PDHonline Course S214 (4 PDH)

Steel Beam Design

Instructor: Matthew Stuart, PE, SE

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5272 Meadow Estates Drive
Fairfax, VA 22030-6658
Phone: 703-988-0088
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Steel Beam Design

D. Matthew Stuart, P.E., S.E., F.ASCE, SECB

COURSE CONTENT

Review of Load & Resistance Factor Design (LRFD)

LRFD: Also known as strength design, limit state design and ultimate load design. The method was first adopted by AISC in 1986. The material presented in this course is based on the latest AISC Steel Construction Manual, 13^{th} Edition, and the IBC 2006

Design Concepts:

a. LRFD accounts for the non-linear nature of the stress-strain diagram of steel subjected to high stress levels, therefore the method provides a better representation of the behavior of steel, steel-concrete composite members and members subjected to seismic loads.

b. LRFD accounts for the greater certainty of the determination of dead loads and the less predictable nature of live loads by specifying higher load factors for live loads.

c. LRFD accounts for the variability in the nominal strength of different types of members by applying different values of resistance factors thereby achieving more uniform reliability for different member types.

LRFD uses probability theory to establish an acceptable margin of safety based on the variability of the anticipated loads and member strength, therefore to assure structural safety:

$$R_u \leq \Phi R_n \qquad \text{(Equation B3-1, AISC } 13^{th} \text{ Edition)}$$

Where: R_u = Required Ultimate Strength = Factored Load Combination = $\sum \gamma Q$

ΦR_n = Design Strength = Available Strength

R_n = Nominal Strength = Member Capacity

Φ = Resistance Factor

Q = Load Effect produced by Service Load

γ = Load Factor

Resistance Factors (Φ) for LRFD are:

Resistance Factor	Flexural Bending	Web Shear (I-Beams)	Shear	Compression	Tensile (Yielding)	Tensile (Fracture)
Φ	0.90	1.00	0.90	0.90	0.90	0.75

I-Beams: W, M and S shapes

Factored loads consist of working, or service, loads multiplied by the appropriate load factors, as required to account for the inherent uncertainty of the respective load. Per Section 1605.2.1 of the IBC the *Required Strength* ($\sum\gamma Q$) is defined by seven load combinations:

During Construction:

$\sum\gamma Q = 1.4D$

During Maximum Occupancy Live Loads:

$\sum\gamma Q = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$

During Maximum Roof Live Load, Rainwater or Snow Load:

$\sum\gamma Q = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L^* \text{ or } 0.8W)$

During Maximum Wind Loads that increase the effects of the Dead Load:

$\sum\gamma Q = 1.2D + 1.6W + 0.5L^* + 0.5(L_r \text{ or } S \text{ or } R)$

During Maximum Seismic Loads that increase the effects of the Dead Load:

$\sum\gamma Q = 1.2D \pm 1.0E + 0.5L^* + 0.2S^{**}$

During Maximum Wind Loads that oppose the effects of the Dead Load:

$\sum\gamma Q = 0.9D + 1.6W + 1.6H$

During Maximum Seismic Loads that oppose the effects of the Dead Load:

$\sum\gamma Q = 0.9D + 1.0E + 1.6H$

Where: D = Dead Load

L = Occupancy Live Load

Lr = Roof Live Load

S = Snow Load

R = Load due to Rainwater or Ice

W = Wind Load

E = Earthquake Load

H = Loads due to Lateral Earth, Ground Water or Bulk Material Pressures

* Use 1.0L for Garages, Places of Public Occupancy or areas where L > 100 PSF

** Use 0.7S for Roof Configurations that do not shed Snow

Review of Allowable Strength Design (ASD)

ASD: The original method of design used for steel structures was referred to as Allowable Stress Design. This is because the actual elastic stress, for example $f_b = M_s/S$ (where; M_s = Service Moment and S = Section Modulus), was compared to a maximum allowable stress (i.e. F_b). Therefore the maximum allowable M_s for any given section = F_bS. The current version of ASD found in the 13th Edition, however, is a strength based method using the plastic section modulus, Z. Therefore; $M_s \leq (F_yZ)/\Omega$.

Although the actual percentage of engineers still using ASD is not known, the fact that AISC published the 13th Edition (2005) of the Steel Construction Manual with provisions for ASD in light of the fact that the last previous ASD manual (9th Edition) was published in 1989 would indicate that the method is still in wide spread use. Although the continuation of the use of ASD may be attributed in some degree to practicing engineers handing the method down from generation to generation (even though only LRFD is typically taught in most engineering schools) the fact does remain, in my opinion, that it is a simpler method of design and cannot be superseded by LRFD for conditions in which serviceability (i.e. deflection) controls the design. The material presented in this course is based on the latest AISC Steel Construction Manual, 13th Edition, and the IBC 2006

ASD is a design method by which the members are sized such that the allowable strength is not less than the *Required Strength*, therefore to assure structural safety:

$$R_a \leq R_n/\Omega \qquad \text{(Equation B3-2, AISC 13th Edition)}$$

Where: R_a = Required Strength (as determined by ASD load combinations)

R_n/Ω = Allowable Strength = Available Strength

R_n = Calculated Nominal Strength of Member = Member Capacity

Ω = Safety Factor

Safety Factors (Ω) for ASD are:

Safety Factor	Flexure Bending	Web Shear (I-Beams)	Shear	Compression	Tensile (Yielding)	Tensile (Fracture)
Ω	1.67	1.50	1.67	1.67	1.67	2.00

To convert the strength formulas in the 13th Edition back to the equivalent 9th Edition provisions, the nominal strength values can be divided by the safety factor (Ω) given in each design specification chapter provision. This arrangement is also evident in all of the 13th Edition design tables, charts and nomographs where the nominal strength value is shown divided by Ω (which are also typically highlighted in green and labeled as ASD).

The *Required Strength* ($\sum Q$) for ASD is defined for eight load combinations in IBC Section 1605.3.1. No increase in allowable stress is permitted with these same load combinations.

During Construction:

$\sum Q = D + F$

During Maximum Occupancy Live Loads:

$\sum Q = D + H + F + L + T$

During Maximum Roof Live Load, Rainwater or Snow Load:

$\sum Q = D + H + F + (L_r \text{ or } S \text{ or } R)$

During Maximum Occupancy Live Loads, Maximum Roof Live Load, Rainwater or Snow Load:

$\sum Q = D + H + F + 0.75(L + T) + 0.75(L_r \text{ or } S \text{ or } R)$

During Maximum Wind Loads or Seismic Loads that increase the effects of the Dead Load:

$\sum Q = D + H + F + (W \text{ or } 0.7E)$

During Maximum Occupancy Live Loads, Maximum Roof Live Load, Rainwater or Snow Load and Maximum Wind Loads or Seismic Loads that increase the effects of the Dead Load:

$\sum Q = D + H + F + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$

During Maximum Wind Loads that oppose the effects of the Dead Load:

$\sum Q = 0.6D + W + H$

During Maximum Seismic Loads that oppose the effects of the Dead Load:

$\sum Q = 0.6D + 0.7E + H$

Where: D = Dead Load

L = Occupancy Live Load

Lr = Roof Live Load

S = Snow Load

R = Load due to Rainwater or Ice

W = Wind Load

E = Earthquake Load

H = Loads due to Lateral Earth, Ground Water or Bulk Material Pressures

F = Loads due to Fluids with well defined pressures and maximum heights

T = Self-Straining Force arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep, differential settlement or combinations thereof

Designing for Flexural Bending

The application of a gradually increasing uniform bending moment applied to a laterally braced steel beam produces the idealized moment/displacement relationship shown in Figure 1a; where M_y is the bending moment at the point of yielding (i.e. F_yS; where S = the Section Modulus) and M_p is the plastic moment (i.e. F_yZ, where Z = the Plastic Section Modulus). The applied moment at first yielding is $M_r = 0.70F_yS$. The shape factor for any section is defined as M_p/M_y or Z/S, which ranges from 1.1 to 1.3 for wide flange sections. The ratio M_p/M_y can be thought of as a measure of the reserve strength of a section. The corresponding stress distribution in a wide flange beam (ignoring the effects of residual stress) for the critical ranges noted in the deflection curve shown in Figure 1a is illustrated in Figure 1b.

