



PLAN

- Unbraced lengths for axial Loading
  - KL_{major} = 30'
  - KL_{minor} = 30'
  - KL_z = 30'
    - Note that possible bracing provided by elevator divider beam is not considered. Elevator divider beams are not fire proofed and therefore can not be considered as bracing member for primary load resisting elements
- Unbraced lengths for bending
  - L_b = unbraced length = 30'

Foundation Modeling

- Determine the appropriate boundary condition at the base
- Simplified assumption (pinned base) may not help satisfy various limit states
- Investigate foundation flexibility and model accordingly
- For columns not rigidly connected to foundations, use G = 10
- For columns rigidly connected to foundations, use G = 1

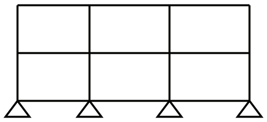
$$G = \frac{{Ic}/{Lc}}{\Sigma\left({Ig}/{Lg}\right)}$$

- Design base plates and anchor rods for proper forces

Column to Foundation Connection Flexibility

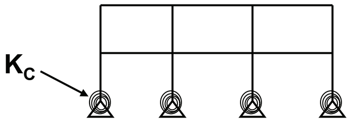
- Model beams connecting the pinned support such that

$$\Sigma\left({}^{Ig}\!/_{Lg}\right)=\frac{{}^{Ic}\!/_{Lc}}{10}$$



or, use rotational spring support with

$$K_c=\frac{0.6EIc}{Lc}$$



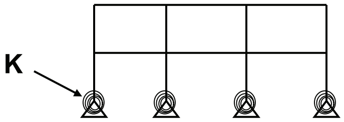
Foundation Flexibility

- Use PCI handbook as guide to determine foundation flexibility
- Function of footing size and modulus of sub-grade reaction

$$\frac{1}{K}=\frac{1}{K_C}+\frac{1}{K_f}$$

K_C = Column to foundation connection stiffness

K_f = Foundation stiffness

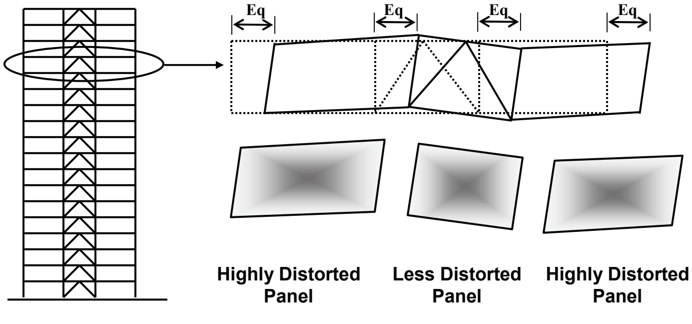


Acceptability Criteria

- **Analysis I — serviceability**
  - ◆ **Interstory drift**
  - ◆ **$\Delta/h \leq 0.0025$ at all levels and at all locations in plan**
    - – **Actual drift limit depends on cladding material**
  - ◆ **Torsion effects must be considered**
  - ◆ **Drift damage index may be appropriate in some tall slender bldg.**
- **Analysis II — Lateral strength (recommended)**
  - ◆ **Interstory stability index $Q = P\Delta/vh$, $P\Delta$ effect $= 1/(1-Q)$**
  - ◆ **$Q \leq 0.25$ — $P\Delta$ effect ≤ 1.33**
- **Analysis III — stability (derived from AISC LRFD Eq. A-6-2)**
  - ◆ **Interstory stability index $Q = P\Delta/vh$, $P\Delta$ effect $= 1/(1-Q)$**
  - ◆ **$Q \leq 0.375$ — $P\Delta$ effect ≤ 1.6**

Acceptability Criteria

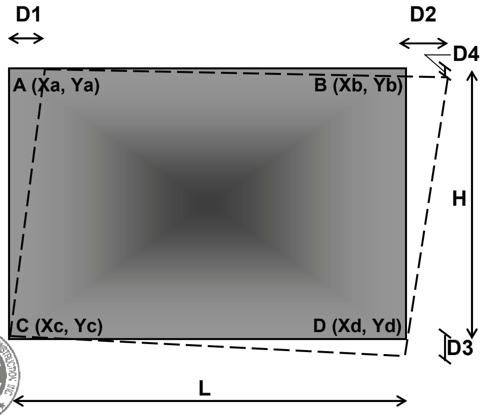
- **Drift Damage Index (DDI ≤ 0.0025)**
  - ◆ **May be appropriate in tall slender building**



˜Reference "serviceability under wind loads" by Lawrence G. Griffis (AISC Q1,1993)

Acceptability Criteria

- Drift Damage Index (DDI)
- Reference “serviceability under wind loads” by Lawrence G. Griffis (AISC Q1,1993)



D1 = (Xa –Xc)/H

D2 = (Xb –Xd)/H

} Will be same if no axial shortening of beam (also represents lateral drift)

D3 = (Yd –Yc)/L

D4 = (Yb –Ya)/L

} Will be non zero if there is axial shortening of column (traditionally ignored)

DDI = 0.5 (D1 + D2 + D3 + D4)

If D3 and D4 are ignored or zero *and*

when D1 = D2 (Rigid diaphragm)

DDI = lateral drift

Story Stability Index

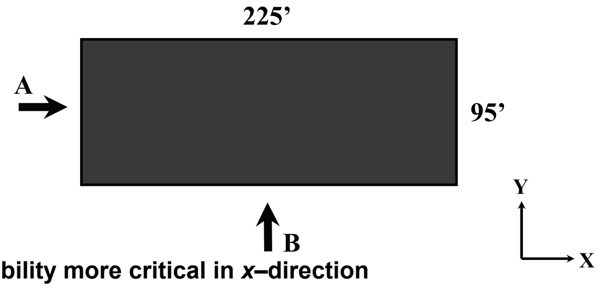
- Measure of the ratio of building weight above a story to the story stiffness
- *Q* = PΔ/VH = P/kh
  - K = V/Δ = story stiffness
  - P = building weight above the story in consideration
  - V = total story shear
  - Δ = story deflection due to story shear, V
  - H = story height
- Designing to satisfy interstory drift ratio does not insure stability

Story Stability

- Stability is more likely to be a problem for buildings located in moderate wind zones
- Buildings likely to have stability problems include:
  - Low lateral loads
  - Heavy (normal weight concrete topping, precast, masonry, etc.)
  - High first story height
  - Bad plan distribution of lateral load resisting elements (torsional stability)
  - Stability in long direction

Story Stability

- Example building



  - Stability more critical in *x*–direction
  - B = 225/95 = 2.37 A
  - Assume drift ratios are same in both *X*– and *y*–directions.
    - Then k_y = 225/95 = 2.37 k_x
  - Q = P/kh, $Q_x > Q_y$

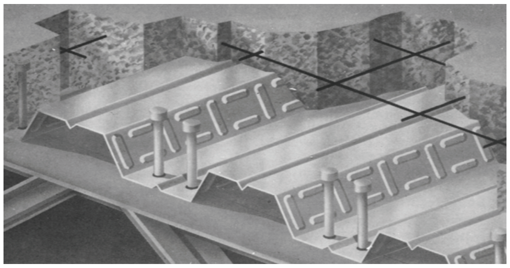
Steel Deck Design

Overview

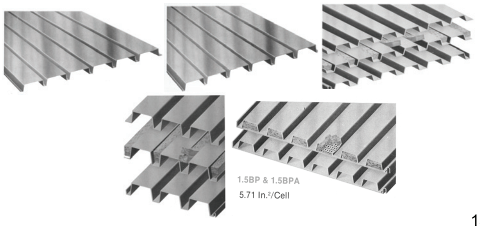
- Deck types
- Composite deck design
- Composite deck finishes and details
- Non-composite deck types
- Non composite deck design

Deck Types

- Composite deck



- Non-composite deck
  - Form for concrete
  - Support for roofing materials



Composite Deck

- Supports wet weight of concrete and also serves as a working platform
- Embossing on deck bonds the concrete to the deck and acts compositely, which serves as the positive moment reinforcing
- Bond under dynamic loading tend to deteriorate over a long time; therefore cannot be used as reinforcing under dynamic loading
- DO NOT USE for floors carrying fork-lift type traffic, dynamic loads, or parking garages. Use non-composite deck with positive and negative reinforcing bars instead
- Commonly used in office buildings, hospitals, public assembly
- Primarily used where uniform, static, loads with small concentrated loads are present

Composite Deck

- **Three standard depths available**
  - **1.5”, 2” and 3”**
- **6” rib spacing available in 1.5” deep deck**
- **12” rib spacing is standard**
- **Concrete depth over deck depends on strength requirement as well as fire resistance requirements**

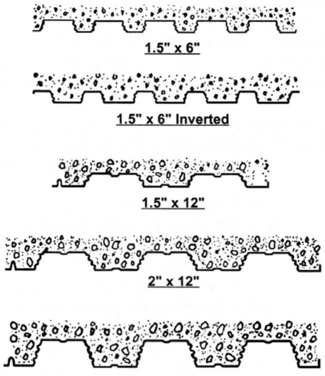
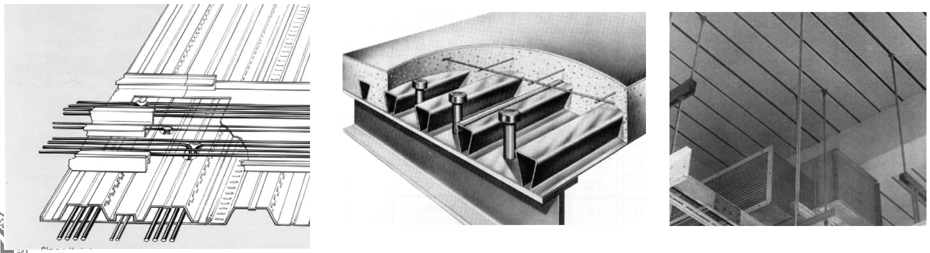


figure 4
COMPOSITE DECK PROFILES

Composite Deck

- **Non standard composite decks**
  - **Cellular (electrified floor) decks**
    - **Beware of strength reduction due to raceways perpendicular to the deck span that displaces concrete**
    - **Must be fireproofed with spray-on fireproofing**
  - **Slotted hanger deck**



Composite Deck Design

- **Required information**
  - Required usage and floor loading
  - Deck span / supporting beam spacing
  - Type of concrete (NW, SLW, LW)
  - Fire resistance rating
- **Design process**
  - Select slab thickness that does not require spray on fire proofing
  - Select deck depth and gage from load tables
  - Choose deck depth and gage so that no shoring is required for two or more span condition
  - Check load capacity of composite section (rarely controls)

Composite Deck Design

Deck Depth (in)	Fire Rating	Concrete Density	Minimum concrete thickness above flutes (in)	Stud Length (in)		
				Minimum	Recommended*	
				After Welding	Before Welding	After Welding
2	2-Hour	LWC	3.5	3 1/2	4 3/16	4
3			3.5	4 1/2	5 3/16	5
2		NWC	4.5	3 1/2	5 3/16	5
3			4.5	4 1/2	6 3/16	6
2	1-Hour	LWC	2.75	3 1/2	3 7/8	3 11/16
3			2.75	4 1/2	4 7/8	4 11/16
2		NWC	3.5	3 1/2	4 3/16	4
3			3.5	4 1/2	5 3/16	5

Composite Deck Design

Recommended deck clear span to satisfy SDI maximum unshored span criteria for 2 span condition:

2-Hr Fire Rat-ing	*3.5” LWC Above Flutes*	**Deck Gage**						
	Deck Depth	**22**	**21**	**20**	**19**	**18**	**17**	**16**
	2	*8’-5”*	*9’-0”*	*9’-6”*	*10’-6”*	*11’-4”*	*12’-1”*	*12’-8”*
	3	*9’-1”*	*10’-9”*	*11’-7”*	*12’-10”*	*13’-8”*	*14’-7”*	*15’-3”*
	4.5” NWC Above Flutes	**Deck Gage**						
	Deck Depth	**22**	**21**	**20**	**19**	**18**	**17**	**16**
	2	*6’-2”*	*7’-6”*	*8’-1”*	*8’-11”*	*9’-8”*	*10’-3”*	*10’-9”*
	3	*6’-6”*	*7’-9”*	*8’-10”*	*10’-10”*	*11’-7”*	*12’-4”*	*13’-0”*
1-Hr Fire Rat-ing	*2.75” LWC Above Flutes*	**Deck Gage**						
	Deck Depth	**22**	**21**	**20**	**19**	**18**	**17**	**16**
	2	*8’-10”*	*9’-6”*	*10’-1”*	*11’-2”*	*12’-0”*	*12’-10”*	*13’-5”*
	3	*10’-1”*	*11’-7”*	*12’-3”*	*13’-6”*	*14’-5”*	*15’-4”*	*16’-1”*
	3.5” NWC Above Flutes	**Deck Gage**						
	Deck Depth	**22**	**21**	**20**	**19**	**18**	**17**	**16**
	2	*7’-2”*	*8’-2”*	*8’-8”*	*9’-7”*	*10’-4”*	*11’-0”*	*11’-7”*
	3	*7’-5”*	*8’-10”*	*10’-1”*	*11’-7”*	*12’-5”*	*13’-3”*	*13’-11”*

*** Chart based on F_y = 40 ksi**

Basis of Composite Deck Design

- **SDI Composite Deck Design Handbook for post-composite design**
- **Design as simple span**
  - **Slab cracks over beam supports**
  - **For continuous design need to reinforce slab more than minimum**
- **Bending Capacity:**
  - **Elastic Design (most catalogs)**
    - **Elastic stress distribution based on transformed section**
    - **Limit based on allowable stresses**
  - **Plastic Design (typically noted as ‘LRFD Design’ in catalogs)**
    - **Plastic stress distribution (similar to reinforced concrete design)**
    - **Requires additional studs over supports to achieve full plastic capacity**
    - **Must provide additional studs (as noted in catalogs) to use higher capacities**

Composite Deck Design

- Other considerations
  - Provide minimum number of studs at end span to develop the required positive moment
  - Check bearing width and web crippling of deck during construction
  - Check shear strength of deck
  - Check for concentrated loads and verify design using SDI criteria
  - Provide reinforcing in the form of welded wire reinforcing or rebar

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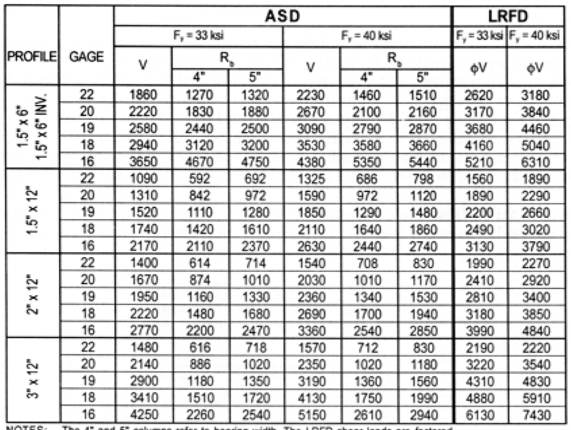
Composite Deck Design

- Shear and Web Crippling Capacity of Deck



PROFILE	GAGE	ASD						LRFD	
		F_y = 33 ksi			F_y = 40 ksi			F_y = 33 ksi	F_y = 40 ksi
		V	R_b		V	R_b		φV	φV
			4"	5"		4"	5"		
1.5" x 6" 1.5" x 6" INV.	22	1860	1270	1320	2230	1460	1510	2620	3180
	20	2220	1830	1880	2670	2100	2160	3170	3840
	19	2580	2440	2500	3090	2790	2870	3680	4460
	18	2940	3120	3200	3530	3580	3660	4160	5040
	16	3650	4670	4750	4380	5350	5440	5210	6310
1.5" x 12"	22	1090	592	692	1325	686	798	1560	1890
	20	1310	842	972	1590	972	1120	1890	2290
	19	1520	1110	1280	1850	1290	1480	2200	2660
	18	1740	1420	1610	2110	1640	1860	2490	3020
	16	2170	2110	2370	2630	2440	2740	3130	3790
2" x 12"	22	1400	614	714	1540	708	830	1990	2270
	20	1670	874	1010	2030	1010	1170	2410	2920
	19	1950	1160	1330	2360	1340	1530	2810	3400
	18	2220	1480	1680	2690	1700	1940	3180	3850
	16	2770	2200	2470	3360	2540	2850	3990	4840
3" x 12"	22	1480	616	718	1570	712	830	2190	2220
	20	2140	886	1020	2350	1020	1180	3220	3540
	19	2900	1180	1350	3190	1360	1560	4310	4830
	18	3410	1510	1720	4130	1750	1990	4880	5910
	16	4250	2260	2540	5150	2610	2940	6130	7430

NOTES: The 4" and 5" columns refer to bearing width. The LRFD shear loads are factored.
The R_b values have not been increased by 33%.

figure 9
SHEAR AND BEARING CAPACITIES
POUNDS (PER FOOT OF WIDTH)

Reference:
"SDI Composite Deck Design Handbook", March 1997

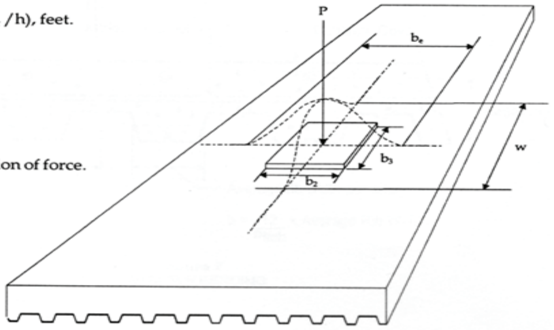
Composite Deck Design

- **Concentrated loads**

$b_m = b_2 + 2t_c + 2t_t$

single span bending: $b_e = b_m + 2(1 - x/l)x$; where x is the location of the load.
continuous span bending: $b_e = b_m + 4/3\,(1 - x/l)x$
shear: $b_e = b_m + (1 - x/l)x$
but in no case shall $b_e > 8.9(t_c/h)$, feet.
weak axis Moment $= \dfrac{Pb_e}{15w}$

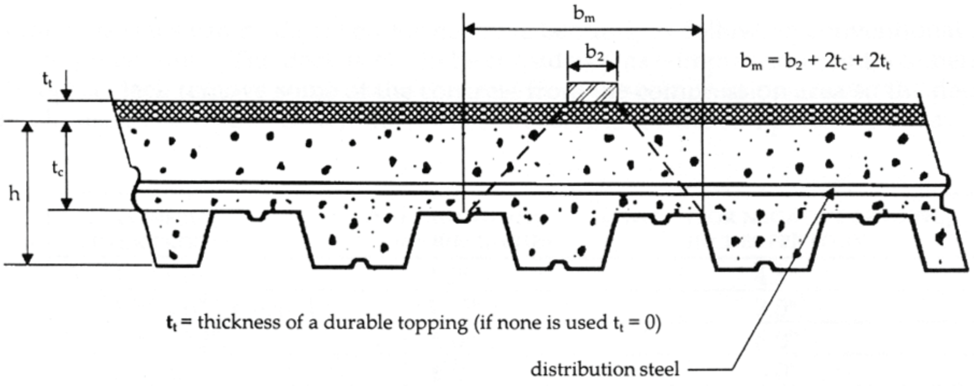
$w = \dfrac{l}{2} + b_3$; but not to exceed l



Curved lines
represent distributuion of force.

