Steel Structures

M.Sc. Structural Engineering

SE-505

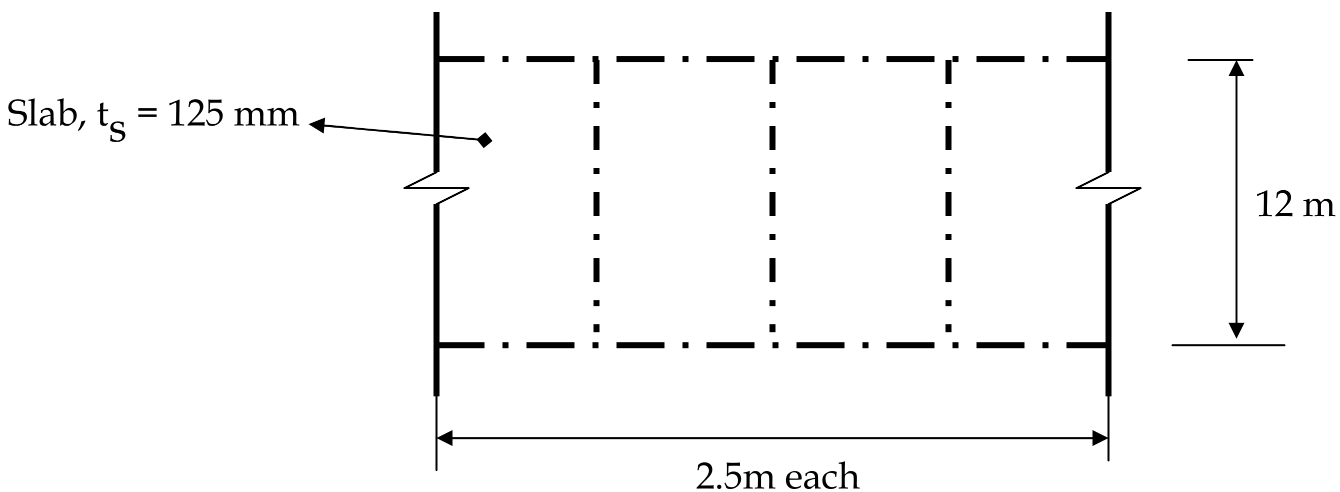
Lecture # 6

Composite Steel-Concrete Construction

Steel Structures

Example:

Design an interior composite and simply supported beam to span 12 m with a beam spacing of 2.5m. Minimum number of 20mmΦ x 80 mm stud connectors of A36 steel are to be used. t_s = 125 mm and slab is poured without shores. Live load = 750 kg/m^2, grade 345 steel, f_c' = 20MPa. Other dead loads are 155 kg/m^2.



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Solution:

$$d = \frac{L}{24} = \frac{12000}{24} = 500mm$$

We can try W 460 or W 530

Dead Load of Slab $= \frac{125}{1000} \times 2400 = 300\,\mathrm{kg/m^2}$

Other dead loads (assumed) $= 155\mathrm{kg/m^2}$

Assumed self weight $= 0.1(300 + 155) \cong 45\mathrm{kg/m^2}$

Total dead load $= (300 + 155 + 45) = 500\mathrm{kg/m^2}$

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Solution: (contd…)

Live Load, $w_L = 750 \text{kg}/\text{m}^2$

$$w_u = (1.2 \times 500 + 1.6 \times 750) \times \frac{9.81}{1000} \times 2.5$$

$$w_u = 44.15 \text{kN} - \text{m}$$

$$M_u = \frac{w_u L^2}{8} = \frac{44.15 \times 12^2}{8}$$

$$= 794.7 \text{kN} - \text{m}$$

Approx. self weight $= \dfrac{M_u}{\left(\dfrac{d}{2} + t_s - \dfrac{a}{2}\right)\phi_b F_y} \times 0.00785 = 60.9 kg/m$

$$= \frac{60.9}{2.5} = 24.4 kg/m^2 < 45 kg/m^2$$

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Solution: (contd…)