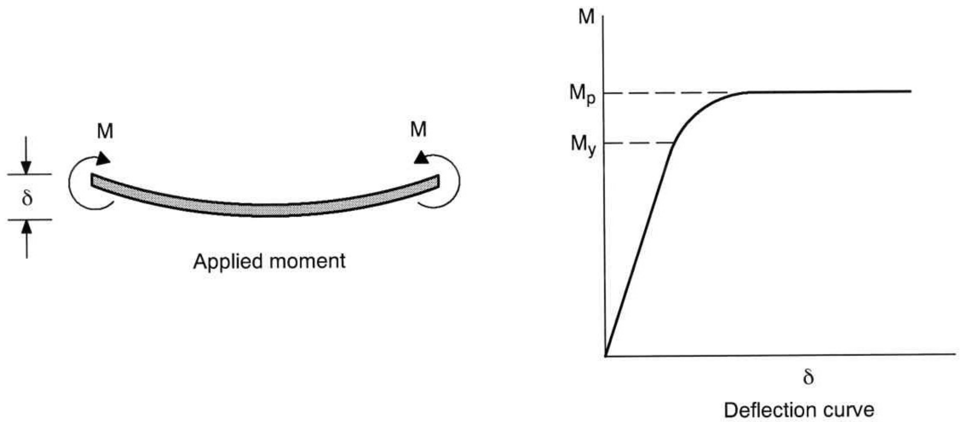


FIGURE 1a

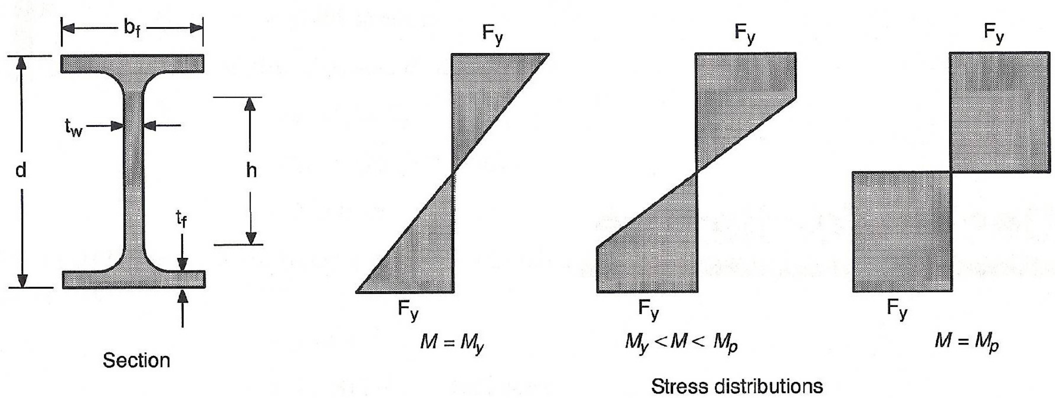


FIGURE 1b

The initial portion of the graph is linear as the stress in the extreme fibers approaches the yield stress (assuming that the compression face of the beam is adequately restrained to prevent lateral torsional buckling, and provided that local flange buckling or local web buckling do not occur).

Lateral Torsional Buckling:

When a beam bends, the compression region behaves similar to a column in that it will buckle if the member is slender enough. Unlike a column, however, the compression region of the cross section is restrained by the tension portion of the same member. The resulting transverse deflection of the compression flange (i.e. Flexural Buckling – deflection about the axis corresponding to the largest slenderness ratio, which for a wide flange section is usually the minor axis – the axis with the smallest radius of gyration) is therefore accompanied by twisting (torsion) of the beam which results in a form of instability called Lateral Torsional Buckling (LTB). LTB can be prevented by bracing the beam against twisting at sufficiently close intervals using either lateral bracing or torsional bracing (see Figure 2).

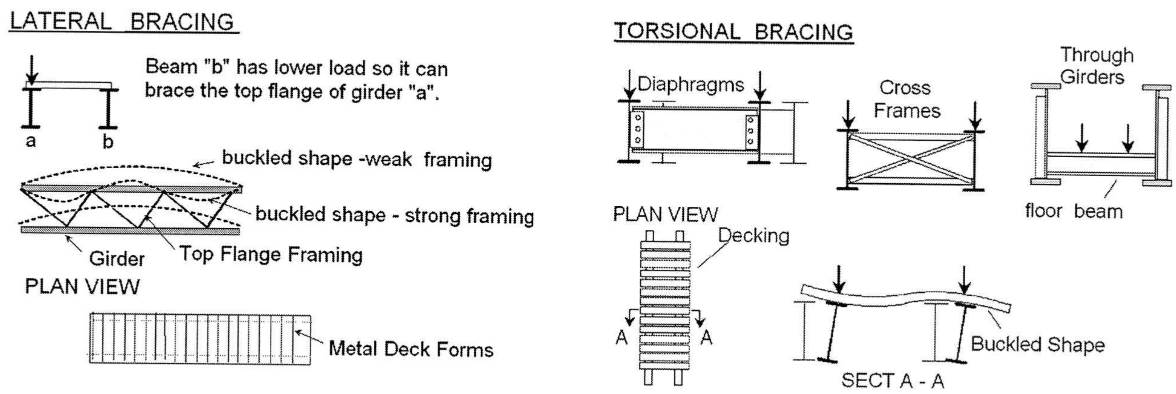


FIGURE 2

Lateral bracing is best located when situated to prevent twisting of the beam. For a simple span beam this would be at the top or compression flange, for a cantilever beam this would be the top or tension flange (see Figure 3).

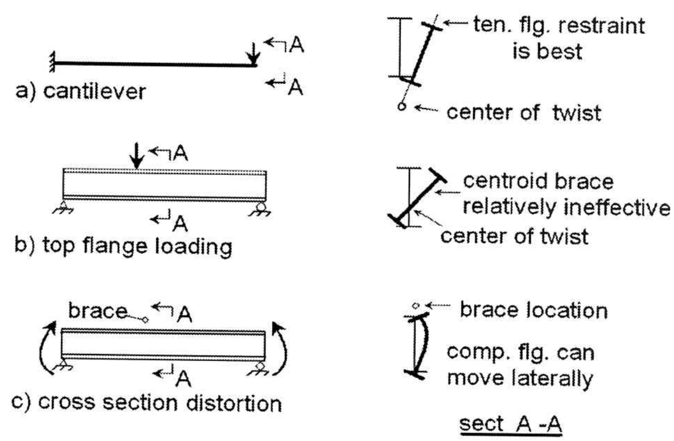


FIGURE 3

For a continuous beam, it is incorrect to assume that an inflection point is a brace point. In fact bracing requirements for a continuous beam are greater than for a beam in single curvature. Also, for a cantilever beam of L length in which no transverse beams or bracing are provided beyond the fixed end, $L_p = L$. For a further discussion of beam bracing see *Fundamentals of Beam Bracing*, AISC Engineering Journal, 1st Quarter 2001, Joseph A. Yura.