Composite Deck Design

- **Concentrated loads**



$b_m = b_2 + 2t_c + 2t_t$

t_t = thickness of a durable topping (if none is used $t_t = 0$)

Composite Deck Design

- Concentrated loads
  - Determine effective width of slab for bending
    - $b_e = b_m + C(1-x/l)x$;
    - C = 2.0 for simple span, 1.33 for continuous span
    - x = location of load along span
    - $b_m = b_2 + 2t_c + 2t_t$ (see previous slide)
    - t_c = thickness of concrete over deck
    - t_t = thickness of any topping
    - b_2 = width of concentrated load perpendicular to span
    - Resist moment by effective width of deck slab and provide reinforcing if needed

---

Composite Deck Design

- Concentrated loads
  - Determine effective width of slab for shear
    - $b_e = b_m + (1-x/l)x$
  - Limit on b_e; no greater than 8.9 (t_c/h)
  - Determine weak direction bending to calculate distribution steel over width $W = L/2 + b_3 < L$
  - Weak direction bending $M = P \times b_e \times 12/(15 \times w)$ in-lbs per ft. Provide reinforcing over deck to resist the moment

Composite Deck

- G90 galvanized finish - roofs and extreme environments such as a swimming pool
- G60 galvanized finish – use minimum recommended finish
- Painted finish - inform the owner of the risk involved
- Deck serves as positive bending reinforcing, it must be designed to last the life of the structure

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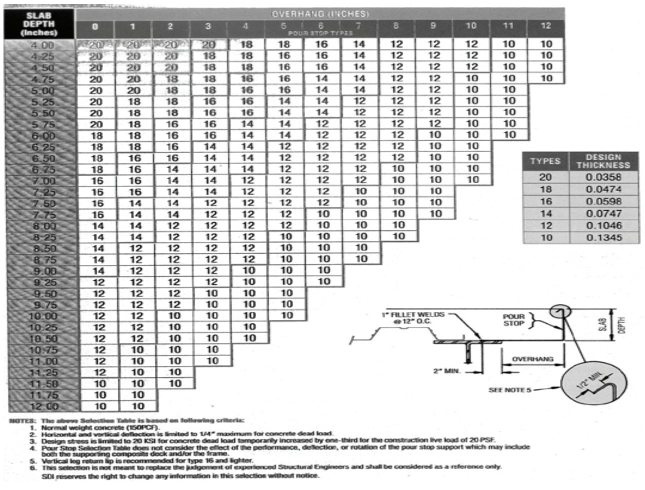
Composite Deck Details

- Slab edge pour closures
- Use bent plates where welding is expected

Ref: "Steel Deck Inst.
Design Manual for Composite Decks, Form Decks, Roof Decks, and Cellular Deck Floor Systems with Electrical Distribution"



SLAB DEPTH (Inches)	OVERHANG (INCHES)												
	0	1	2	3	4	5	6	7	8	9	10	11	12
						POUR STOP TYPES							
4.00	20	20	20	20	18	18	16	14	12	12	12	10	10
4.25	20	20	20	18	18	16	16	14	12	12	12	10	10
4.50	20	20	20	18	18	16	16	14	12	12	12	10	10
4.75	20	20	18	18	16	16	14	14	12	12	10	10	10
5.00	20	20	18	18	16	16	14	14	12	12	10	10	
5.25	20	18	18	16	16	14	14	12	12	12	10	10	
5.50	20	18	18	16	16	14	14	12	12	12	10	10	
5.75	20	18	16	16	14	14	12	12	12	12	10	10	
6.00	18	18	16	16	14	14	12	12	12	10	10	10	
6.25	18	18	16	14	14	12	12	12	12	10	10		
6.50	18	16	16	14	14	12	12	12	12	10	10		
6.75	18	16	14	14	14	12	12	12	10	10	10		
7.00	16	16	14	14	12	12	12	12	10	10	10		
7.25	16	16	14	14	12	12	12	10	10	10			
7.50	16	14	14	12	12	12	12	10	10	10			
7.75	16	14	14	12	12	12	10	10	10	10			
8.00	14	14	12	12	12	12	10	10	10				
8.25	14	14	12	12	12	10	10	10	10				
8.50	14	12	12	12	12	10	10	10					
8.75	14	12	12	12	12	10	10	10					
9.00	14	12	12	12	10	10	10						
9.25	12	12	12	12	10	10	10						
9.50	12	12	12	10	10	10							
9.75	12	12	12	10	10	10							
10.00	12	12	10	10	10	10							
10.25	12	12	10	10	10								
10.50	12	12	10	10	10								
10.75	12	10	10	10									
11.00	12	10	10	10									
11.25	12	10	10										
11.50	10	10	10										
11.75	10	10											
12.00	10	10											

TYPES	DESIGN THICKNESS
20	0.0358
18	0.0474
16	0.0598
14	0.0747
12	0.1046
10	0.1345

NOTES: The above Selection Table is based on following criteria:

1. Normal weight concrete (150PCF).
2. Horizontal and vertical deflection is limited to 1/4" maximum for concrete dead load.
3. Design stress is limited to 20 KSI for concrete dead load temporarily increased by one-third for the construction live load of 20 PSF.
4. Pour Stop Selection Table does not consider the effect of the performance, deflection, or rotation of the pour stop support which may include both the supporting composite deck and/or the frame.
5. Vertical leg return lip is recommended for type 16 and lighter.
6. This selection is not meant to replace the judgement of experienced Structural Engineers and shall be considered as a reference only.

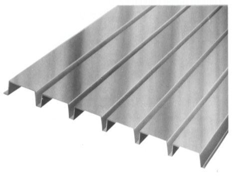
SDI reserves the right to change any information in this selection without notice.

Non-composite Deck

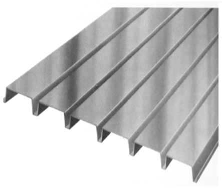
- Three primary types
- <u>Roof Deck</u> supporting rigid insulation
- <u>Vented Form Deck</u> supporting lightweight insulating concrete (LWIC) for roof
- <u>Form Deck</u> supporting concrete deck

Non-composite Deck

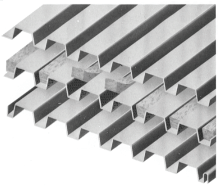
- Roof Deck



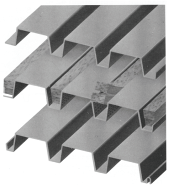
Narrow-Rib
Type A
(1” wide)
(1/2 “ insulation



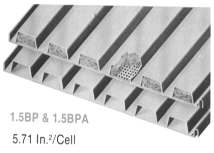
Intermediate-Rib
Type F
(1 ¾” wide)
(1” insulation)



Wide-Rib
Type B
(2 ½ “ wide)
(1” insulation)



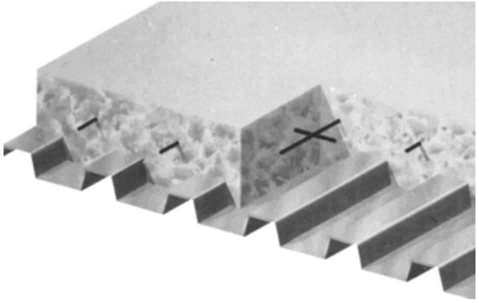
3 in Deck
Type N
(2 5/8” wide)



Cellular

Non-composite Deck

- Form deck
- Thickness of concrete above deck ranges from 1 ½” to 5”
- Comes in various depths; 9/16”, 15/16”,1 5/16”,1 ½”, 2” & 3”
- These decks also come vented for use with lightweight insulating concrete (LWIC).



Non-composite Deck Design

- Maximum span for construction safety
- P = 200 lb./ft at mid-point of one span
- F_a = 26,000 psi
- Δ_a = L/240

Span Limits by Factory Mutual

Deck Type Gage	A-NR	F-IR	B-WR	ER2R
16	-	-	8'-2"	12'-3"
18	6'-0"	6'-3"	7'-5"	10'-8"
20	5'-3"	5'-5"	6'-6"	9'-4"
22	4'-10"	4'-11"	6'-0"	-

Non-composite Deck

- Attachment to supports
- Welding – 5/8” puddle weld is typical
- Powder or compressed-air actuated pins
- Self-drilling fasteners
- Fastener spacing
  - Beware of special requirements beyond what is needed for diaphragm strength
- Minimum SDI fastener spacing for non-hurricane regions where approval or acceptance by factory mutual insurance is not required
  - 18 in. on center in field of roof and
  - 12 in. on center in perimeter and corners if rib spacing ≤ 6” otherwise, at every rib throughout
- Hurricane regions (ASCE 7-02) or FM class I90
  - 12 in. on center in field of roof, 6 in. at perimeters and corners
- FM class I120
  - 6 in. on center in field of roof, 4 ½ in. At perimeters and 3 in. in corners, but only use itwbuildex self-drilling fasteners. Use two fasteners per rib to satisfy fastener spacing less than rib spacing

Non-composite Deck

- Roof deck finishes
  - Prime painted
    - Short period of protection in ordinary atmospheric conditions
    - Do not use as final coat in exposed conditions or in highly corrosive or humid environments
  - Galvanizing
    - Use for corrosive or high humidity environments
    - G90 or G60 finish
  - Use field painting over prime coat for deck in exterior exposed conditions

Non-composite Deck

- Selecting and specifying roof deck
  - Choose deck type and depth to suit spans and insulation type, coordinate with architect
  - Be aware of special requirements such as acoustic criteria or the need for cellular deck.
  - Ensure span limits are as required in factory mutual or underwriters laboratories if applicable
  - Choose gage to carry load
  - Specify depth, gage, and deck type if standard or I, s_p, and s_n for non-standard
  - Choose either galvanized or painted finish.
  - Choose the proper minimum attachment requirements or as required for strength

Non-composite Deck

- Form deck
- Floor construction
  - Short spans (2'- 0" to 4'-6") supported by open-web bar joists
  - Deck acts as form only
  - Concrete slab is reinforced to resist load
  - Concrete is not usually thick enough to be fire-rated without further fireproofing or fire-rated floor-ceiling assembly
  - For concrete slab thickness of 2 ½" above deck or less and spans less than 5 ft, place reinforcing mat at mid-depth of slab
  - For thicker slab above deck drape the reinforcing between supports
- Roof construction
  - Vented form used to support lightweight insulating concrete such as perlite, vermiculite, or cellular foam
  - Deck is permanent and must be galvanized

Non-composite Deck

- Form deck connection to supports at roof
  - Non-hurricane zone with no Factory Mutual or UL requirement
    - To each support member at both side-lap flutes and 18” on center in field of roof and
    - 12” on center in eaves, overhangs, perimeter strips and corners
  - Factory Mutual 1-60, non-hurricane
    - To each support member at each side-lap and every other flute in field of roof and every flute in eaves, overhangs, perimeter strips and corners.
  - Hurricane zone, FM Class 1-75 or above
    - Every flute in field of roof and 2 fasteners per flute in eaves, overhangs, perimeter strips and corners
- Form deck connection to supports at floors
    - At end laps, connect at both side-laps and halfway in-between. At each interior support, connect at each side-lap.
    - This connection pattern is good for all deck spans up to 4’6”. For spans of 4’6” to 8’-0”, add connection midway between side-laps at each interior support

Diaphragms in Steel Buildings

Overview

- The role of a diaphragm in a building
- Types of diaphragms
- Diaphragm design procedure
- Diaphragm deflections
- Lateral analysis and diaphragm modeling issues
- Potential diaphragm problems

The Role of Diaphragms

Purpose: To distribute lateral forces to the elements of the Vertical Lateral Load Resisting system (VLLR)

- **Ties the building together as a unit**
- **Behaves as a horizontal continuous beam spanning between and supported by the vertical lateral load resisting system**
- **Floor acts as web of continuous beam**
- **Members at floor edges act as flanges/chords of the continuous beam**
- **Critical for stability of structure as leaning portion of structure gets stability from VLLR through the diaphragm**

Types of Diaphragms (Materials)

- Concrete slab
- Composite metal deck
- Precast elements with or without concrete topping slab (staggered truss system)
- Untopped metal deck (roof deck)
- Plywood sheathing

* Note: Standing seam roof is <u>not</u> a diaphragm

Diaphragm Classifications

Rigid

- Distributes horizontal forces to VLLR elements in direct proportion to relative rigidities of VLLR elements
- Diaphragm deflection insignificant compared to that of VLLR elements
- Examples: Composite metal deck slabs, concrete slabs (under most conditions)

Diaphragm Classifications

Flexible

- Distributes horizontal forces to VLLR elements independent of relative rigidities of VLLR elements
- Distributes horizontal forces to VLLR elements as a continuous beam on an elastic foundation
- Distributes horizontal forces to VLLR elements based on tributary areas
- Diaphragm deflection significantly large compared to that of VLLR elements
- Examples: Untopped metal decks (under some conditions)

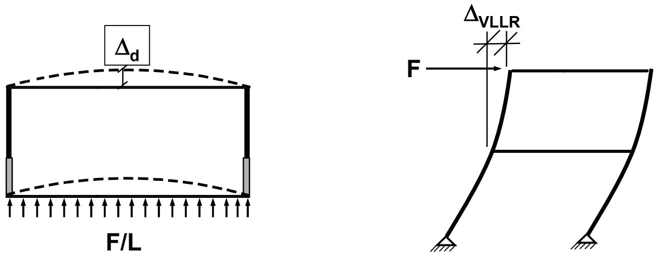
Diaphragm Classifications

3. **Semi-Rigid**
    - Deflection of VLLR elements and diaphragm under horizontal forces are same order of magnitude
    - Must account for relative rigidities of VLLR elements and diaphragm
    - Analogous to beam on elastic foundation

Diaphragm Classifications

- **UBC 1997 (§1630.6) and IBC 2003 (§1602) :**

  “A diaphragm shall be considered flexible when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story”



Plan **Elevation**

If $\Delta_d > 2\Delta_{VLLR}$ Diaphragm is Flexible

Diaphragm Classifications

(Steel Deck Institute)

G’	Flexibility	
6.67 - 14.3	flexible	Metal Deck
14.3 - 100	semi-flexible	
100 - 1000	semi-rigid	Filled
> 1000	rigid	

<u>Note</u>: These classifications are guidelines. To accurately classify a diaphragm, the stiffness of the diaphragm must be compared to that of the VLLR system.