Assuming N.A. to lie with in the slab

$$(A_s)_{req} = \frac{M_u}{\left(\frac{d}{2} + t_s - \frac{a}{2}\right)\phi_b F_y}$$

$$= \frac{794.7 \times 10^6}{\left(\frac{460}{2} + 125 - \frac{50}{2}\right) 0.9 \times 345}$$

$$\boxed{(A_s)_{req} = 7756\, mm^2}$$

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Solution: (contd…)

Trial Section: W 460 x 68

A_s = 8710 mm^2, d = 459 mm, I_x = 29,600 x 10^4 mm^4

b_f = 154 mm, t_f = 15.4 mm, t_w = 9.1 mm, h/t_w = 44.6

Local stability of web

For the case if N.A. is within the steel section, some part of web may be in compression.

$$\lambda = 3.76\sqrt{\frac{E}{F_y}} = 90$$

$$\frac{h}{t_w} < \lambda = 90 \qquad \text{O.K.}$$

Plastic stress distribution is also allowed now.

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Solution: (contd…)

Effective Width, b_E

b_E is smaller of

1. $L/4 = 12000/4 = 3000$ mm
2. $s = 2500$ mm

$$\boxed{b_E = 2500 \text{ mm}}$$

Minimum Number of Shear Connectors

$$s_{\max} = 8t_s = 8 \times 125 = 1000mm$$

Two connectors at a section

Minimum Number of Shear Connectors $= \left(\dfrac{12000}{1000} + 1 \right) \times 2 = 26$

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Solution: (contd…)

Strength Provided by Connectors in Half Span

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \le R_g R_p A_{sc} F_u$$

$$= \frac{0.5}{1000}\left(\frac{\pi}{4}\times 20^2\right)\sqrt{20\times 21019} \le \left(\frac{\pi}{4}\times 20^2\right)\frac{400}{1000}$$

$$= 101.84 \le 125.6 kN \quad \Rightarrow \quad \boxed{Q_n = 101.84 kN}$$

$$\Sigma Q_n = \frac{26}{2}\times 101.84 = 1323.92 kN$$

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Solution: (contd…)

$$T_{\max} = A_s F_y = 8710 \times {}^{345}\!\big/_{1000} = 3004.95kN$$

$$\Sigma Q_n < T_{\max}$$

Maximum value of C_c will be equal to ΣQ_n for equilibrium at the interface.

Since the force in the slab based on connector strength is less than the maximum steel force so P.N.A lie within steel section

The section will be partially composite

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Solution: (contd…)

Location of P.N.A

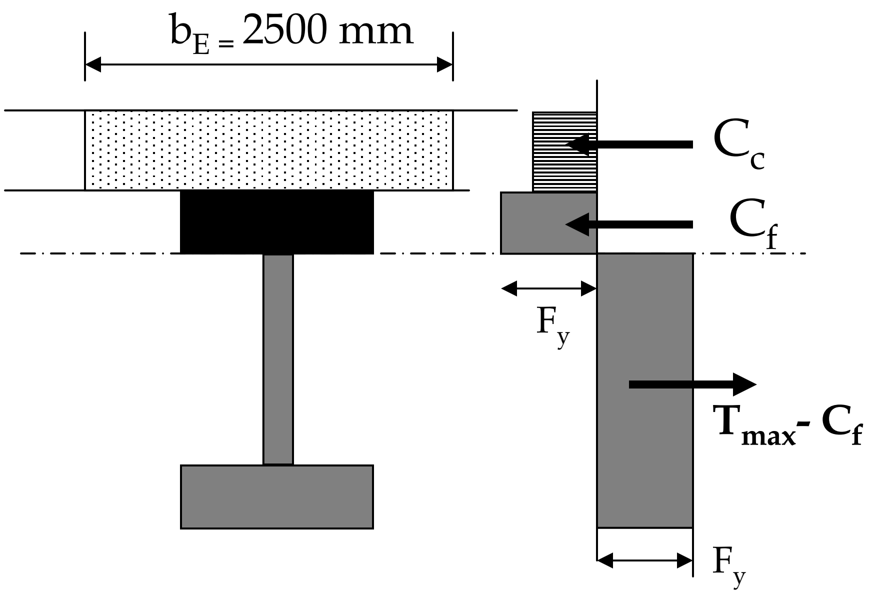
Assuming P.N.A. to be at the junction of flange and web.