Local Flange Buckling and Local Web Buckling:

Whether a beam can sustain a moment large enough to bring it to a fully plastic condition also depends on the ability of the member to maintain local cross-sectional integrity. This integrity is lost if one of the compression elements (i.e. the unstiffened element - outboard flange edge, and the stiffened element – portion of the web in compression) buckles. This type of buckling is referred to as Local Flange Buckling (LFB) and Local Web Buckling (LWB), respectively. Whether either type of buckling occurs depends on the width-thickness ratios of the compression elements of the cross section. Limiting values of width-thickness ratios are given in the 13th Edition Specifications, Chapter B, Section B4 (page 16.1-14, and page 16.1-223 of the Commentary).

By definition a Compact section is a member that can develop a plastic hinge prior to LFB or LWB, provided adequate bracing against LTB is provided. Most rolled sections qualify as compact sections based on the criteria defined in Table B4.1. In fact all rolled wide flange and channel sections have compact flanges for $F_y = 50$ ksi with the exception of W21x48, W14x99, W14x90, W12x65, W10x12, W8x31, W8x10, W6x15, W6x9 and W6x8.5 (…and M4x6). All rolled I-beams and channel sections also have compact webs for $F_y = 50$ ksi.

A Non-Compact section by definition is a member that can develop, prior to LFB or LWB, a nominal flexural strength (M_n) given by; $M_r \le M_n < M_p$

A Slender section by definition is a member that cannot develop a stress of Fy prior to LFB or LWB. The nominal moment strength of a slender section is given by Equation (F3-2) of the 13th Edition. There are no I-beams with slender flanges or webs.

The moment strength of a compact section is a function of the unbraced length (L_b) which is defined as the distance between points of lateral support or bracing. If L_b is no greater than L_p (the limiting laterally unbraced length for the yielding limit state) the beam is considered to have full lateral support and $M_n = M_p$. If L_b is greater than L_p but less than or equal to L_r (the limiting laterally unbraced length for the inelastic LTB limit state) then the full plasticity of the section cannot be achieved before LTB occurs and $M_n = M_r$. If L_b is greater than L_r then M_n is equal to the nominal flexural strength which is governed by elastic LTB. The following equations and Figure 4 summarize the above discussion.

$\mathbf{L_b < L_p}$; $M_n = M_p = F_y Z$

Where: $L_p = 1.76 r_y (E/F_y)^{1/2}$ [Equation (F2-5)]

$\mathbf{L_p < L_b \le L_r}$; $M_n = C_b(M_p - (M_p - 0.70F_yS)((L_b - L_p)/(L_r - L_p))) \le M_p$ [Equation (F2-2)]

Where: $Lr = 1.95 r_{ts}(E/0.70F_y)(Jc/(Sh_o))^{1/2}(1 + (1 + 6.76((0.70F_ySh_o)/EJc)^2)^{1/2})^{1/2}$ [Equation (F2-6)]

$c = 1.0$ for doubly symmetrical I-beam sections

h_o = distance between flange centroids = $d - t_f$

$r_{ts}^2 = (I_yC_w)^{1/2}/S$

C_w = warping constant = $(I_yh_o^2)/4$ for doubly symmetrical I-beam sections

C_b = (see following section)

$\mathbf{L_b > L_r}$; $M_n = F_{cr}S \le M_p$ [Equation (F2-3)]

Where: $F_{cr} = (C_b\pi^2E/(L_b/r_{ts})^2)(1 + 0.078(Jc/Sh_o)(L_b/r_{ts})^2)^{1/2}$ [Equation (F2-4)]

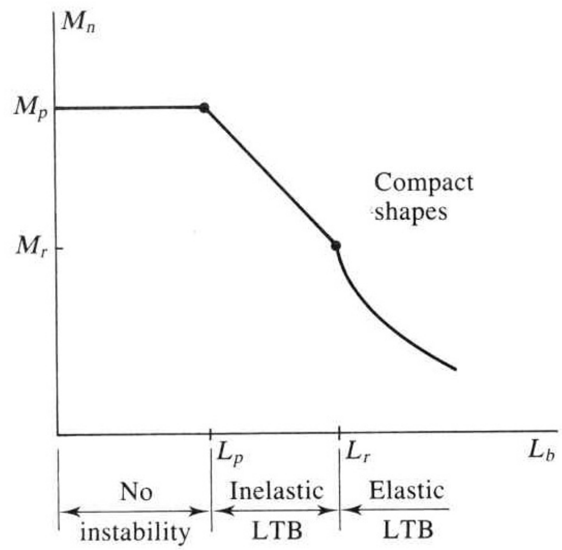


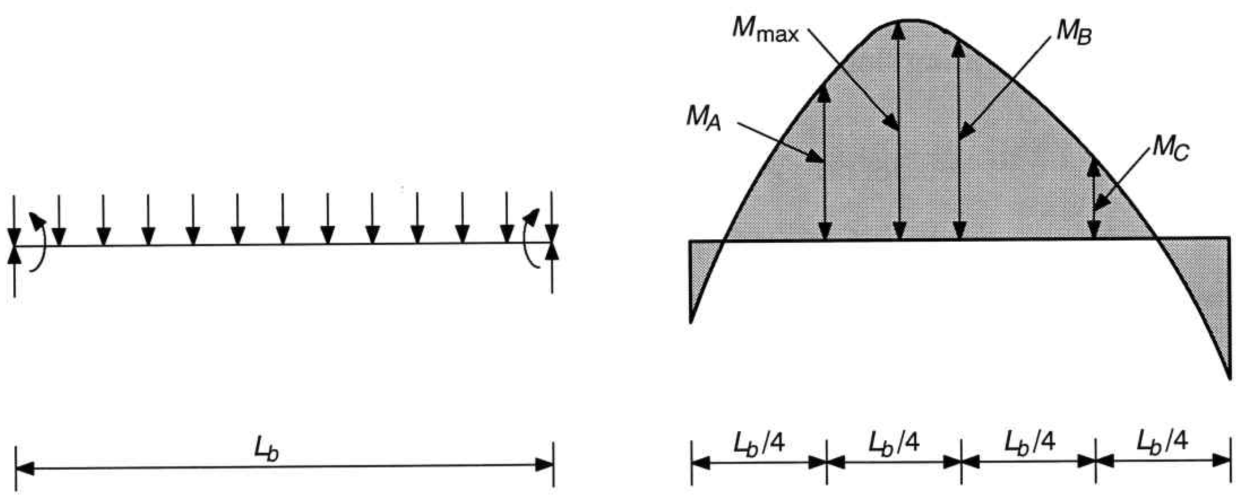
FIGURE 4

Table 3-2 of the 13th Edition tabulates all of the above information for 50 ksi wide flange sections. BF, or the Bending Factor = $\Phi_b(M_p – M_r)/(L_r – L_p)$.

Table 3-2 assumes $C_b = 1.0$, therefore in order to use for example the Available Moment vs. Unbraced Length curves provided in Table 3-10 of the 13th Edition (which are also based on $C_b = 1.0$) and account for the actual C_b (the Bending Coefficient as detailed below), you can divide $\Phi_b M_n$ by C_b before entering the curves to determine the required beam size for the given unbraced length.