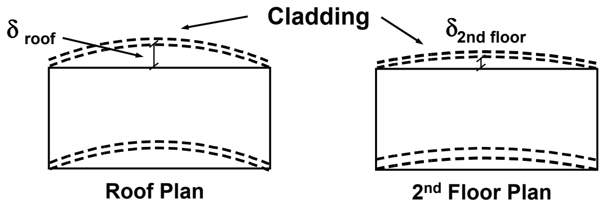
Diaphragms

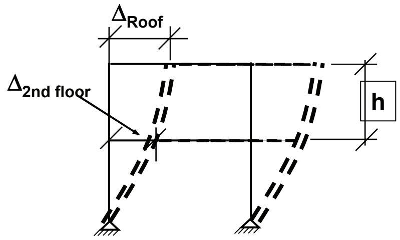
In-plane deflection:

- Deflection shall not exceed permissible deflection of attached elements
- Permissible deflection is that which will permit the attached element to maintain its structural integrity under the applied lateral loads and continue to support self weight and vertical load if applicable

$$\frac{\Delta_{roof} - \Delta_{2nd\ floor} + \left(\delta_{roof} - \delta_{2nd\ floor}\right)}{h}$$

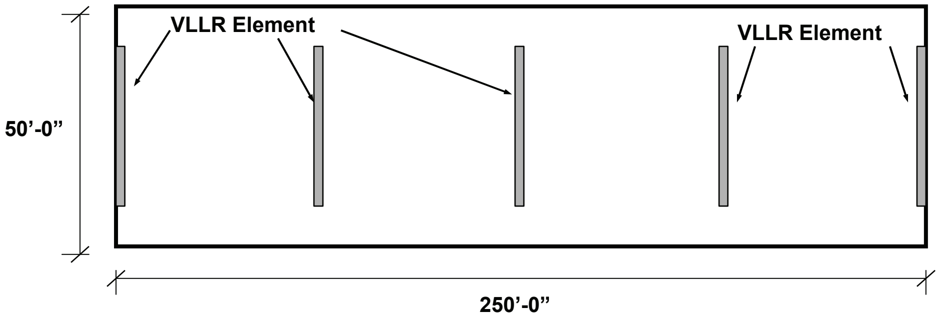
Drift ratio experienced by cladding (out-of-plane):



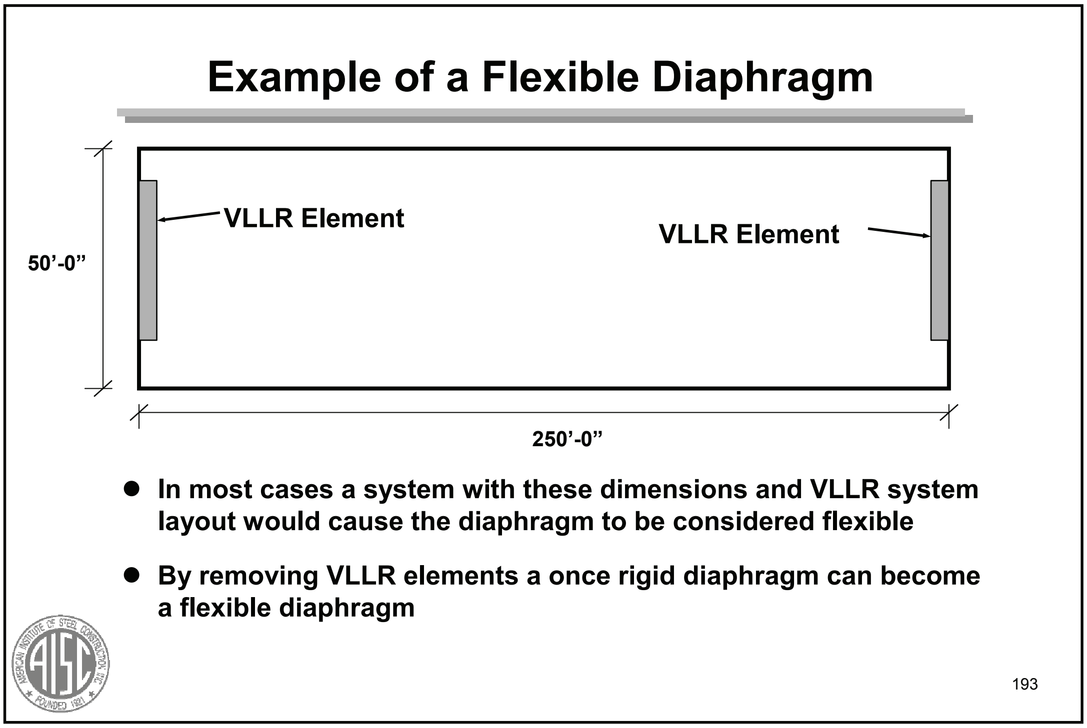


Note: The h/400 limit state is generally for the in-plane deflections. The out-of-plane drift limits are generally less stringent.

Example of a Rigid Diaphragm

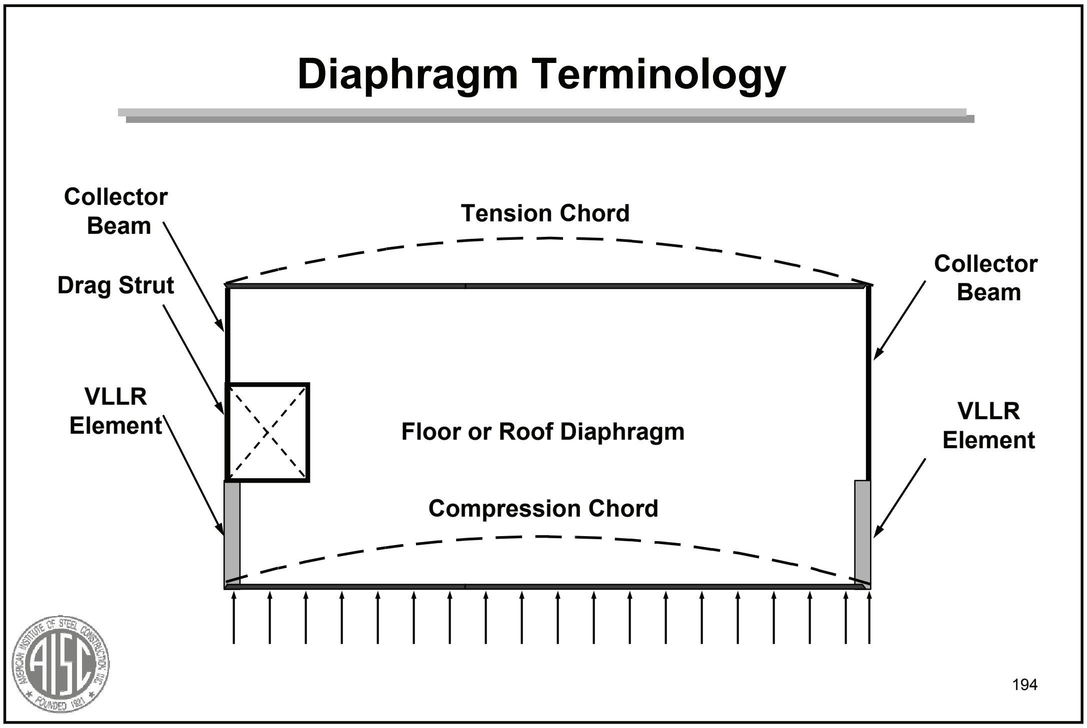


- Diaphragm flexibility is not solely based on geometry
- The location and relative stiffness of the VLLR elements is key to determining diaphragm flexibility



Example of a Flexible Diaphragm

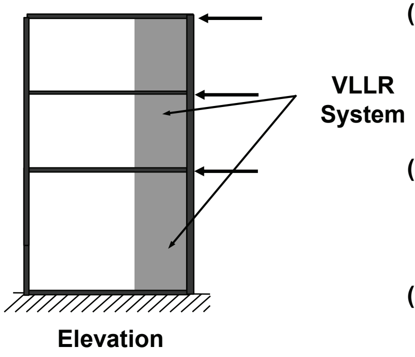
- In most cases a system with these dimensions and VLLR system layout would cause the diaphragm to be considered flexible
- By removing VLLR elements a once rigid diaphragm can become a flexible diaphragm



Diaphragm Terminology

Diaphragm Design Procedure

Calculate diaphragm design forces at all levels.



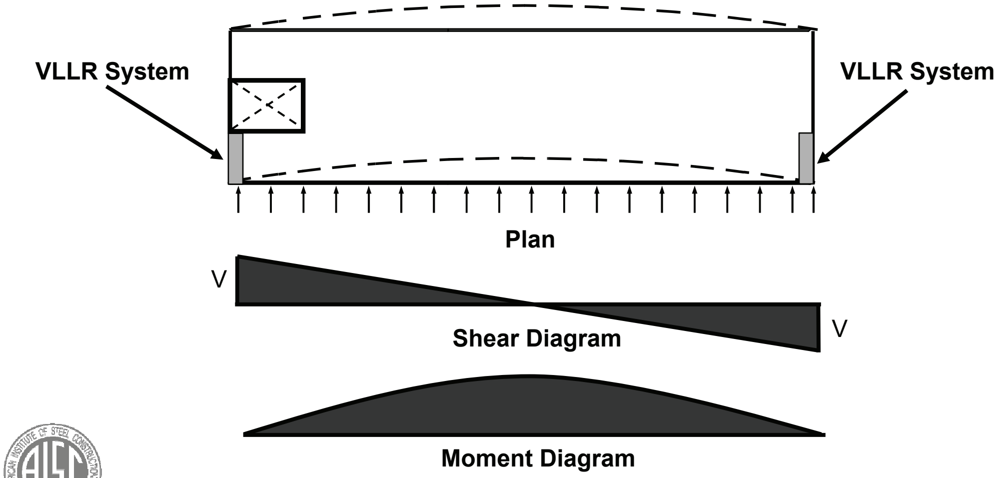
(A) For seismic forces follow the building code to determine F_{eq}. Distribute F_{eq} along the floor in the same proportion as the mass distribution at that level.

(B) For wind forces distribute F_w along the floor based on the wind pressure and tributary height.

(C) Add any shears caused by offsets in VLLR system and changes in stiffness of VLLR system.

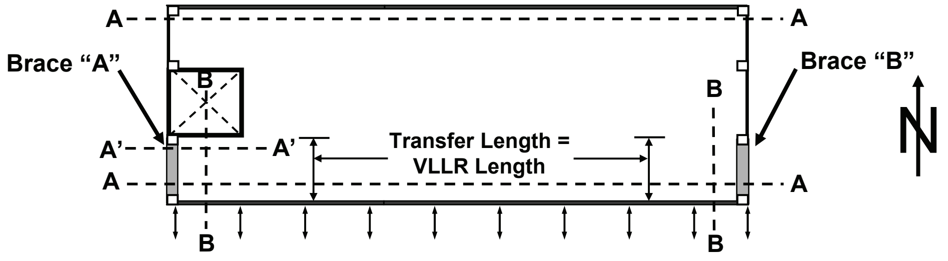
Diaphragm Design Procedure

Draw diaphragm shear and moment diagrams.



Diaphragm Design Procedure

- Approach 1: All shear transferred directly to VLLR system
  - (No collector beams, No drag struts)



A. Chord tension/compression capacity of diaphragm

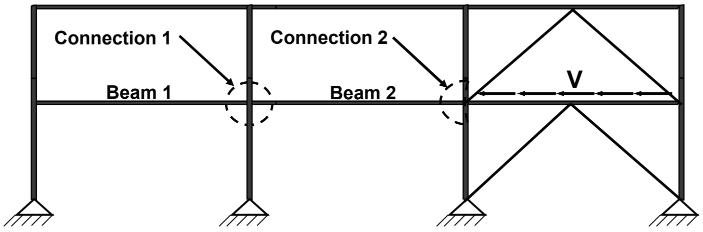
Chord force, T = M_u/d (d = Effective depth of diaphragm)

A'. Chord force at corner opening near brace "A" must be checked (same as coped beam)

B. Diaphragm shear capacity (check over VLLR length)

Diaphragm Design Procedure

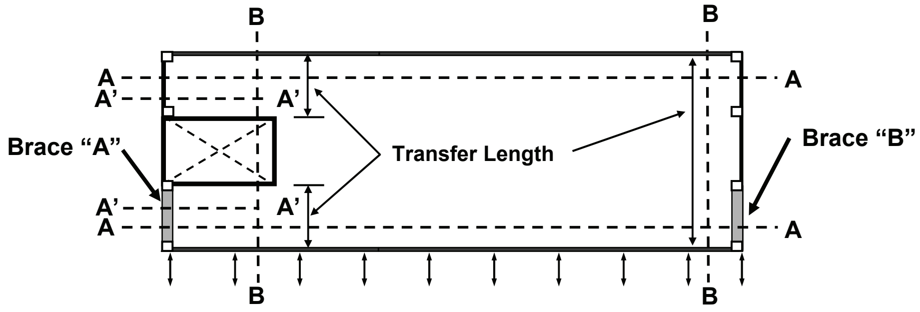
- Connection design: Approach 1
  - Braces "A" and "B"



- All shear is transferred directly to the VLLR system.
- No special consideration of diaphragm forces must be given to the design of beam to column connections 1 and 2 or beams 1 and 2.

Diaphragm Design Procedure

- Approach 2: Shear transferred to collector beams and VLLR system



A. Chord tension/compression capacity related to Overall diaphragm behavior

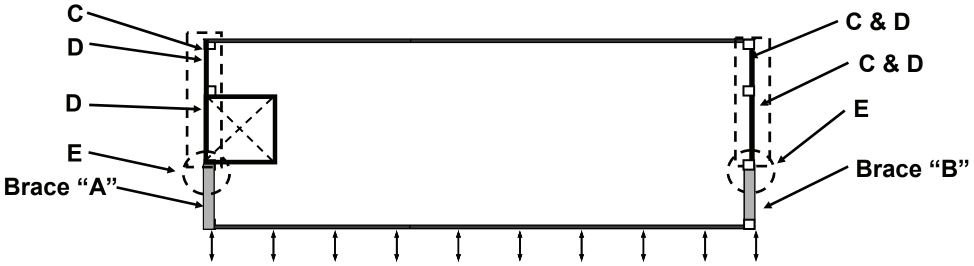
Chord force, T = M_u/d (d = Effective depth of diaphragm)

A'. Chord tension/compression capacity related to local diaphragm bending (same as coped beam)

B. Diaphragm shear capacity (check over transfer length)

Diaphragm Design Procedure

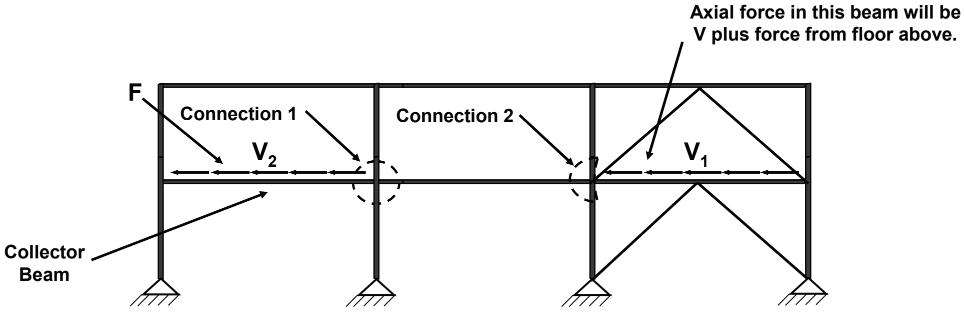
- Design collector beams and drag struts



C. Force transfer from diaphragm to collector beam
D. Axial capacity (collector beam / drag strut)
E. Connection to VLLR system

Diaphragm Design Procedure

- Connection design method 2 (Brace A)



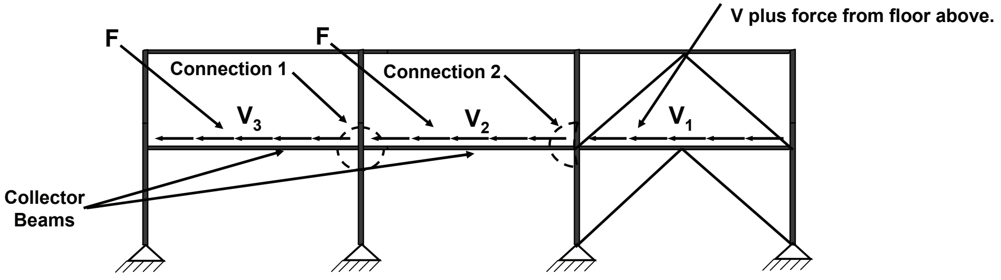
$V_1 + V_2 = V$

F. Provide connection between diaphragm and collector beam to transfer diaphragm force. Design collector beam for this force.

G. Connections 1 and 2 must be designed to carry the axial force (V_2) through the beam/column connection.

Diaphragm Design Procedure

- Connection design method 2 (Brace B)



$V_1 + V_2 + V_3 = V$

F. Provide connection between diaphragm and collector beam to transfer diaphragm force. Design collector beam for this force plus any force from previous collector beams.

G. Connections 1 and 2 must be designed to carry the axial forces (1 for V_3) (2 for $V_2 + V_3$) through the beam/column connection.