Remaining tensile force $= T_{max} - C_f$

$$= 3004.95 - b_f t_f F_y$$

$$= 3004.95 - \frac{154 \times 15.4 \times 345}{1000}$$

$$= 3004.95 - 818.2 = 2186.75 \text{kN}$$



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Solution: (contd…)

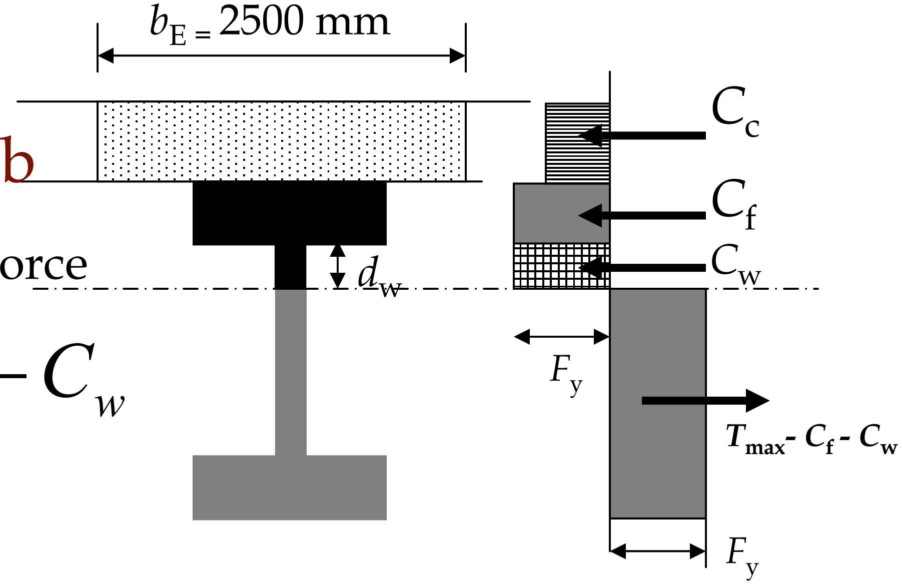
Location of P.N.A

Total compressive force is:

$$\Sigma Q_n + C_f = 1323.92 + 818.2 = 2142.12kN$$

$$\boxed{T_{\max} - C_f > \Sigma Q_n + C_f}$$

So P.N.A. has gone into the web



Total Compressive force = Total tensile force

$$\Sigma Q_n + C_f + C_w = T_{\max} - C_f - C_w$$

$$2C_w = T_{\max} - \Sigma Q_n - 2C_f$$

$$\boxed{C_w = \frac{T_{\max} - \Sigma Q_n}{2} - C_f}$$

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Solution: (contd…)