The Bending Coefficient (C_b) is a factor that accounts for non-uniform bending within the unbraced length (L_b) of a beam. If the moment within the unbraced length is uniform (i.e. constant) there is no moment gradient and $C_b = 1.0$. If there is a moment gradient then per Equation (F1-1) of the 13th Edition:

$$C_b = 12.5M_{max}R_m/(2.5M_{max} + 3M_A + 4M_B + 3M_C) \leq 3.0$$



where: M_A = absolute value of the bending moment at the quarter point of an unbraced segment

M_B = absolute value of the bending moment at the centerline of an unbraced segment

M_C = absolute value of the bending moment at the three-quarter point of an unbraced segment

M_{max} = absolute value of the maximum bending moment in the unbraced segment

R_m = cross-section monosymmetry parameter

= 1.0 ... for doubly symmetric members

= 1.0 ... for singly symmetric members in single curvature

= $0.5 + 2(I_{yc}/I_y)$ ... for singly symmetric members in reverse curvature

I_{yc} = weak-axis moment of inertia of the compression flange ... for members in single curvature

= weak-axis moment of inertia of the smaller flange ... for members in double curvature

I_y = moment of inertia about the principal y-axis

Values of C_b for various loading and restraint conditions are illustrated in Table 3.1 of the 13th Edition. C_b for unbraced cantilever beams is 1.0. C_b is permitted to be conservatively taken as 1.0 for all cases.

Designing for Shear

The web of a wide flange beam is the primary resisting element for shear stresses. The area of the web (A_w) = dt_w. When the slenderness parameter (h/t_w; where h = clear distance between flanges less the fillet or corner radius for rolled shapes) of the web $\leq 1.10(k_vE/F_y)^{½}$ then:

$$V_n = 0.60F_yA_w$$

When $1.10(k_vE/F_y)^{½} < h/t_w \leq 1.37(k_vE/F_y)^{½}$ then: $V_n = 0.60F_yA_wC_v$

Where: $C_v = 1.10(k_vE/F_y)^{½}/(h/t_w)$

When $h/t_w > 1.37(k_vE/F_y)^{½}$ then: $V_n = 0.60F_yA_wC_v$

Where: $C_v = 1.51(k_vE)/(F_y(h/t_w)^2)$

$k_v = 5$ for unstiffened webs with $h/t_w < 260$ in all cases. Table 3-2 and 3-6 provide Φ_vV_n values for 50 ksi wide flange beams.

When concentrated loads are applied to beams or beam bearings, localized yielding from high compressive stresses can occur which is flowed by inelastic buckling in the web region adjacent to the toe of the fillet with the flange in the vicinity of the concentrated load. This combined yielding and inelastic behavior was formally described as web crippling. However, since compression related conditions can lead to two types of behavior, yielding and instability, the 13th Edition now recognizes three separate categories; local web yielding, web crippling and sidesway web buckling.

As shown in Figure 5, bearing plates are used to distribute concentrated loads applied to the beam flange so as to prevent local web yielding. Web yielding is a compressive crushing of a beam web caused by the application of compressive force to the flange directly above or below the web. Bearing plates are also used to distribute the bearing stresses on the member or support receiving the reaction. The load is assumed to be dispersed at a slope of 2.5 to 1.0 to the toe of the web fillet (k). For loads applied greater than d from the member end the nominal strength; $R_n = (5k + N)F_{yw}t_w$ [Equation J10-2]. For loads applied less than d from the end of the member; $R_n = (2.5k + N)F_{yw}t_w$ [Equation J10-3]. When the factored applied load exceeds ΦR_n (where $\Phi = 1.0$) either transverse stiffeners or a doubler plate is required with a minimum length equal to half the depth of the beam centered at the location of the concentrated load.

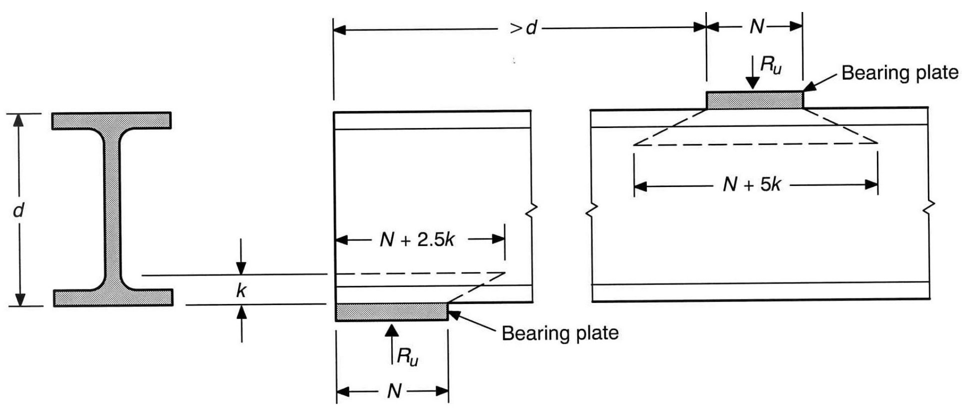


FIGURE 5

Web crippling is buckling of the web caused by the compressive force delivered through the flange. The nominal web crippling strength (R_n) of a beam for various load conditions and locations are provided in Equations J10-4, J10-5a and J10-5b of the 13th Edition. As with local web yielding, when the factored applied load exceeds ΦR_n (where $\Phi = 0.75$) either transverse stiffeners or a doubler plate is required with a minimum length equal to half the depth of the beam centered at the location of the concentrated load.

Sidesway web buckling of a beam may occur when the compression flange is not restrained against movement relative to the tension flange. The nominal sidesway web buckling strength (R_n) of a beam for various conditions is provided in Equations J10-6 and J10-7 of the 13th Edition. When the factored applied load exceeds ΦR_n (where $\Phi = 0.85$) and the loaded flange is restrained, either transverse stiffeners or a doubler plate is required with a minimum length equal to half the depth of the beam centered at the location of the concentrated load. As an alternate, lateral bracing in accordance with Section J10.4 of the 13th Edition Commentary may be provided to both flanges at the location of the load. Per the recommendations of the research performed by Joseph A. Yura, the lateral bracing must be designed for 1.0% of the applied load. For situations in which the factored applied load exceeds ΦR_n and the loaded flange is unrestrained lateral bracing must be provided to both flanges at the location of the load stiffener or doubler plates alone are in effective in this situation.

If stiffeners are used to resist the effectives of concentrated loads, limit state checks of web yielding and web crippling do not have to be performed as a part of the design process.

Designing for Serviceability

In addition to being structurally safe, a structure must also be serviceable. A serviceable structure is one that performs satisfactorily and does not cause discomfort or perceptions of an unsafe condition by the occupants or users. These areas of concern involve primarily vibrations and deflections for buildings. For a further discussion on vibrations see *Floor Vibrations Due to Human Activity*, AISC Steel Design Guide 11. For further discussion on general serviceability issues see *Serviceability Design Considerations for Steel Buildings*, AISC Steel Design Guide 3.

Chapter L of the 13th Edition provides little guidance concerning acceptable deflections. However, deflection can more often than not control the required size of a beam or other similar member as dictated by the requirements of the supported equipment, wall, ceiling, cladding or other architectural appurtenances. The Commentary for Chapter L recommends a deflection limitation of L/360 for floor members and L/240 for roof members. Both of these criteria are reasonable, but the actual deflection constraints for a given beam should be evaluated on a project by project basis. ACI 530 (Building Code Requirements for Masonry Structures) limits deflections of beams (and lintels) supporting masonry to the lesser of L/600 or 0.30 inches. The deflection analysis of a beam in all cases is based on the actual working or service, unfactored loads.