Diaphragm Force Transfer

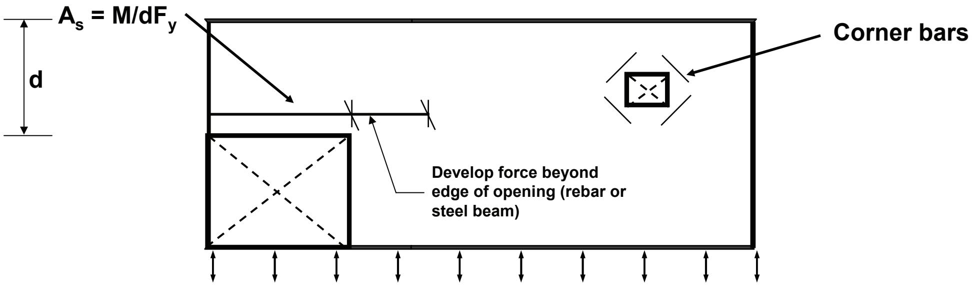
(With Joists as Purlins)

- The joist seat causes a gap to exist between the roof deck and the beam
- Forces generated within the diaphragm must be transferred to the beam
- Filler tubes (or inverted angle) of joist seat depth must be provided to transfer the diaphragm force
- Show clearly on plan where typical detail applies
- In certain cases (very small structures) a limited amount of force can be transferred through the joist seat. An example of this method is shown of page 269 of "Designing with Steel Joists, Joist Girders, and Steel Deck" by Fisher, West, and Van de Pas. If this method is used both the weld strength and the strength of the joist seat should be checked.

Diaphragm Design Procedure

(Deflections and Detailing)

- Check anticipated areas of stress concentrations, add reinforcement if necessary (e.g. at reentrant corners and around openings)



Plan

Diaphragm In-Plane Deflections

- **Bending Deflection:**
  - ◆ **(Distributed Load on Simple Span)**

$$\Delta_B = \frac{5\,q\,L^4}{384\;EI}$$

- **Shear Deflection:**
  - ◆ **(Distributed Load on Simple Span)**

$$\Delta_S = \frac{q\,L^2}{8\,B\,tG}$$

- **Shear Deflection:**
  - ◆ **(Point Load on Simple Span)**

$$\Delta_S = \frac{V\,L}{4\;B\,tG}$$

$$\Delta = \Delta_B + \Delta_S$$

- **t*G is referred to as G' in SDI and Vulcraft publications**
- **In most diaphragms ΔS dominates the total deflection.**

Diaphragm Shear Stiffness

- **SDI equation 3.3-3:**
  Based on diaphragm geometry and connector flexibilities (See SDI Manual)

$$G' = \frac{Et}{2.6\left(\frac{s}{d}\right) + \phi D_N + C}$$

- **Vulcraft equation:**
  Derived directly from SDI equation

$$G' = \frac{K2}{\alpha + \left(\frac{0.3D_X}{SPAN}\right) + \left(3 \times K \times SPAN\right)}$$

Factor to account for warping

Connector slip parameter

Filled Diaphragm Shear Stiffness

- SDI equation 5.6-1:

$$G' = \frac{Et}{2.6\left(\frac{s}{d}\right) + C} + 3.5d_c\left(f'_c\right)^{0.7}$$

- Vulcraft equation:

$$G' = \frac{K2}{\alpha + \left(3 \times K \times SPAN\right)} + K3$$

Parameter to account for fill

- ϕD_N (D_X) term from earlier equations approaches 0 in filled decks because the fill “substantially eliminates panel end warping for loads within the design range” (SDI Manual)

Structural Concrete Diaphragm Shear Stiffness

$$G = \frac{E}{2(1+\nu)} \qquad \nu \approx 0.2$$ = Poisons ratio

- Flexural deformation may be significant compared to shear deformation

$$I = \frac{tb^3}{12}$$

- Finite element model using appropriate t and geometry can be used to determine diaphragm stiffness

Metal Deck Diaphragm Strength

- Metal deck strength (S) is a function of:
  - Edge fastener strength
  - Interior panel strength
  - Corner fastener strength
  - Shear buckling strength
- Equations for the above limit states are described in chapter 2 of the SDI manual
- Diaphragm shear strengths for given deck / fastener combinations are tabulated in the manufacturer catalog
- Diaphragm strengths for decks or fasteners not listed in manufacturer catalog can be computed using equations given in SDI manual

Metal Deck Diaphragm Strength

$$S = \frac{S_n}{SF}$$

- S_n = Ultimate Shear Strength
- S = Allowable Shear Strength
- SF = Safety factor
  - SF = 2.75 bare steel diaphragms – welded
  - SF = 2.35 bare steel diaphragms – screwed
  - SF = 3.25 filled diaphragms
- Values tabulated in the manufacturer catalog are generally based on Allowable Stress Design
- Values tabulated are based on a 1/3 increase in stress due to short term or wind loading. <u>Do not increase the tabulated capacities</u>.

LRFD Metal Deck Diaphragm Design

$\underline{\text{Capacity}}$ x $\underline{\text{S.F.}}$ = S_n, $\phi S_n > S_u$

From Catalog

- **ϕ = 0.60** For diaphragms for which the failure mode is that of buckling, otherwise;
- **ϕ = 0.50** For diaphragms welded to the structure subjected to earthquake loads, or subjected to load combinations which include earthquake loads.
- **ϕ = 0.55** For diaphragms welded to the structure subjected to wind loads, or subjected to load combinations which include wind loads.

Compare to factored loads S_u per ASCE 7

Ref: How to Update Diaphragm Tables by Larry Luttrel, Ph.D., P.E. (From SDI)

LRFD Metal Deck Diaphragm Design

$\underline{\text{Capacity}}$ x $\underline{\text{S.F.}}$ = S_n, $\phi S_n > S_u$

From Catalog

- **ϕ = 0.60** For diaphragms mechanically connected to the structure subjected to earthquake loads, or subjected to load combinations which include earthquake loads.
- **ϕ = 0.65** For diaphragms mechanically connected to the structure subjected to wind loads, or subjected to load combinations which include wind loads.
- **ϕ = 0.65** For diaphragms connected to the structure by either mechanical fastening or welding subjected to load combinations not involving wind or earthquake loads.

Compare to factored loads S_u per ASCE 7

Ref: How to Update Diaphragm Tables by Larry Luttrel, Ph.D., P.E. (From SDI)

Concrete Diaphragm Strength

- **The strength of cast in place concrete diaphragms is discussed in Chapter 21 of ACI 318**
  - **The strength of a cast in place concrete diaphragm (V_n) is governed by the following equation:**

$$V_n = A_{cv}\left(2\sqrt{f_c^{'}} + \rho_n f_y\right) \quad \text{ACI Eqn. 21-10}$$

  - **And shall not exceed:**

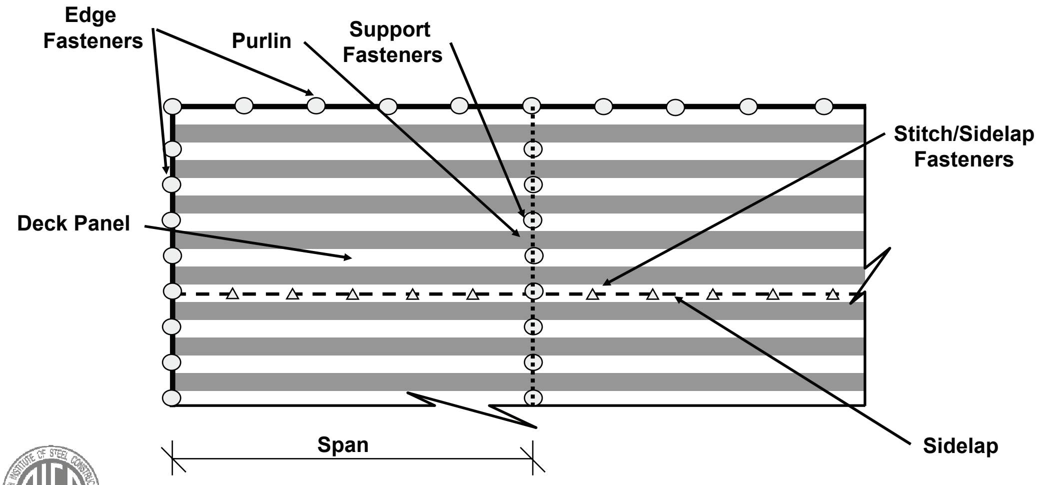
$$V_{n_{max}} = A_{cv}\left(8\sqrt{f_c^{'}}\right)$$

  - **<u>Notes</u>:**
    - **Design of concrete diaphragms is based on ultimate strength**
    - **F_y shall not exceed 60 ksi**
    - **For lightweight concrete f'_c shall not exceed 4000 psi**
    - **No reduction factor is used for lightweight concrete**

$$\phi V_n > V_u$$

$$\phi = .85 \text{ ACI 318-99, } \phi = .75 \text{ ACI 318-02}$$

Metal Deck Diaphragm Terminology



Edge Fasteners

Purlin

Support Fasteners

Stitch/Sidelap Fasteners

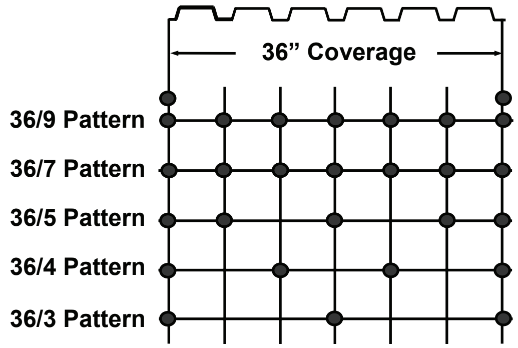
Deck Panel

Span

Sidelap

Metal Deck Diaphragm Fasteners

(Typical Fastener Layouts)



- 36/9 = 36” Deck Width with 9 Fasteners Per Deck Width
- Note that Fasteners at Panel Overlaps are “Double Counted”

Metal Deck Diaphragm Fasteners

(Computation of No. of Sidelap Fasteners)

- Most Deck Manufacturers (Vulcraft, USD, CSI)
  - Sidelap fasteners in deck valleys
  - No sidelap fasteners above purlins
  - No. of sidelaps per span = (span length / spacing) -1

Vulcraft deck above supports

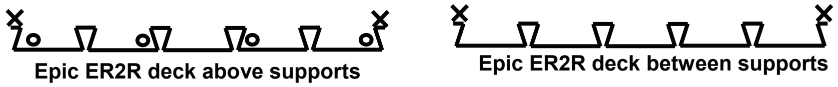
Vulcraft deck between supports

o End fastener
× Sidelap fastener

Metal Deck Diaphragm Fasteners

(Computation of No. of Sidelap Fasteners)

- **Epic Decks (ER2R)**
  - Sidelap fasteners at deck ridges
  - Sidelap fasteners above purlins
  - No. of sidelaps per span = (span length / spacing) + 1



○ End fastener
× Sidelap fastener

Diaphragm Chord Forces

- Design beams for axial force in addition to gravity loads
- Design connections to transfer the through force

OR

- Provide a continuous edge angle designed to carry the axial force.
  - Note that the angle must be continuous and the connection between angle sections must be capable of transmitting axial force
  - Welding of angle is very difficult due to field tolerances and beam cambers
  - Consider using splice plates to lap two adjacent angles to transfer axial force
  - In the case of a concrete filled diaphragm continuous reinforcing can be provided to resist chord forces

Diaphragm Modeling

(Rigid diaphragms)

- Rigid Diaphragm
  - No relative in-plane displacement of joints within diaphragm
  - Beams with both end joints connected to a rigid diaphragm shall not have axial force

    - Design using post processors will be wrong
  - Beams with both end joints connected to a rigid diaphragm shall not have axial deformation

    - In a braced frame, the building stiffness could be overestimated by as much as 15% if axial deformation of beams is not considered
  - Release the beam ends strategically from the rigid diaphragm so that axial force and deformation are captured
  - Release of excessive joints may create instability

Diaphragm Modeling

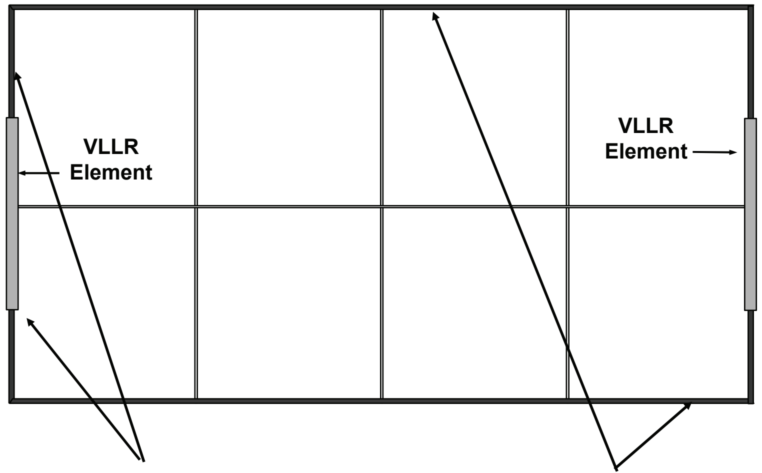
(Flexible diaphragms)

- Flexible Diaphragm
  - Modeled using in-plane shell elements
  - Beams axial force is function of shell in-plane stiffness

    - Design using post processors will be wrong
  - Beam design force = force in beam + axial force in shell tributary to the beam
  - Manual calculations required for proper force determination

Diaphragm Modeling

(Include load path in modeling)



Drag struts must be modeled
(to capture load path stiffness)

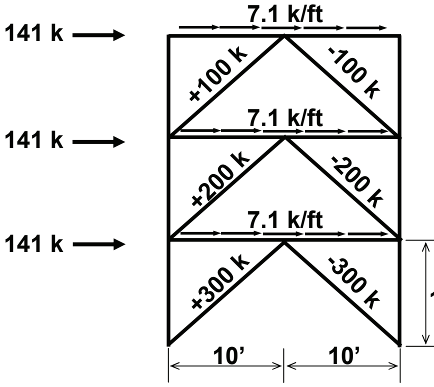
Chords must be modeled
(to capture effective I)

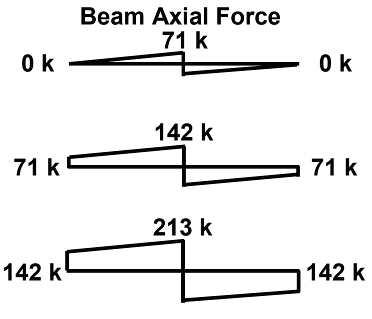
Diaphragm Modeling

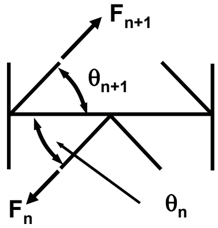
- **Flexible diaphragms can be modeled as membrane elements**
- **To assign a rigid diaphragm select joints and assign rigid diaphragm constraints or master slave relationship**

Diaphragm Modeling

(Determination of beam axial force by static)



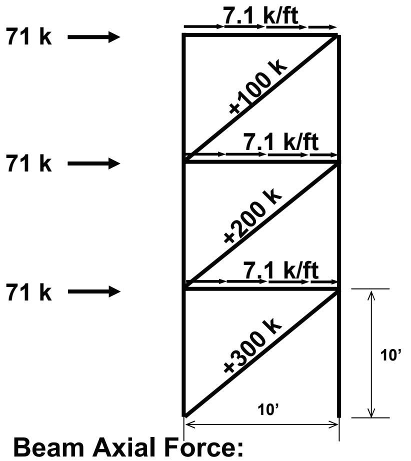


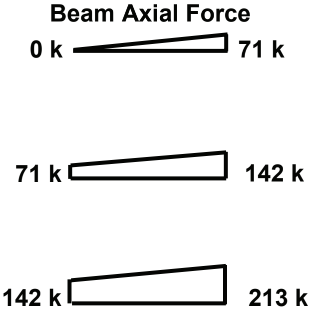


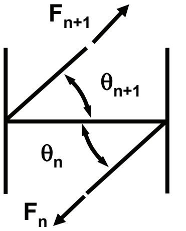
- **Beam Axial Force:**
  = $F_{n+1}\ cos\theta_{n+1} + (F_n\ cos\theta_n - F_{n+1}\ cos\theta_{n+1})$

Diaphragm Modeling

(Determination of beam axial force by statics: Approach 1)



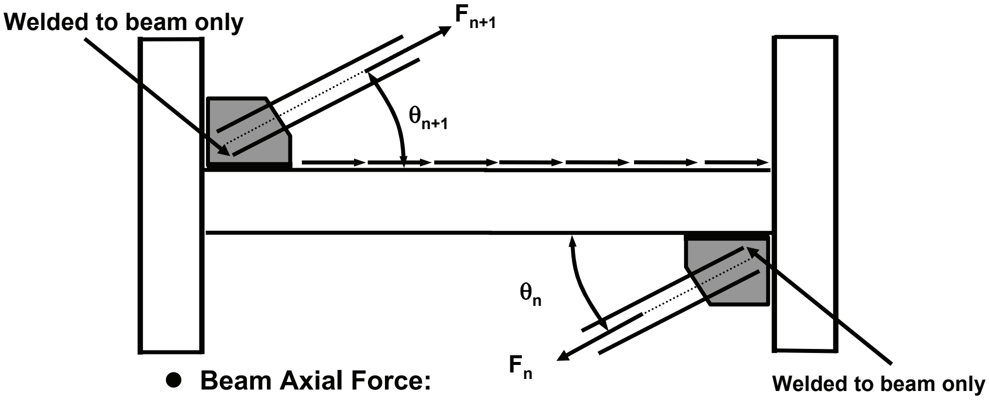




- **Beam Axial Force:**
  = $F_{n+1}\ cos\theta_{n+1} + (F_n\ cos\theta_n - F_{n+1}\ cos\theta_{n+1})$