Location of P.N.A

$$C_w = \frac{3004.95 - 1323.92}{2} - 818.20$$

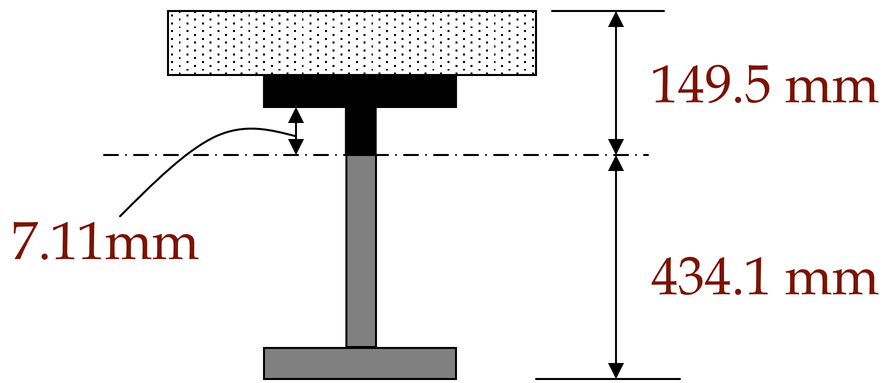
$$= 22.32kN$$

$$= t_w d_w F_y$$

$$22.32 = 9.1 \times d_w \frac{345}{1000}$$

$$d_w = 7.11mm$$

Depth of web in compression



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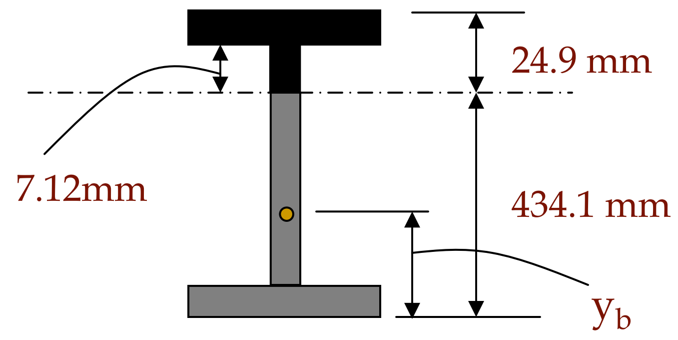
Solution: (contd…)

Portion of Steel Section in Tension

$$Area = 8710 - 154 \times 15.4 - 9.1 \times 7.11$$

$$= 6273.7\ mm^2$$



$$y_b = \frac{8710 \times 459/2 - 154 \times 15.4 \times (459 - 15.4/2) - 7.11 \times 9.1 \times (459 - 15.4 - 7.11/2)}{6273.7}$$

$$y_b = 143.48\ mm$$

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Solution: (contd…)

Moment Capacity

C_c is lesser of

1- $$0.85 f_c' b_E t_s = 0.85 \times 20 \times 2500 \times 125 / 1000$$

$$= 5312.5\ kN$$

2- $$\Sigma Q_n = 1323.92\ kN$$

$$\boxed{C_c = 1323.92\ kN}$$

As $C_c < 0.85 f_c' b_E t_s$ it is partially composite beam

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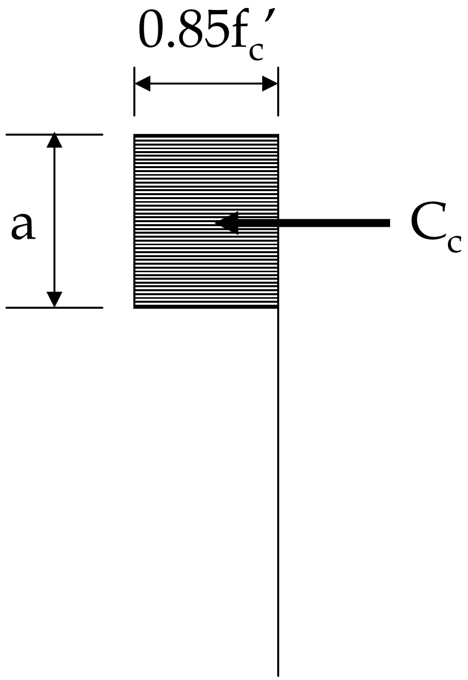
Solution: (contd…)