To accommodate dead load deflection that occurs during concrete floor slab placement and to produce a level finish floor, three alternates, or combinations thereof are used:

1. Let the beam deflect and pour a varying thickness slab (referred to sometimes as "ponding").

2. Shore the beams prior to concrete placement (applicable to composite construction only and is rarely if ever used anymore).

3. Camber the beams to compensate for the anticipated deflection of the self weight of the beam, the weight of metal deck and the weight of the wet concrete and in some cases portions of other superimposed loads.

When using the "ponding" method, the additional dead load of the concrete has to be taken into account during the design of the supporting beams and the contractor should be notified on the Construction Documents that additional concrete material will be required as a result of this approach. This later situation is normally handled with a general note such as this:

> *The concrete slabs shall be finished flat and level within tolerance, to the elevation indicated on the drawings. The Contractor shall provide additional concrete required due to formwork, metal deck, and framing deflection to achieve this finished top of slab elevation. The Contractor shall provide for a minimum of 5/8" average thickness for additional concrete during placement for all slabs supported and formed on steel deck over the entire floor area. The Contractor shall provide the means by which the maximum and minimum concrete slab thickness can be monitored and verified during and after the placing and finishing operations*

Beams less than 30 feet in length and or beams that require only ¾ inch or less camber are normally not specified with a required camber. This recommendation is based on my own experience and is more restrictive than that recommended by AISC. Recommendations on what portion of the deflection should be offset by camber vary and range from 75% to 80% of the dead load to all of the dead plus a portion of the live load. For a further discussion of this subject see *Cambering Steel Beams*, September 2004, Modern Steel Construction.

Beam Design Summary

The beam design process can be outlined as follows:

1. Establish the loads on the beam including an assumption for the dead load of the yet to be determined beam size. An approximate beam depth can first be established by using the rule of thumb that the maximum beam depth required will be one-half the span in inches (i.e. 24 feet span/2 = W12 beam).

2. Compute the maximum required moment and shear strength based on the load combinations required by the Code (using factored loads for LRFD and unfactored loads for ASD).

3. Select a beam shape that satisfies the strength requirements. This can be approached in two ways:

    a. Assume a beam size, compute the available strength and compare to the required strength. Revise as necessary.

    b. Use the beam design charts in Table 3-10 of the 13th Edition. This method is more commonly used than the iterative trial method described in 3a.

4. Check the shear strength.

5. Check the deflection.

Composite Beam Design

Composite beam construction originally included only steel beams totally encased in concrete. The concrete was intended primarily as fireproofing and in some cases protection against a corrosive environment. Provisions for the composite design of concrete encased steel beams were not initially included as a part of the early design of this type of section. Instead the steel beam was designed to carry all loads as a bare section. However, even today the current AISC Steel Manual includes provisions for the design of a totally encased steel beam as indicated by the minimum clear cover dimensions shown in Figure 6.

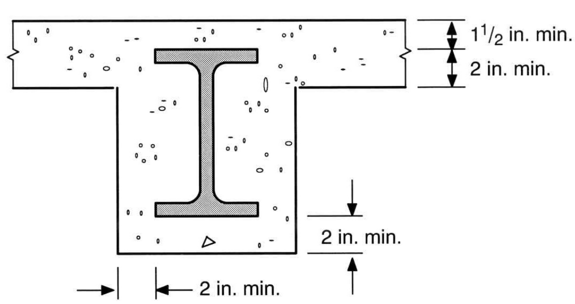


FIGURE 6

The 13th Edition does not however include composite design provisions for this type of construction, instead Section I3.3 of the Specifications allows for all of the load to be supported by the bare beam section, however, since the beam is completely restrained from both lateral and torsional buckling the design flexural strength for the member is;

$$\Phi_b M_n = \Phi_b M_p = 0.90ZF_y$$

The above type of construction is rarely if ever used today. Instead, most composite beam and concrete slab construction today involve a concrete slab constructed integrally with the beam for the resistance of all applied moments. Shear connectors (typically headed studs) welded to the top of the beam prior to the placement of the concrete assure composite action between the concrete slab and the steel beam. Although solid formed slabs can be used in this type of composite construction, most if not all framed concrete floor slabs are constructed today with stay in place composite metal deck. A discussion of the design and selection of metal floor (and roof) decks will be provided in the next Lecture.

It should be noted that composite beam and slab construction can be constructed as shored or unshored. In shored construction the beam is supported temporarily during the placement of the concrete slab and until the concrete reaches approximately 75% of it's required ultimate strength. This form of construction allows for the composite beam and slab section to support all dead and live loads. This type of construction is rarely if ever used today, primarily because of the extra cost and construction time required. In unshored construction the steel beam supports its self weight, the weight of the metal deck and the weight of the wet concrete as a bare section. The resulting composite section is then capable of supporting only the future live and other superimposed loads. Open web steel joists can also be used in composite construction. A discussion of the design and selection of open web steel composite joists will be provided in the next Lecture.

Another slightly different variation of composite construction used in retrofit work involves the strengthening of existing non-composite steel beams to enable them to support additional loads via the installation of new shear connectors. In this method, core holes in the existing framed slab are drilled directly above the beam top flange to allow for the headed studs to be installed. Once the shear connectors are welded in place the core holes are filled back in with a high strength, non-shrink grout thereby converting the bare section into a composite section.

Although it is possible to design composite beams as continuous, for the purposes of this course, only simple span composite beams are considered.

Advantages of composite construction over non-composite construction include:

1. Reduction in weight of steel beam (20% to 30%)
2. Shallower steel beams.
3. Increased floor stiffness.
4. Increased span length for the same given member.

Because of the shear lag in a composite section, the compressive stress (for positive bending only) in the concrete slab is not uniform and instead varies over the width of the slab (see Figure 7). To account for this phenomenon and to simplify the design of a composite section an effective slab width is defined with an assumed equivalent uniform stress. Section I3.1 of the 13th Edition specifies that the maximum width of the concrete slab on either side of the beam centerline is the lesser of;

1. 1/8 of the beam span
2. 1/2 of the beam spacing
3. The distance to the edge of the slab.

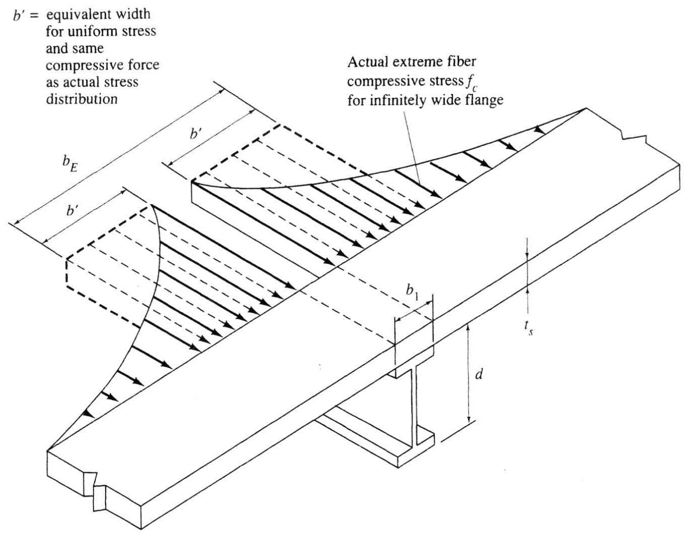


FIGURE 7

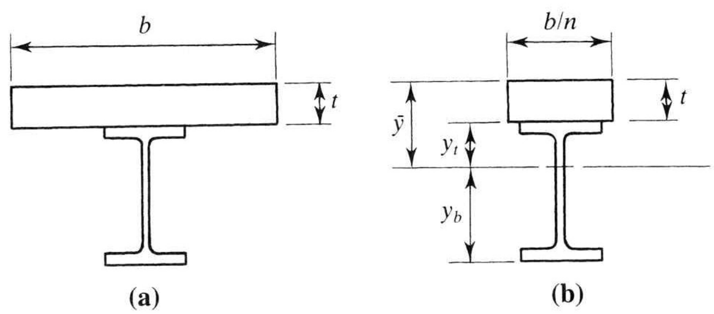
Although the available strength of a composite beam is usually based on conditions at failure, an understanding of the behavior of a composite section at service loads is important when calculating the deflection of the section. Flexural and shearing stresses in a beam consisting of homogenous materials can be computed from the formulas;