Diaphragm Modeling

(Beam axial force by statics: detail)



- **Beam Axial Force:**

$$= \underline{F_{n+1}\cos\theta_{n+1}} + (\underline{F_n\cos\theta_n - F_{n+1}\cos\theta_{n+1})}$$

Force transferred from brace to beam through gusset plate

Force transferred from diaphragm to beam

Diaphragm Modeling

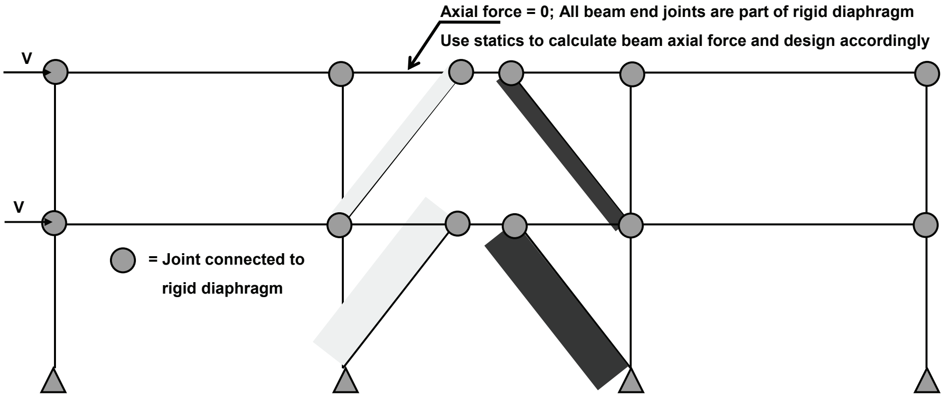
- Rigid diaphragm
  - No relative in-plane displacement of joints within diaphragm
  - Beams with both end joints connected to rigid diaphragm will not have axial force

    - Design using post processors will be wrong
  - Beams with both end joints connected to rigid diaphragm will not have axial deformation

    - In braced frames, the building stiffness could be overestimated by as much as 15% if axial deformation of beams is not considered
  - Release the beam ends strategically from rigid diaphragm so that axial force and deformation is captured
  - Release of excessive joints may create instability
  - When release of the beam ends from rigid diaphragm is not possible, use statics to determine beam force and manual design (covered under diaphragm seminar)

Diaphragm Modeling (Rigid)

All joints connected to diaphragm (eccentric or concentric brace)



Axial force = 0; All beam end joints are part of rigid diaphragm

Use statics to calculate beam axial force and design accordingly

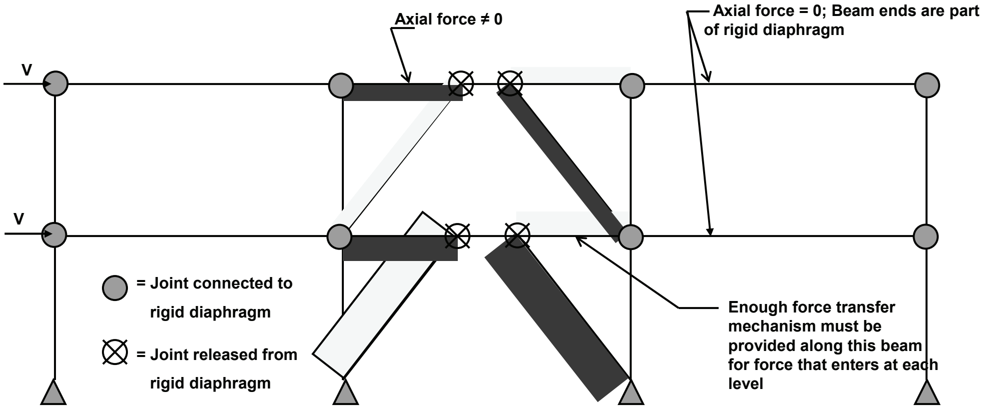
● = Joint connected to rigid diaphragm

Axial force diagram

Not Recommended

Diaphragm Modeling (Approach #1)

(Selected joints released from diaphragm; eccentric or concentric brace)



Axial force ≠ 0

Axial force = 0; Beam ends are part of rigid diaphragm

● = Joint connected to rigid diaphragm

⊗ = Joint released from rigid diaphragm

Enough force transfer mechanism must be provided along this beam for force that enters at each level

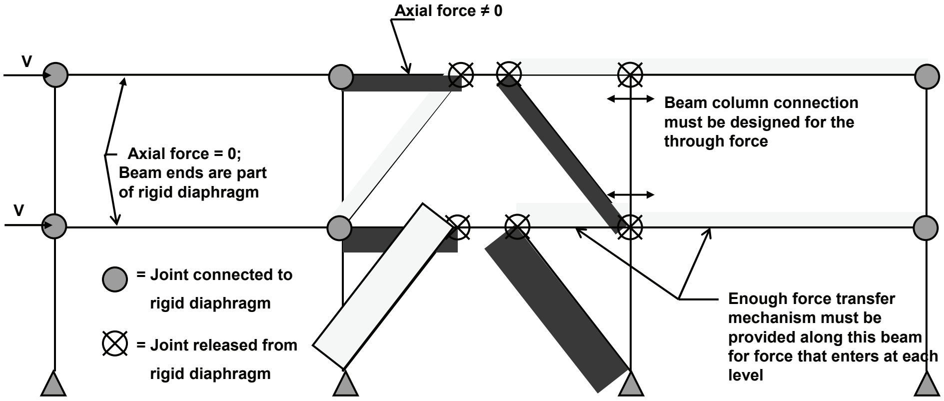
This approach should be used only when enough force transfer mechanism can be provided within the brace beam length

Axial force diagram

Recommended

Diaphragm Modeling (Approach #2)

(Selected joints released from diaphragm; eccentric or concentric brace)



This approach should be used only when approach #1 can not be used (enough force transfer mechanism can not be provided within the brace beam length)

Axial force diagram

Recommended when required

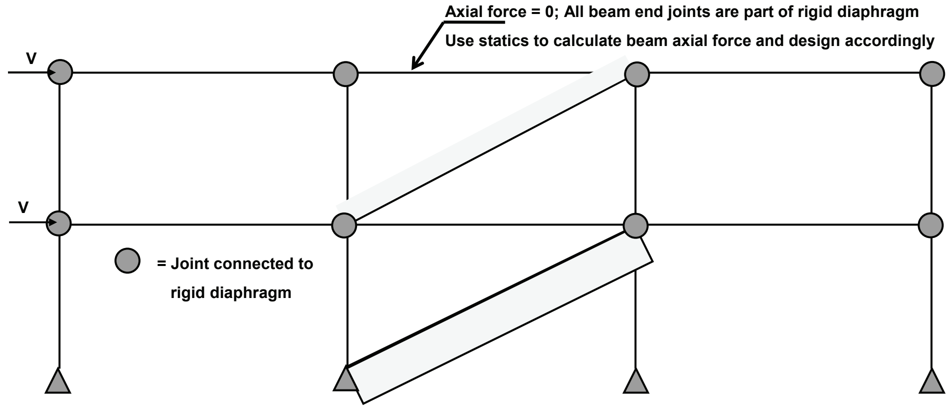
Diaphragm Modeling (Approach #1)

(Selected Joints Released from Diaphragm)

- **Approach #1 is generally accepted when the concrete floor diaphragm is continuously attached to the brace beam**
- **The shear strength of the floor diaphragm over the brace beam shall be adequate to transmit the force that enters the brace at the floor in consideration**
- **Enough shear connectors must be provided over the brace beam length to transfer above force**
- **If force transfer within the brace length is not possible use approach #2**
- **The beam column connections are not required to be designed for any through force in this approach**

Diaphragm Modeling (Rigid)

All joints connected to diaphragm (concentric brace)



Axial force = 0; All beam end joints are part of rigid diaphragm

Use statics to calculate beam axial force and design accordingly

V

V

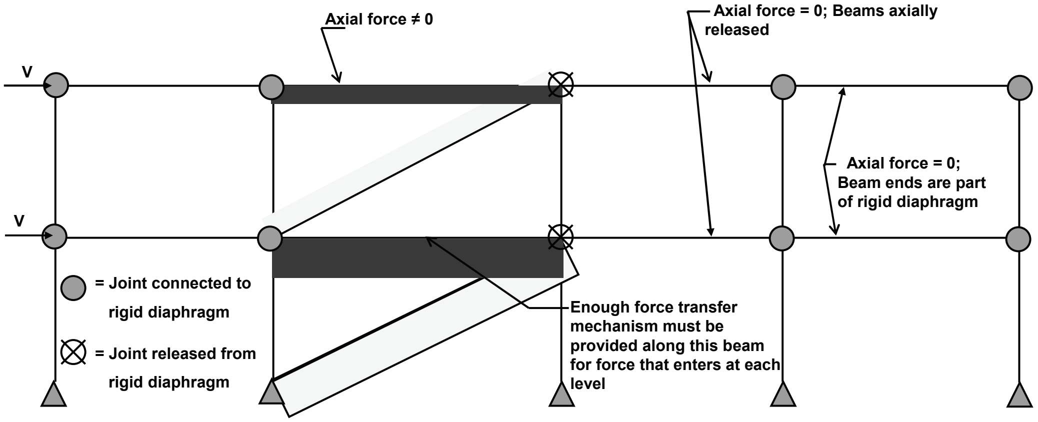
● = Joint connected to rigid diaphragm

Axial force diagram

Not Recommended

Diaphragm Modeling (Approach #1)

(Selected joints released from diaphragm; concentric brace)



Axial force ≠ 0

Axial force = 0; Beams axially released

Axial force = 0; Beam ends are part of rigid diaphragm

V

V

● = Joint connected to rigid diaphragm

⊗ = Joint released from rigid diaphragm

Enough force transfer mechanism must be provided along this beam for force that enters at each level

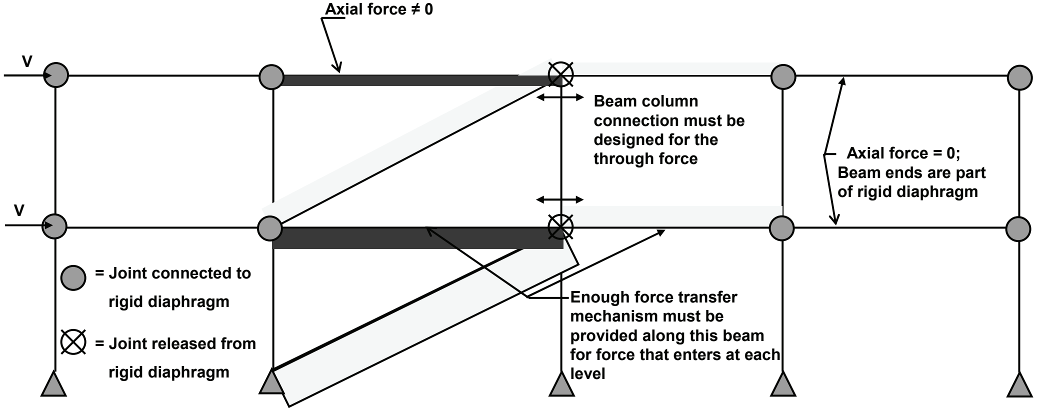
This approach should be used only when enough force transfer mechanism can be provided within the brace beam length

Axial force diagram

Recommended

Diaphragm Modeling (Approach #2)

(Selected joints released from diaphragm; concentric brace)



Beam column connection must be designed for the through force

Axial force = 0; Beam ends are part of rigid diaphragm

● = Joint connected to rigid diaphragm

⊗ = Joint released from rigid diaphragm

Enough force transfer mechanism must be provided along this beam for force that enters at each level

This approach should be used only when approach #1 can not be used (enough force transfer mechanism can not be provided within the brace beam length)

Axial force diagram

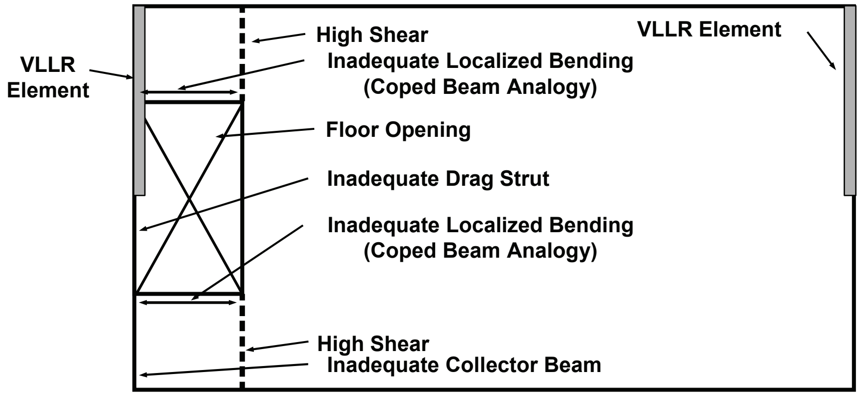
Recommended when required

Diaphragm Modeling

- Flexible diaphragm
  - Modeled using in-plane shell elements
  - Beams axial force is function of shell in-plane stiffness
    - Design using post processors will be wrong
  - Beam design force is = force in beam + axial force in shell tributary to the beam
  - Statics can be used to determine beam force followed by manual design

Potential Diaphragm Problems

(Isolated lateral load resisting system)



VLLR Element

High Shear

Inadequate Localized Bending
(Coped Beam Analogy)

VLLR Element

Floor Opening

Inadequate Drag Strut

Inadequate Localized Bending
(Coped Beam Analogy)

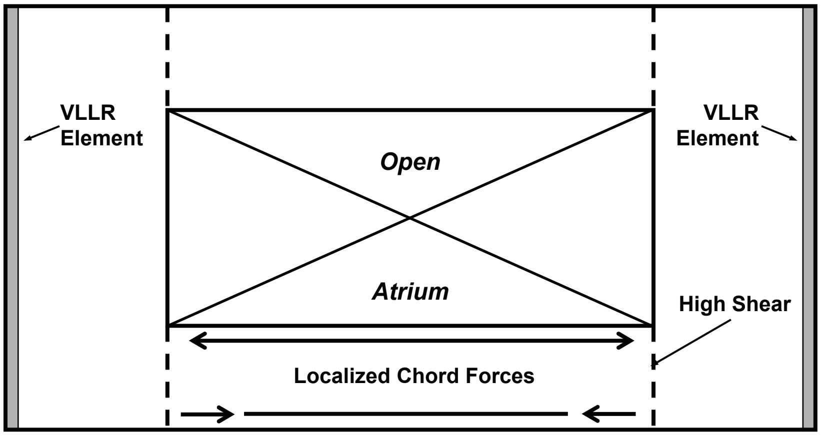
High Shear

Inadequate Collector Beam

***Problem*: Too little diaphragm contact to VLLR System**

Potential Diaphragm Problems

(Large opening in floor diaphragm)



VLLR Element

VLLR Element

Open

Atrium

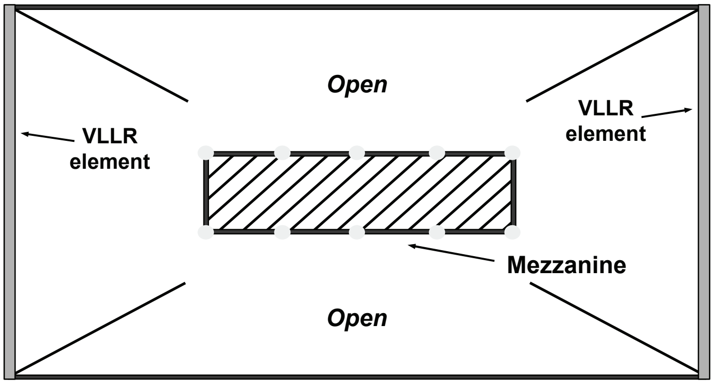
High Shear

Localized Chord Forces

***Problem*: High shear and/or bending stress in diaphragm**

Potential Diaphragm Problems

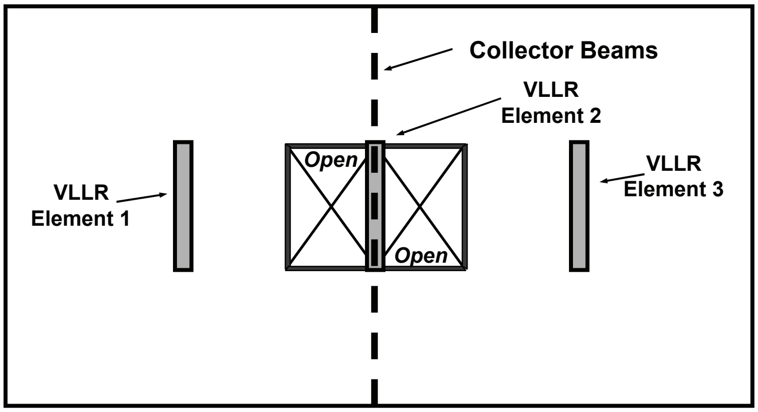
(Partial diaphragm)



Problem: **Improper column bracing for core columns**

Potential Diaphragm Problems

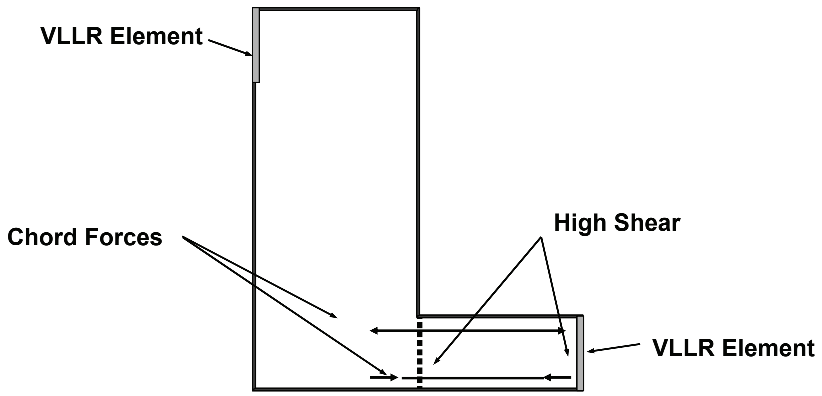
(Isolated VLLR element)



Problem: **100% of force for VLLR2 must come through the collector beam. Beam and connections must be designed for these forces.**

Potential Diaphragm Problems

(Narrow diaphragm near VLLR)



VLLR Element

Chord Forces

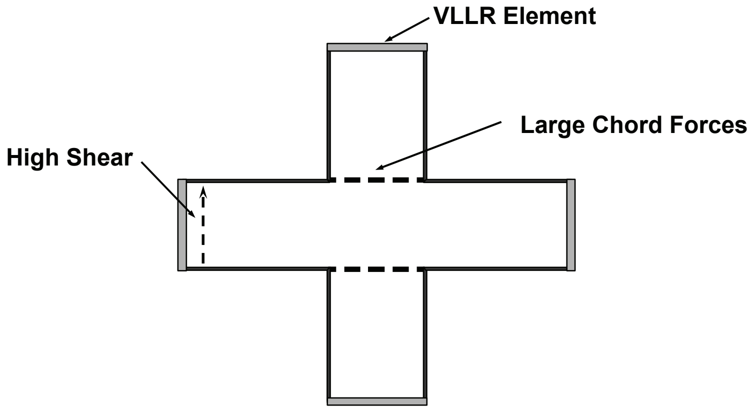
High Shear

VLLR Element

Problem: **High shears in narrow diaphragm**

Potential Diaphragm Problems

(Long Narrow Diaphragms)



VLLR Element

Large Chord Forces

High Shear

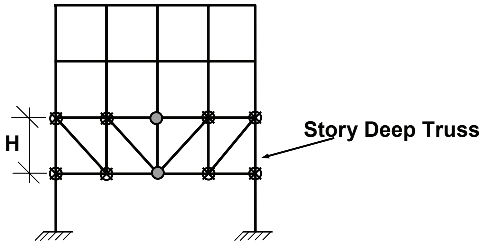
Problem: **High Shears, Diaphragm Deflections**

Potential Diaphragm Problems

Story Deep Truss Subjected To Gravity and Lateral Loads

● = Joint connected to rigid diaphragm

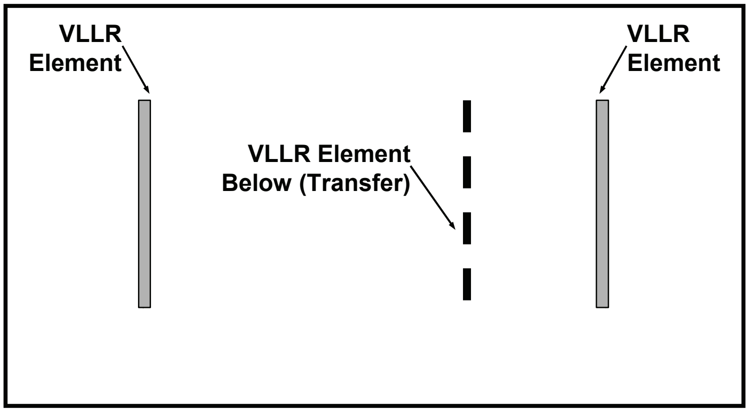
⊠ = Joint released from rigid diaphragm



- Truss $I \sim 2A(H/2)^2$
  - Where A = axial area of chords
- With rigid diaphragm $A = \infty$. Therefore deflection and force distribution will be incorrect.
- The joints of truss chords must be released from the diaphragm for correct axial forces
- Release all but joint at center from rigid diaphragm

Potential Diaphragm Problems

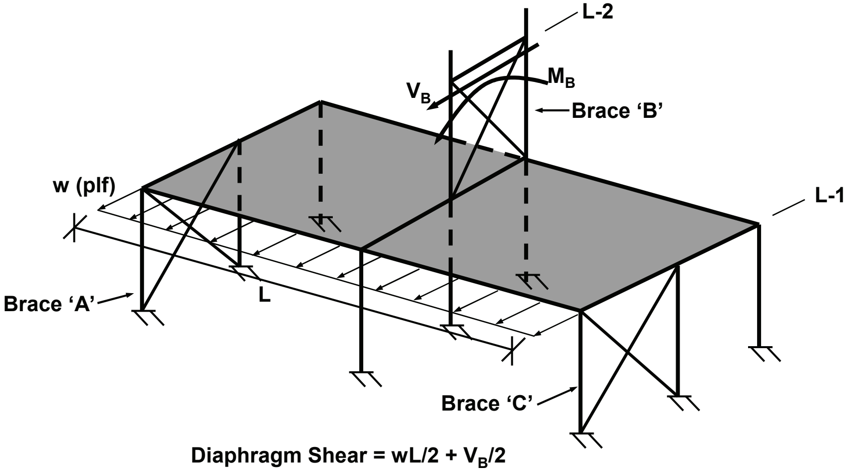
(Transfer of VLLR)



Problem: **High shear stress in diaphragm at transfer**

Potential Diaphragm Problems

(Transfer of VLLR)

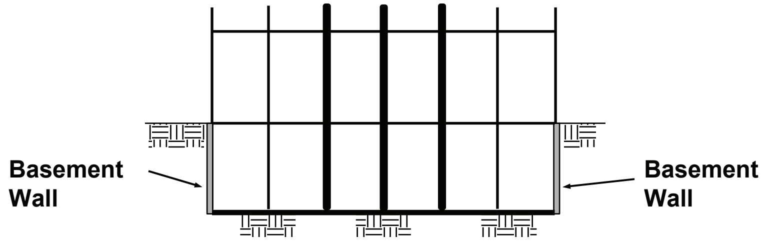


Diaphragm Shear = $wL/2 + V_B/2$

Diaphragm Moment = $wL^2/8 + V_BL/4$

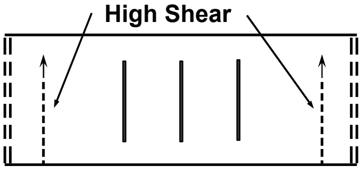
Potential Diaphragm Problems

(Transfer of VLLR at base)



BASE

Elevation



Ground Floor Plan

***Problem*: <u>Large</u> transfer in story shear**

Diaphragm “Top 10”

- **Top 10 Reasons Why You May Have Diaphragm Related Problems:**

10. Diaphragm shear capacity is not checked

9. Connections are not designed to transfer chord and collector forces

8. Force transfer from the diaphragm to collector beams / VLLR system is not considered, (load path is not clearly defined)

7. Chord and collector beams are not designed properly

6. Most of the time diaphragms are not modeled in an analytical model

5. Axial force in beams is incorrect

4. Large openings in the floor are not given proper attention

3. Building has basement walls and these walls are ignored (Thinking it is conservative to ignore them)

2. Some of the VLLR elements do not go all the way to foundation but transfer into plan

1. Computer analysis results do not tell the Engineer if there is a problem!

Column Base

Column Base

- Critical interface between the steel structure and the foundation
- Required to resist significant level of forces
- Involves design of:
  - Weld between column and base plate
  - Base plate (including stiffeners when used)
  - Grout
  - Anchor rods and plate washers
  - Concrete base

Column Base

- Connection at the base can have significant effect on the behavior of the structure
- Analytical models often assumes pinned or fixed column supports
- Improper boundary conditions can lead to error in the computed drift, P-Δ effect

Column Base

- **Base Plate**
  - **A36 (Gr 36 up to 8 inch and Gr 32 over 8 inch)**
  - **A572 (Gr 50 up to 4 inch Gr 42 over 4 and up to 6 inch)**
  - **There is no reason for higher grade material**
  - **Increasing thickness is preferred to increasing strength**
  - **Fillet welds are preferred over PJP or CJP**

Column Base

- **Grout**
  - **Grout is used for building column bases**
  - **Non-Metallic Non-Shrink Grout conform to ASTM C1107**
  - **Metallic grout only required under vibratory machinery**
  - **Grout strength as determined by cube test at 28 days is based on supporting concrete strength:**
    - **6,000 psi for supporting concrete with $f'_c \leq 3{,}000$ psi**
    - **8,000 psi for supporting concrete with $f'_c > 3{,}000$ psi to 4,000 psi**
    - **The ratio of 2 represents sqrt(A_2/A_1) for bearing check**
  - **Grouting under base plate is not recommended when double nut is used for outdoor connection**
    - **Grout may crack and retain moisture → Corrosion**

Column Base

- Anchor rods
  - Smooth rods (with head or nut at embedded end) are generally used
  - ASTM F1554 rods are used with heavy hex nut at embedded end
    - Available grades: 36, 55 and 105
    - Grade 36 normally weldable
    - Supplemental specifications for Grade 55 weldable anchors must be requested
    - Grade 105 not weldable
  - Galvanized anchors are used for exposed conditions
  - Galvanized anchors and nuts should be purchased from same supplier and should be shipped preassembled
  - Post installed anchors are suitable only for very small loads and not practical for building columns

---

Column Base

- Anchor Rods (F1554)
  - Marking at the exposed end

Grade	Color
36	Blue
55	Yellow
105	Red

- White paint mark at embedded end for Grade 55 when weldable

Column Base

- Nuts
  - ASTM 563
  - If bearing area of the head is smaller than required for concrete strength, plate washer above nut may be required (ACI 318-02 Appendix D)
- Washers
  - ASTM F436
  - Standard flat circular, square or rectangular beveled
  - Plate washers need not be welded to base plate unless anchor rods are required to resist shear and movement required to engage the anchor rods in bearing can not be tolerated by structure

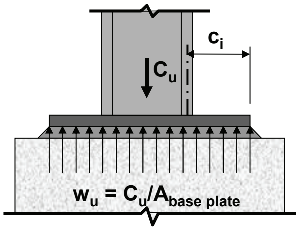
Design of Column Bases

- Column base subjected to:
  - Axial load only (compression or tension)
  - Axial load and shear
  - Axial load and moment
- Column base design should satisfy OSHA requirements
  - The requirements exclude posts which weigh less than 300 pounds
  - Min. 4 anchor rods in column base plate connections
  - Moment capacity to resist a minimum eccentric gravity load of 300 pounds located 18 in. from the extreme outer face of the column in each direction

Column Base with Axial Load Only

- Compression:
  - Base plate design
  - Concrete bearing
- Tension
  - Develop and implement mechanism to transfer of force to base
  - Base plate design
  - Anchor rod design

Base Plate (Axial Load only)



<u>ϕM_n of plate ≥ M_u at each direction</u>

$M_u = w_u \times L_i \times c_i^2 / 2$

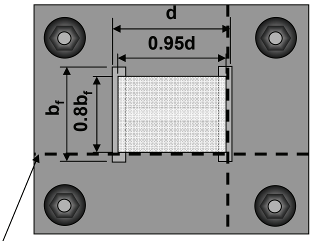
: L_i = Width of plate parallel to the section M_u is being calculated

c_i = Max. cantilever length from critical sections ≥ $\lambda \times n'$

$n' = \sqrt{(d\, b_f)}/4$ and

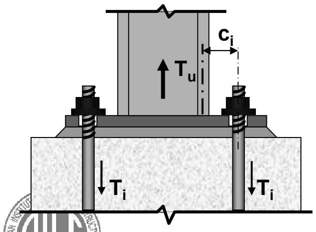
λ = conservatively 1

Refer to AISC Sec. 14 (Fig. 14-3)



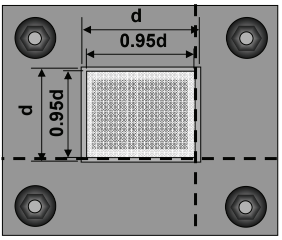
W-shape col.

Critical section for bending capacity of plate



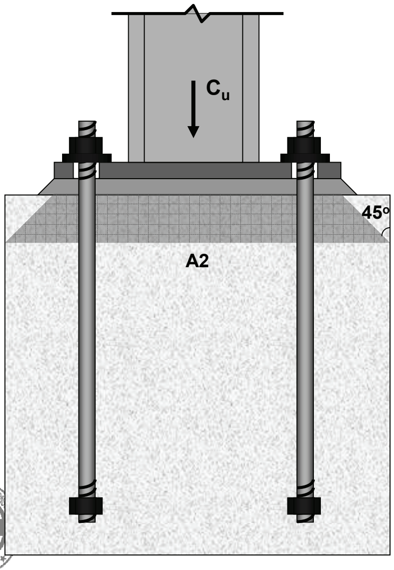
$M_u = n \times T_i \times c_i$

n = No. of anchors outside of the critical plane



Box-shape col.

Concrete Bearing Capacity (Compression)

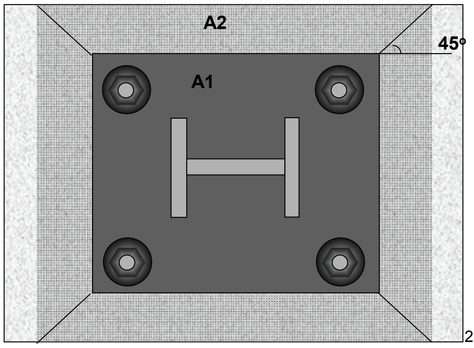


- Concrete bearing capacity (ϕP_n)

$$= \phi \times 0.85f'_c \times A_1 \times \sqrt{(A_2/A_1)}$$

$$(\phi = 0.65,\ \sqrt{(A_2/A_1)} \le 2.0)$$

- Min. number of anchor rods required for OSHA safety standards for steel erection

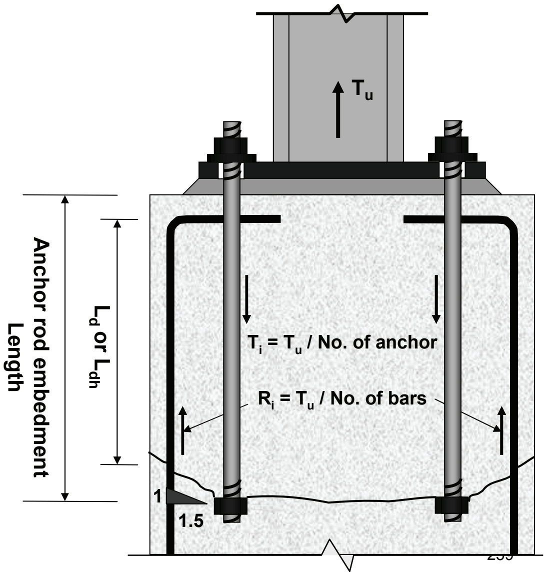


Mechanism to Transfer Tension Force to Base

- ACI 318-02 Appendix D:
  - Mainly specify the anchor strength based on concrete breakout capacity
  - Concrete breakout capacity by ACI specification is valid only for anchors with diameters not exceeding 2 in., and tensile embedment length not exceeding 25 in. in depth
  - Section D.4.2.1 allows to include the effect of supplementary reinforcement provided to confine the concrete breakout failure. However, it does not provide any specific guideline to assess the effect of reinforcement
- In general, when piers are used, concrete breakout capacity alone cannot transfer the significant level of tensile forces from steel column to the concrete base

Anchor Rod Design (Tension)

- The column force should be transferred to concrete base by properly lapping the anchors to the reinforcement in the base.
- Base plate is designed for a moment due to tension T_i in anchor rods located outside of the critical section
- Anchor rod embedment length is critical in transferring the tension to reinforcing
- Anchor Embedment Length = Top cover to reinforcing + L_d or L_{dh} (if hooked) + 0.75 times distance from rod to rebar but not less than 17 times rod diameter



Column Base Subjected to Axial Load and Shear

- Several valid mechanisms for transferring shear
  - Shear strength of anchor rod
  - Friction
    - Friction under applied loads
    - Friction under clamping force (shear friction)
  - Bearing on column or shear lug
- Cannot combine strength from different mechanism as peak resistance occur for different mechanism at different slip or deformation level
- “Shear friction” and “Shear strength of anchor rod” are commonly used to transfer shear
- However, ACI 318-02 Appendix D addresses shear transfer only through anchor rod shear

Shear Resistance through Friction/Shear Friction

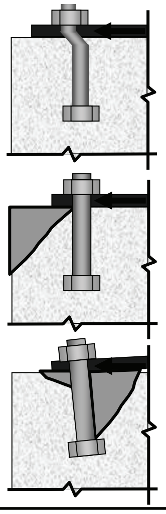
- **<u>For non-seismic condition:</u>**
  - $\phi V_n = \phi \mu P_u$