$$f_b = Mc/I = M/S \text{ and } f_v = VQ/Ib$$

A composite beam is not homogenous however so to be able to employ the above formulas a transformed section must first be calculated to "convert" the concrete slab into an equivalent amount of steel without alternating the section properties of the member. This is done via the use of the modular ratio ($n = E_s/E_c$) which allows for the effective width of the slab to be converted to an equivalent width (see Figure 8). The transformed section property is then calculated as I_{tr}. For positive bending only (i.e. compression at the top and tension at the bottom) the stress at the top and bottom of the steel beam are then calculated, respectively as;

$$f_{st} = My_t/I_{tr} \text{ and } f_{sb} = My_b/I_{tr}$$

The maximum compressive stress in the concrete slab is calculated as $f_c = M\tilde{y}/nI_{tr}$



where

I_{tr} = moment of inertia about the neutral axis (same as the centroidal axis for this homogeneous section).

y_t = distance from the neutral axis to the top of the steel

y_b = distance from the neutral axis to the bottom of the steel

FIGURE 8

In most cases the nominal flexural strength of a composite beam will be reached when the entire steel cross section yields and the concrete slab crushes in compression (for positive bending). This condition is referred to as the plastic stress distribution. AISC provisions for flexural strength of a composite beam are:

1. The nominal strength (M_n) is obtained from the plastic stress distribution for shapes with compact webs (i.e. $h/t_w \leq 3.76(E/F_y)^{1/2}$. Since all of the W, M and S shapes tabulated in the 13th Edition have compact webs (for flexure) for $F_y \leq 50$ ksi, this condition covers all composite beams except built-up steel shapes.

2. For LRFD the design strength = $\Phi_b M_n$; where $\Phi_b = 0.90$.

3. For ASD the allowable strength = M_n/Ω; where $\Omega = 1.67$

When a composite beam has reached the plastic limit state the stresses can be distributed in one of three ways (see Figure 9). The concrete stress is represented as a uniform compressive stress block of $0.85f'_c$ extending from the top of the slab to a depth equal to or less than the total slab thickness. Figure 9a shows a stress distribution corresponding to full tensile yielding of the steel and compression in only a portion of the concrete slab with the plastic neutral axis (PNA) in the slab. In this scenario (Case 1) the tensile contribution of the concrete is ignored. This stress distribution usually exists when enough shear connectors are provided to assure full composite action. Case 1 indicates that the slab is capable of developing in compression the full nominal strength of the steel beam in tension. In Figure 9b the concrete stress block extends the full depth of the slab and the PNA is in the top flange of the beam. In this scenario (Case 2-1) part of the top flange of the beam will therefore be in compression contributing to the compressive force in the slab. Figure 9c shows the condition in which the PNA is in the web of the steel beam (Case 2-2).

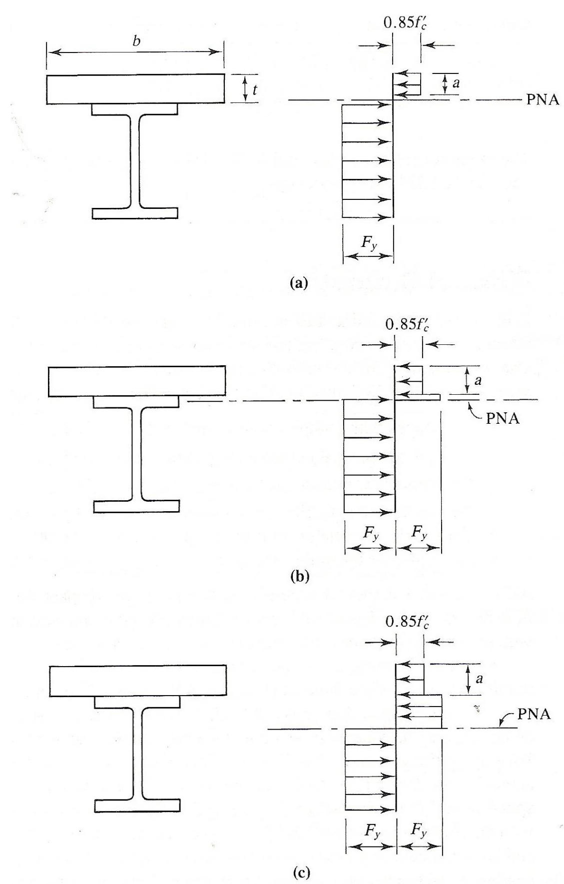


FIGURE 9

For each case represented in Figure 9, the nominal moment capacity can be found by computing the force couple moment formed by the compressive and tensile resultants. This can be accomplished by summing the moments of the resultants about any convenient point. It should also be noted that because the steel beam is braced by the concrete slab via shear connectors, lateral-torsional buckling is not a problem. To determine which of the three cases shown in Figure 9 controls compute the compressive resultant as the smallest of:

1. A_sF_y
2. $0.85f'_cA_c$
3. $\sum Q_n$

Where; A_s = total cross-sectional area of steel beam

A_c = Area of concrete = t_c x b

For full depth, formed slabs; t_c = the total slab thickness.

For slabs on fluted metal deck with the ribs perpendicular to the beam span; t_c = the thickness of the slab above the top of the highest deck flute. ***

For slabs on fluted metal deck with the ribs parallel to the beam span; t_c = the average thickness of the slab above and below the highest deck flute. ***

Q_n = Total shear strength of the shear connectors.

(Note that t in Figure 9 is the same as t_c in Figures 10 & 11)

*** The top of concrete for the purposes of calculating the PNA is still assumed to remain at the physical top of the slab.

For Case 1:
(see Figure 10)

$T_{stl} = A_sF_y = 0.85f'_cA_c = C_{con}$

$A_c = b \text{ x } a;\ a = A_sF_y / 0.85f'_cb$

$Y_2 = Y_{con} - a/2$

Summing moments about the PNA;

$M_n = C_{con}Y_2 + T_{stl}d/2 = 0.85f'_cbaY_2 + A_sF_yd/2$

Alternately; $y = Y_2 + d/2$

Summing moments about the PNA;

$M_n = T_{stl}y = T_{stl}(Y_2 + d/2)$

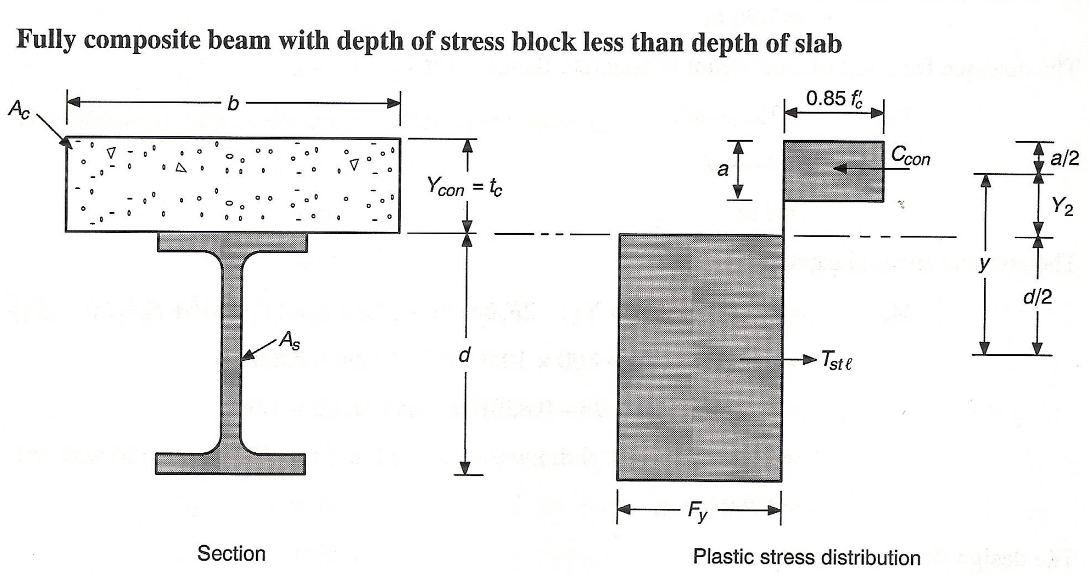


FIGURE 10

For Case 2-2: [Case 2-1 Similar] (see Figure 11)

$T_{stl} = (T - C_f - C_w) = (C_{con} + C_f + C_w)$

$A_c = b \times t_c$

$C_f = F_y b_f t_f$

$C_w = F_y t_w (Y_1 - t_f)$

Since; $(T - C_f - C_w) = (C_{con} + C_f + C_w)$

Then; $0 = C_{con} + 2C_f + 2C_w - T = 0.85f'_c A_c + 2\ F_y b_f t_f + (2\ F_y t_w (Y_1 - t_f)) - A_s F_y$

Therefore; $Y_1 = t_f + (A_s/2 - b_f t_f - (0.85f'_c A_c/2F_y))/t_w$

$Y_2 = Y_{con} - a/2 = t_c - t_c/2$

Summing moments about the PNA;

$M_n = (C_{con}(Y_1 + Y_2)) + (2C_f(Y_1 - t_f/2)) + (C_w(Y_1 - t_f)^2) + (T(d/2 - Y_1))$

or; $Mn = (0.85f'_c A_c (Y_1 + Y_2)) + (2\ F_y b_f t_f (Y_1 - t_f/2)) + (F_y t_w (Y_1 - t_f)^2) + (A_s F_y (d/2 - Y_1))$

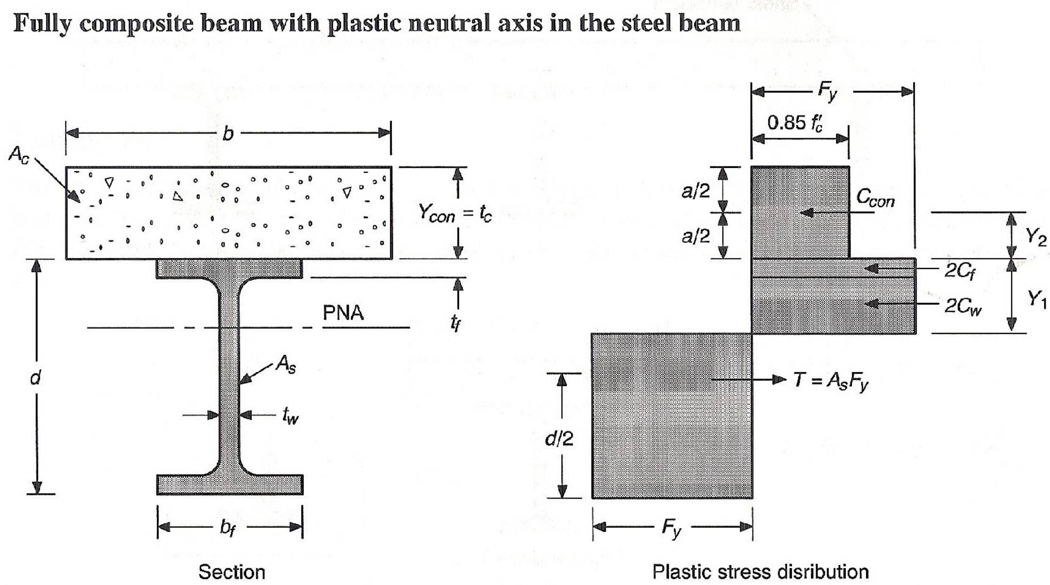


Figure 11

To assist in the determination of composite beam section properties for wide flange sections with full composite action and known values of $\sum Q_n$ and Y_2, Table 3-19 of the 13th Edition provides values of Y_1 and ΦM_n where;

$\sum Q_n = 0.85f'_cA_c$ (PNA located in the steel beam)

or; $\sum Q_n = A_sF_y$ (PNA located in the concrete slab)

Section I3.1b of the 13th Edition conservatively requires that all shear on a composite beam be resisted by the web of the steel beam as provided by Chapter G of the Specifications, the same as that discussed above for standard beams.

Shear Connectors:

To achieve composite actions between the steel beam and the concrete slab shear connectors are provided to transfer the horizontal shear force across the interface between the top of the beam and the bottom of the slab, and to prevent vertical separation. Headed studs are the most commonly used type of shear connector. The following requirements for headed studs are provided in Section I3.2d(6) of the 13th Edition unless otherwise noted.:

- Maximum diameter = 2.5 x flange thickness of the steel shape (unless located directly over the web)
- Minimum length = 4 x stud diameter (see Section I1.3)
- Minimum longitudinal center-to-center spacing = 6 x stud diameter
- Maximum longitudinal center-to-center spacing = 8 x slab thickness ≤ 36 inches
- Minimum transverse center-to-center spacing = 4 x stud diameter
- Minimum cover above studs (applicable to formed deck only) = ½ inch (see Section I3.2c)

- Minimum height of stud above the top of metal deck = 1½ inches (see Section I3.2c)
- Maximum stud diameter = ¾ inch (applicable to formed deck only); otherwise $2.5t_f$
- Minimum lateral cover = 1 inch (except for studs in formed deck)

Full Composite Action; The number of connectors required to provide full composite action is derived from ultimate strength concepts. When the PNA is at the top flange of the beam the steel beam has fully yielded and for full composite action the horizontal shear force at the beam/slab interface between the point of maximum positive moment and the adjacent points of zero moment is;

$$V' = T_{stl}$$

When the PNA is within the steel beam (Case 2-1 & 2-2) the full depth of the concrete slab is stressed to it's maximum compression capacity and for full composite action the horizontal shear force at the beam/slab interface between the point of maximum positive moment and the adjacent points of zero moment is;

$$V' = 0.85f'_cA_c = C_{con}$$

It is not necessary to determine the location of the PNA to establish which of the above two values controls as the lesser of the two values governs. In addition, it is not necessary to adjust the spacing of the connectors to accommodate variations in the shear along the beam because the flexibility and ductility of the connectors allows for the redistribution of stress from the more heavily loaded connectors to the more lightly loaded connectors. Therefore, for a uniformly loaded, simple span beam the required number of connectors between the maximum positive moment and the ends of the beam may be uniformly distributed. When a concentrated load is applied to a composite simple span beam, the number of connectors provided between the support and the concentrated load must be adequate to develop the bending moment at the load.