    μ = **0.55 for steel on grout**

    = 0.7 for steel on concrete
  - **When friction resistance under column load is less than applied factored shear force, a small movement will probably occur at ultimate load**
  - **Shear friction based on clamping force instead of column axial load can resist the applied shear. Anchor rods have to be designed for additional demand.**
- **<u>Seismic condition:</u>**
  - **For seismic condition, column load should not be used to develop shear resistance by friction, use shear friction based on clamping force**

Shear Resistance by ACI 318-02

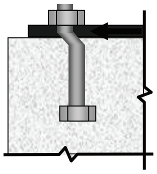
- **V_n according to ACI 318-02 Appendix D limited by:**



  - **Steel strength of anchor in shear**
  - **Concrete breakout strength of anchor in shear**
  - **Concrete pryout strength of anchor in shear**

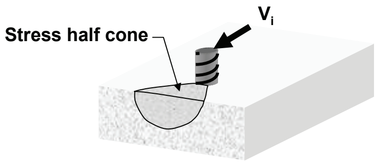
Shear Resistance by ACI 318-02

- Steel strength of anchor in shear
  - For cast-in headed stud anchors: $V_s = nA_{se}f_{ut}$ X 0.6
    For cast-in headed bolt and hooked bolt anchors: $V_s = nA_{se}f_{ut}$ x 0.6
    For post-installed anchors: $V_s = n(A_{se}f_{ut}$ x 0.6 + $A_{sl}f_{utsl}$ x 0.4)
    - n = number of anchors
    - A_{se} = effective cross sectional area
    - f_{ut} = specified tensile strength of anchor
    - A_{sl} = effective cross sectional area of sleeve
    - f_{utsl} = specified tensile strength of anchor sleeve
  - Where $f_{ut} < 1.9f_y$ or 125 ksi
  - 20% reduction in shear capacity for anchors with built-up grout pads

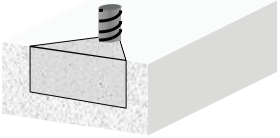


Shear Resistance by ACI 318-02

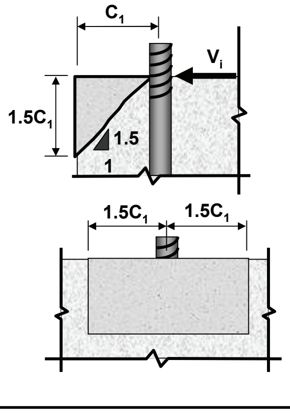
- Concrete breakout strength of anchor in shear
  - Refer to ACI for calculation details
  - If this limit controls, reinforcement should be provided to prevent the concrete breakout failure



Concrete breakout cone for shear

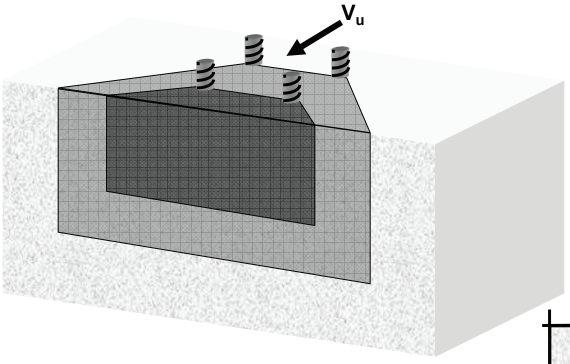


Idealized concrete breakout cone

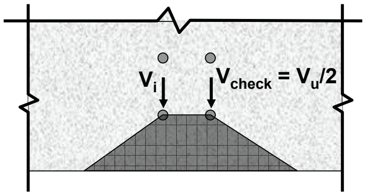


* Refer to ACI318-02 Appendix D for more information

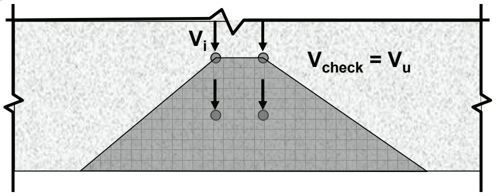
Concrete Breakout Failure (Anchor Group)



<u>Multi layer of anchor rods subjected to shear</u>



Concrete failure by outer layer of anchors

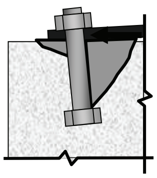


Concrete failure by inner layer of anchors

- Concrete breakout strength can be calculated according to ACI 318-02 Appendix D

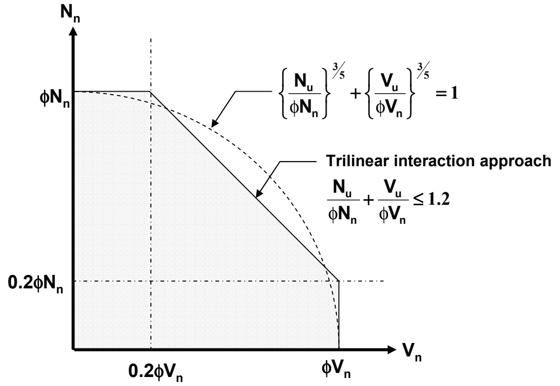
Shear Resistance by ACI 318-02

- Concrete pryout strength of anchor in shear
  - $V_{cp} = k_{cp}N_{cb}$ (k_{cp} = 1.0 for h_{ef} < 2.5 in., and k_{cp} = 2.0 for $h_{ef} \geq$ 2.5 in.)
    - N_{cb} = nominal concrete breakout strength in tension of a single anchor
    - h_{ef} = effective anchor embedment length
  - Will assure proper length of anchors
  - When adequate embedment length of anchor rods is provided or reinforcement is lapped as discussed in column base subjected to tension, this limit state will not control



Interaction of Tensile and Shear Forces by ACI

- ACI 318-02 Appendix D
  - If $V_u \le 0.2\phi V_n$ or $N_u \le 0.2\phi N_n$, interaction needs not to be considered.
  - Otherwise, trilinear interaction approach shall be satisfied.

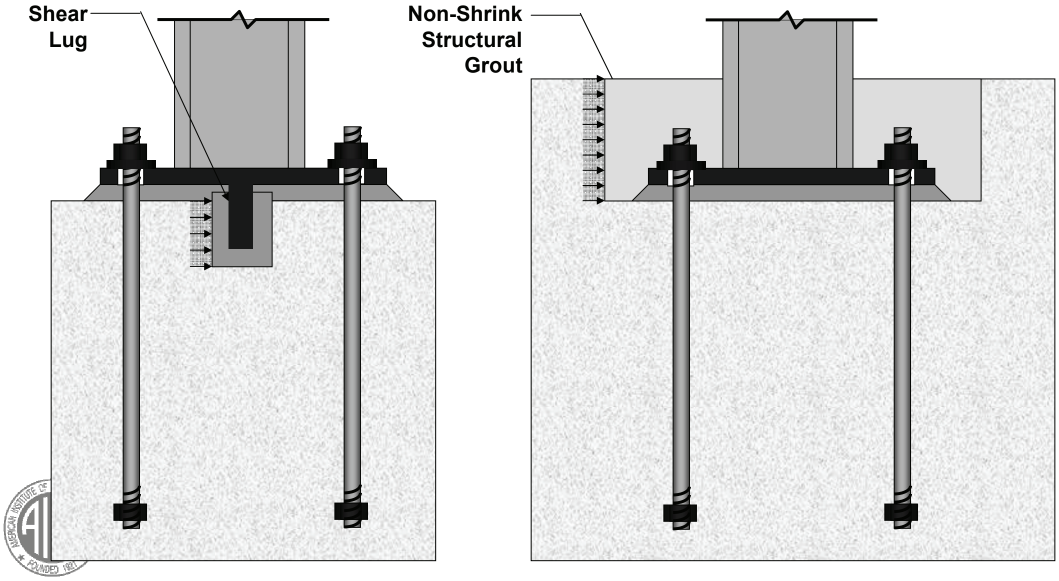


$$\left\{\frac{N_u}{\phi N_n}\right\}^{5/3} + \left\{\frac{V_u}{\phi V_n}\right\}^{5/3} = 1$$

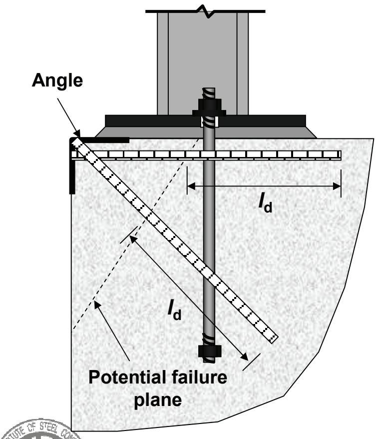
$$\frac{N_u}{\phi N_n} + \frac{V_u}{\phi V_n} \le 1.2$$

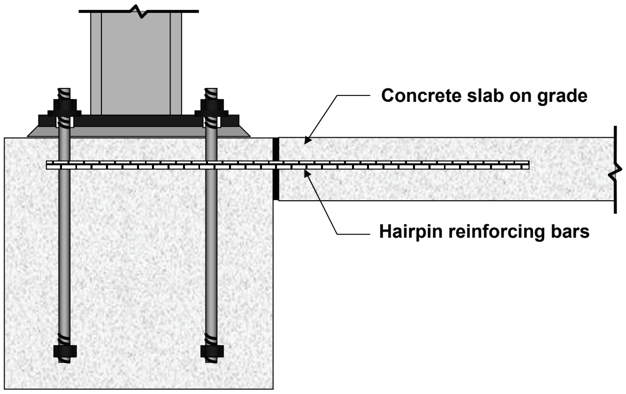
When concrete breakout strength and pryout strength for shear resistance does not control, "ACI 318-02" approach becomes equivalent to "shear-friction approach"

Shear Resistance through Bearing

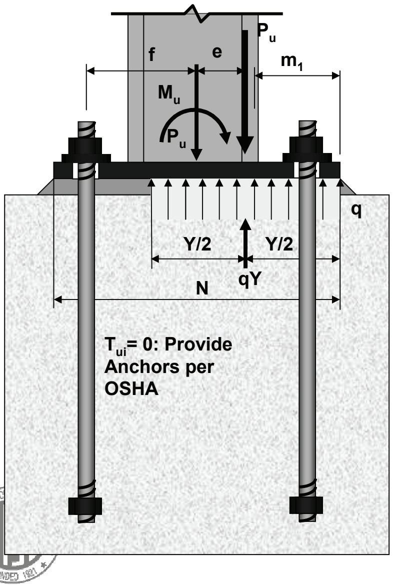


Reinforcing against Concrete Breakout (Shear)



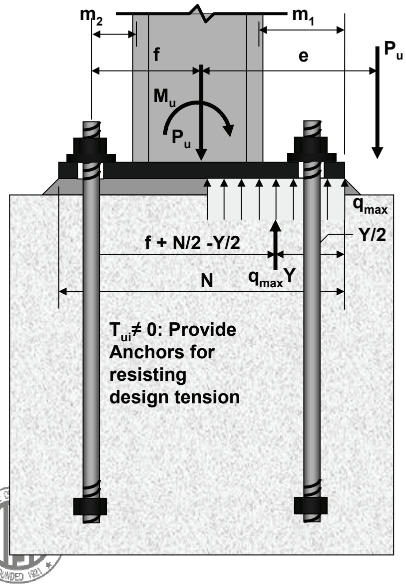


Column Base subjected to Small Moment



- Small moment, so $T_{ui} = 0$:
  - $0 < e\ (= {}^{M_u}/_{P_u}) \leq {}^{N}/_{2} - {}^{P_u}/_{2q}$
- Concrete bearing capacity (ϕP_n)
  - $P_u \leq q_{max}Y$
    - $Y = N - 2e$
    - q_{max} (per in.) = $\phi_c \times 0.85f'_c \times B \times \sqrt{(A_2/A_1)}$
      - $\phi_c = 0.65$
      - $A_1 = B \times Y$
      - $\sqrt{(A_2/A_1)}$ can be taken as 1.0
- Base plate yielding limit
  - At bearing interface
    - $q = P_u/ Y$
    - $M_{pl} = q \times (m_1^2/2) \leq \phi M_{n,plate}$
      - $\phi M_{n,plate} = 0.9 \times t_{pl}^2/4$
  - At tension interface: No tension force

Column Base subjected to Large Moment

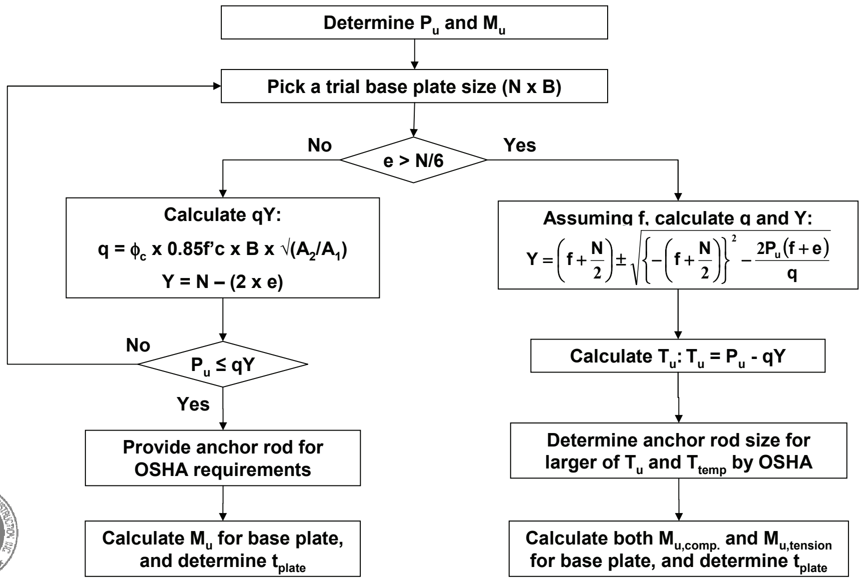


- Large moment, so $T_{ui} \neq 0$:
  - $e\ (= {}^{M_u}/_{P_u}) > {}^{N}/_{2} - {}^{P_u}/_{2q_{max}}$
- Concrete bearing capacity (ϕP_n)
  - $P_u + T_u \leq q_{max}Y$
    - q_{max} (per in.) = $\phi_c \times 0.85f'_c \times B \times \sqrt{(A_2/A_1)}$
    - $Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left\{-\left(f + \frac{N}{2}\right)\right\}^2 - \frac{2P_u(f+e)}{q_{max}}}$,

      By retaining only negative sign of the root, $Y = \frac{P_u}{q_{max}}$
    - Then $T_u = q_{max}Y - P_u$

      ($T_{ui} = T_u$/No. of anchor rods at tension side)
- Base plate yielding limit
  - At bearing interface
    - $M_{u,comp.} = q_{max} \times (m_1^2/2) \leq \phi M_{n,plate}$
  - At tension interface
    - $M_{u,tension} = T_u \times m_2 \leq \phi M_{n,plate}$

Design Procedure For Column Base subjected to Moment



Determine P_u and M_u

Pick a trial base plate size (N x B)

e > N/6

No

Calculate qY:

$q = \phi_c \times 0.85f'c \times B \times \sqrt{(A_2/A_1)}$

$Y = N - (2 \times e)$

$P_u \leq qY$

No

Yes

Provide anchor rod for OSHA requirements

Calculate M_u for base plate, and determine t_{plate}

Yes

Assuming f, calculate q and Y:

$Y = \left(f + \frac{N}{2}\right) \pm \sqrt{\left\{-\left(f + \frac{N}{2}\right)\right\}^2 - \frac{2P_u(f+e)}{q}}$

Calculate T_u: $T_u = P_u - qY$

Determine anchor rod size for larger of T_u and T_{temp} by OSHA

Calculate both $M_{u,comp.}$ and $M_{u,tension}$ for base plate, and determine t_{plate}

Avoiding Common Construction Problems

- Use a qualified field engineer to layout the anchor rods
- Use AISC-recommended hole sizes in the base plates
  - Table
- Use symmetric patterns for the anchor rods
- Use wood or steel templates firmly fastened to the footing or pier forms

Solving Misplaced Anchors Problem

Available options

- Evaluate the need for the incorrectly located rods; perhaps not all of them are required.
- Cut rods and use epoxy anchors
- Make larger hole and use plate washers
- Fabricate a new base plate (Moment must be considered)
- Relocate the column on the base plate (Moment must be considered)
- Bend the rods into position. This could require chipping of concrete

Avoiding Short Anchor Extensions

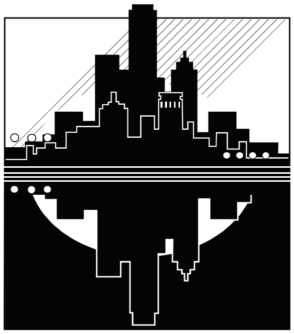
- Provide a design with ample length, and ample thread length
- If possible, do not use high-strength-steel anchor rods, use larger-diameter rods
- Specify supplemental specification for weldable anchors (not available for grade 105)

Anchors too short

Available options

- Extend a short anchor rod by welding on a threaded extension. First check if the anchor rod material is weldable
- Use a coupling nut to extend the rod
- Cut the rod(s) and use epoxy anchors
- Weld the base plate to the rods (not high for strength rods): A plate washer can be slipped over the anchor rod so that the anchor rod can be welded to the plate washer
- Perform analysis for nut using the threads engaged: This can be done based on a linear interpolation of full threads engaged, versus the number of threads in the nut

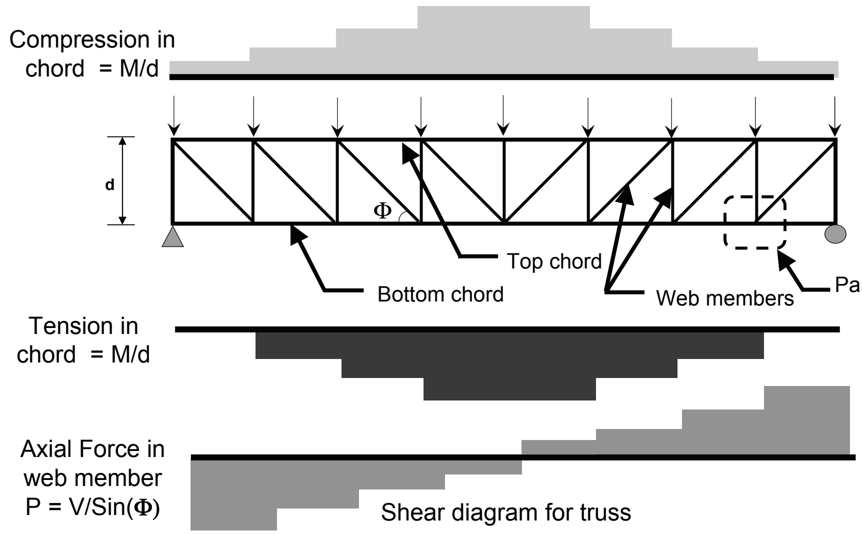
Steel Trusses



Overview

- Types and anatomy
- Strategies in geometry
- Member selection
- Connections
- Practical tips

Truss Anatomy



Compression in chord = M/d

d

Φ

Top chord

Bottom chord

Web members

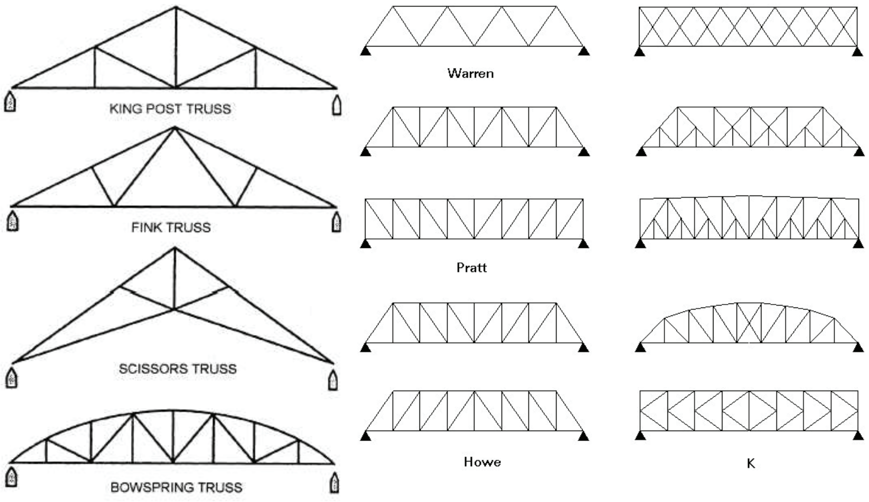
Panel point

Tension in chord = M/d

Axial Force in web member P = V/Sin(Φ)

Shear diagram for truss

Types of Trusses



Strategies for Geometry

- Depth
    - Too small depth → Large axial forces in chords
    - Too large depth → Long web members → Tonnage increase from additional length as well as higher KL/r
    - Optimal span to depth ratio = 10 to 12
    - Constant depth
        - Change chord member sizes along span (economical)
        - Simplified connections
        - Less number of chord splices and therefore less connection material
        - Depth optimized only at mid-span and therefore, diagonal members become unnecessarily long away from mid span
    - Variable depth
        - Depth follows moment diagram / top chord generally slopes
        - Chord forces stays nearly constant
        - Segments require chord splices
    - Architectural requirements often governs truss geometry

Strategies for Geometry

- Panel point spacing
    - Primarily governed by location of loads (purlin spacing)
    - Try to maintain diagonals at 45 degrees
    - When closer purlin spacing is required (limited by spanning capability of floor/deck), consider using sub posts
    - Panel points also offer in-plane bracing to chord members; spacing should be such that efficient use of chord material is achieved

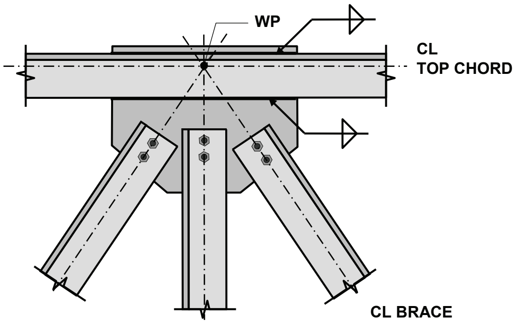
Member Design

- Keep focus on connection while selecting member types
- Keep focus on connection while selecting orientation of member
- Net section → Member can not be utilized for its full tension capacity without member end supplemental plates
- Moments in chord member must be considered when diagonal work point away from chord centerline
- Secondary moments in chord can be ignored if design is based on KL = distance between panel point (K = 1)
- If load exists between panel points, moments must be considered → may affect member selection

---

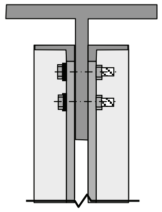
Member Types and Connections

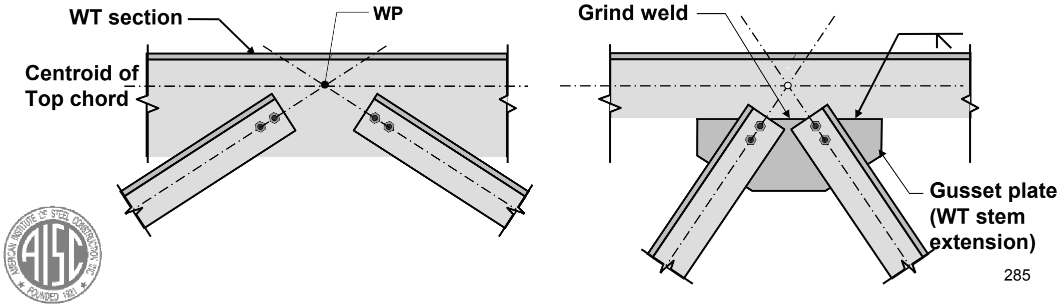
- Double angles
  - Simple connections with gusset plate
  - Not suitable for long span as KL/r will be very high → less efficient use of material
  - Primarily used as chord material in joists and joist girders
  - A36 preferred material



Member Types and Connections

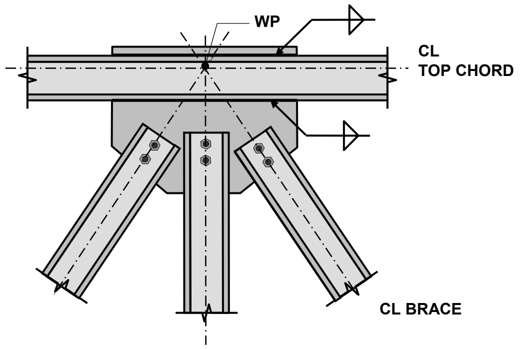
- WT chords with angles as web members
- Simple connections with/without gusset plate
- Not suitable for long span as KL/r will be very high → less efficient use of material





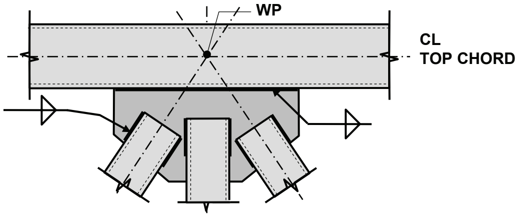
Member Types and Connections

- Double channels
  - Simple connections with gusset plate
  - Limited selection of channel shapes

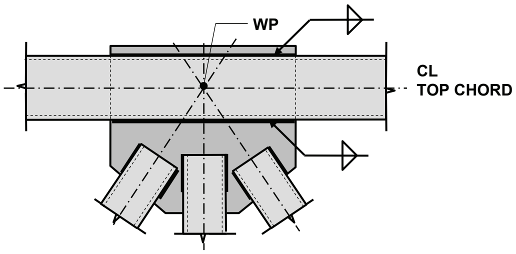


Member Types and Connections

- HSS members
- Variety of connection possibilities
  - Welded
    - Ratio of widths important
    - Punching of supporting wall
  - Single gusset plate
    - Through
    - Wall mounted
  - Double gusset plates
- Efficient for long unbraced lengths as LTB is generally not an issue



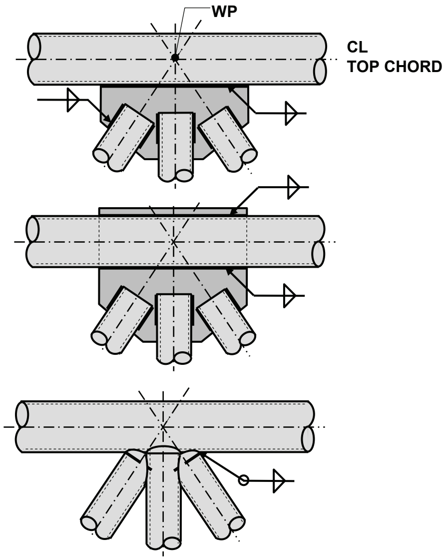
Wall mounted gusset plate



Through gusset plate

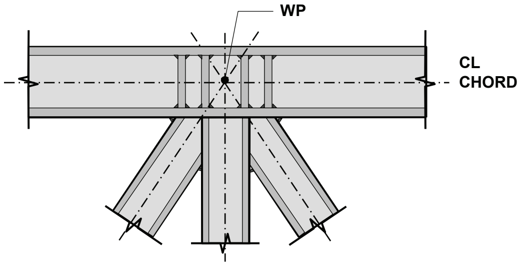
Member Types and Connections

- Round HSS members
- Aesthetically pleasant
- Contoured welded connections difficult to fabricate and expensive
- Design guidelines available only for simple connections
- Welding process requires highly skilled personnel



Member Types and Connections

- Wide flange shapes
  - Primarily used for long spans
  - Large selection of shapes (W14x43 – 730)
  - Several connection possibilities
  - Special consideration required for restrained conditions developed for welds

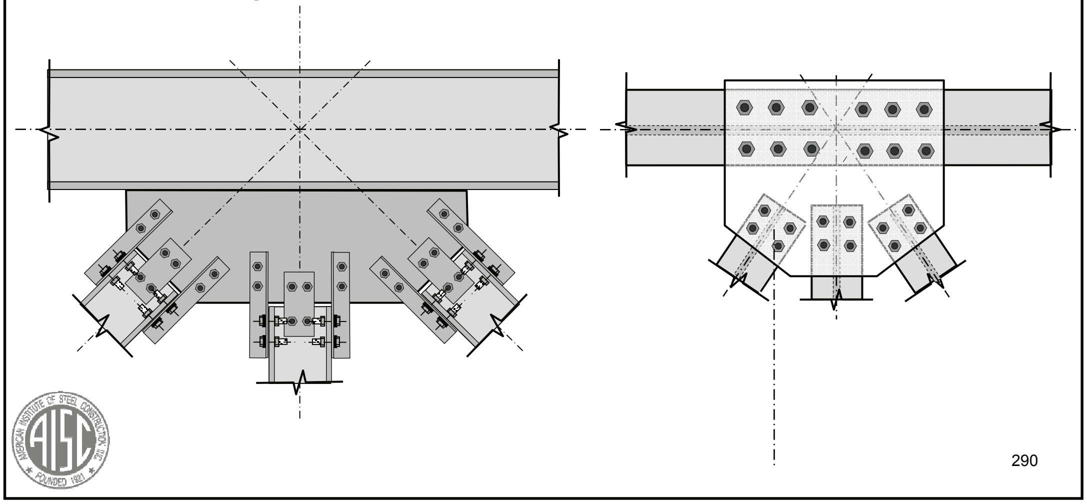


All Welded Connection

(ship Shop fabricated modules and field bolted splice)

Member Types and Connections

- Wide flange shapes: other connection possibilities



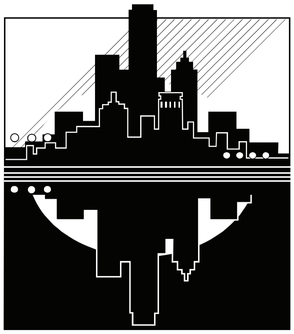
Practical Tips for Truss Design

- Consider connection design in the member selection/design process
- Engage fabricator/erector in the design process as early as possible
- Consider the weight of connection in the design
  - Wide flange connection allowance can be as high as 20% of the weight of truss members
- Try to use filed bolted connections and limit welding to shop welding only
- For repetitive trusses, consider grouping member sizes; The additional tonnage may offset the savings in detailing and fabrication
- Document detailed erection plan and assumptions made in the design
- Design and detail connections on construction documents. If not, clearly show the design forces including through forces
- Envelop of forces may not be sufficient for independent engineer to design connections
- Study the camber requirements and show on construction documents clearly

Practical Tips for Truss Design

- Bracing member forces from computer analysis may not proper when tolerances/out of straightness not considered in analytical model → Design such members and their connections for required minimum bracing force
- For roof trusses, verify if net uplift is possible or not and properly brace bottom chord under compression
- When story deep trusses are used, consider their participation with lateral load resisting system
- Consider using slip critical bolts in oversize holes to alleviate fit up problems. When bearing bolts in standard holes are used, consider shop fitting
- For long spans, consider using higher strength material for reducing member sizes and tonnage

Computer Modeling and Verification



Modeling Issues

- General checks
  - Make sure that units are correct throughout the model → geometry, section properties, material, loads
  - View deflected shape of structure → many problems are visually noticeable
  - Perform second order analysis or consider P-Delta analysis using alternative methods
  - Consider performing modal analysis of complex structures → many modeling errors will be caught under low frequency modes
  - Model material properties that you specify on contract documents (A36 VS A572)
  - Don’t forget about serviceability issues
  - Check stability and drift at all levels
  - ALWAYS check equilibrium of structure

Modeling Issues

- **Boundary conditions**
  - Model pinned base when rotational stiffness at the base is less than EI/2L of column
  - Model fixed base when rotational stiffness at the base is greater than 18EI/L
  - Spring stiffness can be obtained using PCI method
  - Do not casually model roller support; even Teflon coated slide bearings offer coefficient of friction of 5%
  - Do not model truss type structures with horizontal restraints at each support
  - When structure seem too sensitive to the assumed boundary condition, consider to bound the solution
  - Pay special attention to basements in high-rise buildings → Basement walls may act as shear wall generating very high demand on diaphragm or floor slab
  - Use effective lengths of compression member in design consistent with the boundary conditions

Modeling Issues

- **Member modeling**
  - Use material properties that are consistent with contract documents
  - Consider modeling HSS members with properties using 0.93 * nominal thickness
  - When rigid end offsets are used, consider the deformations that occur inside of the beam column joints
  - Member end release should reflect the connection fixity/flexibility
    - Do not release moment at the end of member and provide moment connection
    - Model how you are going to detail ←→ Detail how you modeled

Modeling Issues

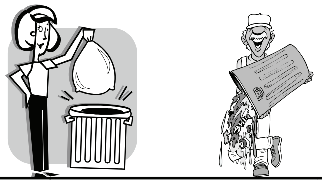
- Diaphragm and master slave
  - Verify the assumption of rigid diaphragm before modeling
  - Rigid diaphragm (floor master – slave) modeling results zero axial force in members within the diaphragm → Design of brace beams require manual force determination
  - Verify the strength of connection (studs or deck attachment)
  - Check of connection between diaphragm and brace beam can transfer the story load or not → When required, collector beam may be required → Computer program will not be able to make decision on such issues
  - Through force must be indicated on construction document unless connection design is performed by EOR

Modeling Issues

- Loading
  - Consider eccentricity in wind as well as seismic loading
  - Response spectrum analysis results do not carry signs (tension VS compression) → take special care in designing connections and transfer forces
  - Use response modification factor consistent with the detailing of connections, member design criteria
  - Do not use conservative estimate of design loads when combinations for overturning (net uplift) are considered
  - Make allowance for connections in self-weight calculations
  - When only lateral load resisting system is modeled, consider the loading on leaning system for stability purpose
  - When exposed to whether, consider thermal loading

Modeling Issues

- Know default values considered by the design programs and change them as required
  - Codes: ASD or LRFD → Consistent with load combinations
  - Un-braced length → Default may be full length without modeling of deck
  - Effective length of compression members → Default may be braced frame (non sway frame) where as model may be for moment frame (sway frame)
  - Material strength → default may be 50 ksi requiring new definition for HSS or A36



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