To provide complete shear connection and full composite action the required number of connectors on each side of the point of maximum moment is;

$$n = V'/Q_n$$

Where; Q_n = the nominal strength of the a single shear connector

$$Q_n = 0.5A_{sc}(f'_cE_c)^{1/2} \leq R_gR_pA_{sc}F_u$$

Where; A_{sc} = cross-sectional area of a stud shear connector, in^2

F_u = minimum specified tensile strength of a stud shear connector, ksi

F_u = 65 ksi for Type B, ASTM A108 stud

R_g = Stud group coefficient (see page 16.1-86, 13th Edition for values)

R_p = Stud position coefficient (see page 16.1-86, 13th Edition for values)

Since; $E_c = w^{1.5}(f'_c)^{1/2}$

Then for regular weight concrete (w = 145 PCF); $Q_n = 0.5A_{sc}(wf'_c)^{3/4} \leq R_gR_pA_{sc}F_u$

It should be noted that there is no distinction between LRFD and ASD when designing shear connectors for full composite action. This is because the required number of studs is determined by dividing the nominal shear strength (V') by a nominal strength Q_n , which involves no applied loads.

Partial Composite Action; If full composite action is not required and a smaller number of connectors (n_r) are provided, only partial composite action will be achieved and the nominal flexural strength of the composite member is reduced. With partial composite action neither the full strength of the concrete nor the steel can be developed and the PNA will usually fall within the steel section. Since the steel strength will not be fully developed in a partially composite beam a larger shape will be required than with a full composite behavior. However, fewer studs will be required, therefore the cost of the studs must be taken into account when assessing the economics of a partially composite beam vs. full composite beam. In almost all cases a full composite beam will have excess capacity therefore it is possible to fine tune the design of a full composite section by reducing the number of studs to allow for a partially composite beam.

For partial composite design (see Figure 12):

$$V' = n_r Q_n = \sum Q_n$$

$$\sum Q_n = C_{con}$$

$$a = \sum Q_n / 0.85 f'_c b$$

$$Y_2 = Y_{con} - a/2$$

$$Y_1 = t_f + (A_s/2 - b_f t_f - (\sum Q_n / 2F_y))/t_w$$

Summing moments about the PNA;

$$M_n = \sum Q_n(Y_1 + Y_2) + (2F_y b_f t_f(Y_1 - t_f/2)) + (F_y t_w (Y_1 - t_f)^2) + (F_y A_s(d/2 - Y_1))$$

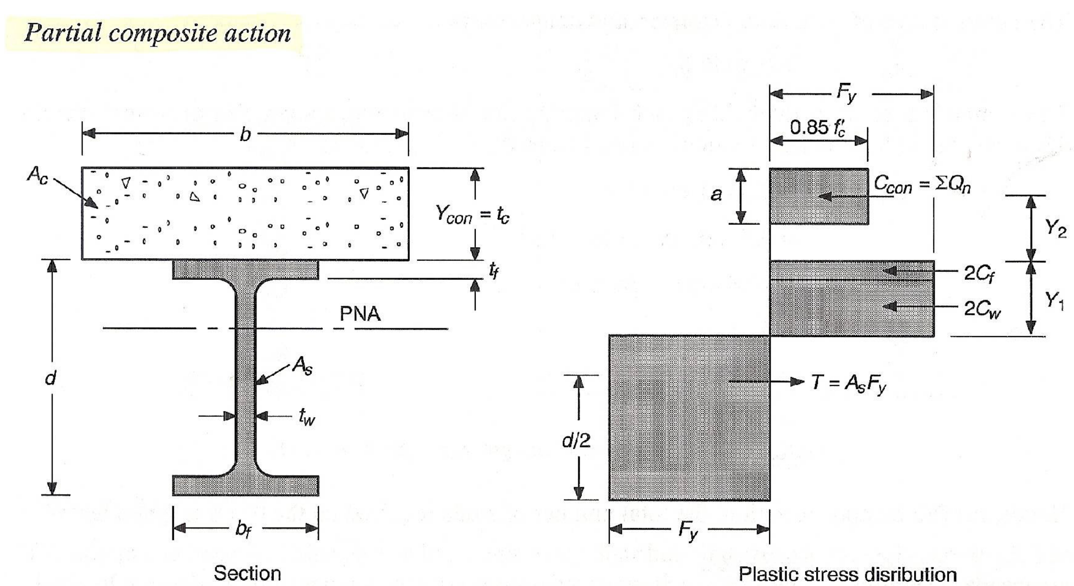


Figure 12

Commentary Section I3.1.3 indicates that less than 50% composite action produces large deflections and limited ductility of the member. For an elastic analysis of partially composite beams, when deflections are being calculated, an estimate of the moment of inertia of the partially composite section must be made. Per Equation C-I3-3 of the 13th Edition Commentary;

$$I_{eff} = I_s + ((\sum Q_n/C_f)^{1/2}(I_{tr} - I_s))$$

Where; Cf is the smaller of A_sF_y or $0.85f'_cA_c$

I_s = moment of inertia of bare steel beam

If the fraction of "compositeness", $\sum Q_n/C_f$, is less than 0.25, then the use of the above formula is not recommended.

Designing for Serviceability:

Because of the large moment of inertia of the transformed composite section, deflections for composite beams are less than that of a non-composite beam. For unshored construction the use of the larger moment of inertia is only applicable after the concrete slab has obtained adequate strength, therefore deflections that occur before the concrete has obtained adequate strength must be based on the bare steel section.

In addition, if the composite beam is subjected to sustained live or superimposed dead loads the creep of the concrete slab will allow additional deformation of the section to take place over a long period of time. Long-term deflection can only be estimated. One method of estimating long term deflection is to reduce the area of the concrete by using a factored modular ratio of 2n or 3n.

Section I3.1 of the 13th Edition Commentary indicates that the use of the transformed section (I_{tr}) for computing composite beam deflections underestimates the deflections by 15% to 30%. To offset this effect the Commentary recommends reducing the I_{tr} by 25 % (i.e. $I_{eff} = 0.75I_{tr}$)

Unshored Composite Beam Design Summary:

The unshored composite beam design process can be outlined as follows:

1. Select the appropriate floor system and deck (discussion of the design and selection of metal floor decks will be provided in the next Lecture).

2. Establish the loads on the beam including an assumption for the dead load of the yet to be determined beam size.

3. Compute the maximum required moment and shear strength based on the load combinations required by the Code (using factored loads for LRFD and unfactored loads for ASD).

4. Select a trial steel shape.

5. Compare the available strength of the steel shape to the required moment strength for the support of the dead weight of the beam, metal deck and wet concrete. Assume that the metal deck provides adequate lateral bracing of the beam.

6. Compute the available strength of the composite section and compare it to the total required moment strength.

7. Check the shear strength of the bare steel section.

8. Design the shear connectors.

    a. Compute V' at the interface between the top of the beam and bottom of the slab.

    b. Divide V' by Qn to obtain half of the total number of studs required on the beam.

9. Check deflections.