$$a = \frac{C_c}{0.85 f_c' b_E}$$

$$= \frac{1323.92 \times 1000}{0.85 \times 20 \times 2500}$$

$$\boxed{a = 31.2\, mm}$$



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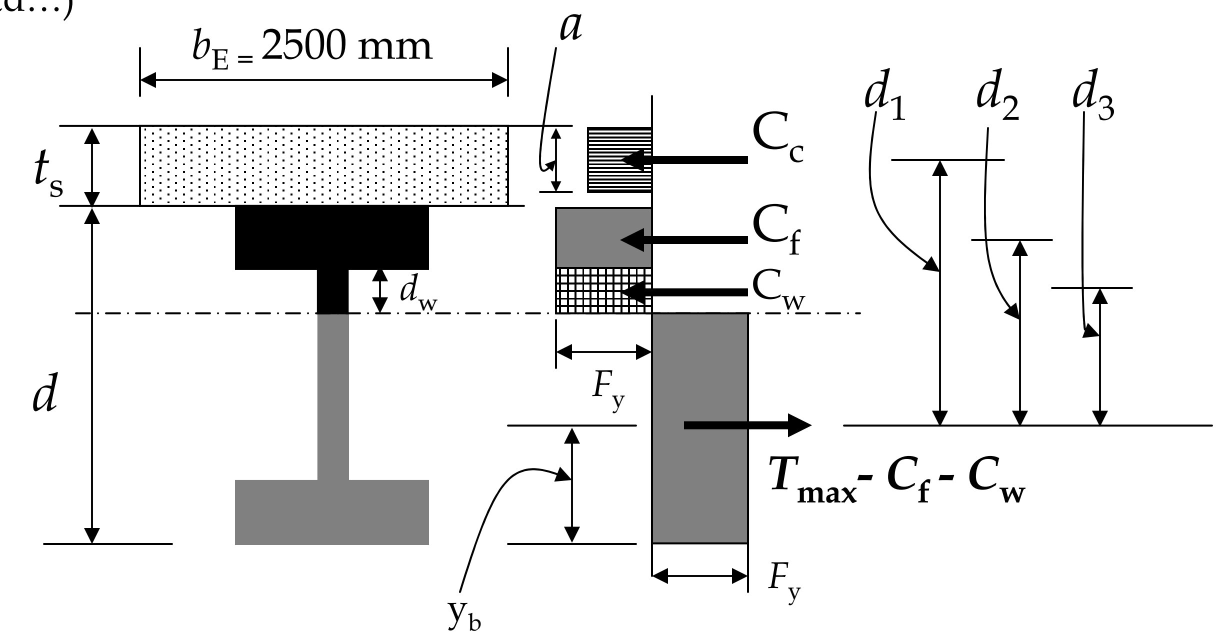
Solution: (contd…)



$$d_1 = d + t_s - y_b - \frac{a}{2} = 424.92mm$$

$$d_2 = d - y_b - \frac{t_f}{2} = 307.82mm$$

$$d_3 = d - y_b - t_f - \frac{d_w}{2} = 296.56mm$$

d = 459 mm
t_s = 125 mm
y_b = 143.48 mm
a = 31.2
t_f = 15.4 mm
d_w = 7.11

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Solution: (contd…)

$$\phi_b M_n = \phi_b \left[C_c d_1 + C_f d_2 + C_w d_3 \right]$$

$$= \frac{0.90}{1000} \times \left[1323.92 \times 424.92 + 818.2 \times 307.82 + 22.32 \times 296.56 \right]$$

$$= 738.93 kN - m < Mu = 794.61 kN - m$$ Not O.K.

Revise

The section has failed because in the beginning we assumed the section to be fully composite and considered the N.A with in the Slab

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Solution: (contd…)

Trial # 2, W 530 x 66

A = 8390 mm^2, d = 525 mm, b_f = 165 mm, t_f = 11.4 mm,

t_w = 8.9 mm, h/t_w = 53.6

Location of P.N.A.

$$T_{\max} = A_s F_y = 8390 \times 345/1000 = 2894.55 kN$$

$$\Sigma Q_n = 1323.92 kN < T_{\max}$$

So N.A. is with in the steel section

Steel Structures

Solution: (contd…)

Location of P.N.A.

$$C_f = b_f t_f F_y = 165 \times 11.4 \times 345/1000 = 648.95kN$$

$$\Sigma Q_n + C_f = 1309.23 + 778.87 = 20881.1kN < T_{\max}$$

So N.A. is in the web

$$C_w = \frac{T_{\max} - \Sigma Q_n}{2} - C_f$$

$$= \frac{2894.55 - 1323.92}{2} - 648.95$$

$$\boxed{C_w = 136.36kN}$$

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Solution: (contd…)

Location of P.N.A.

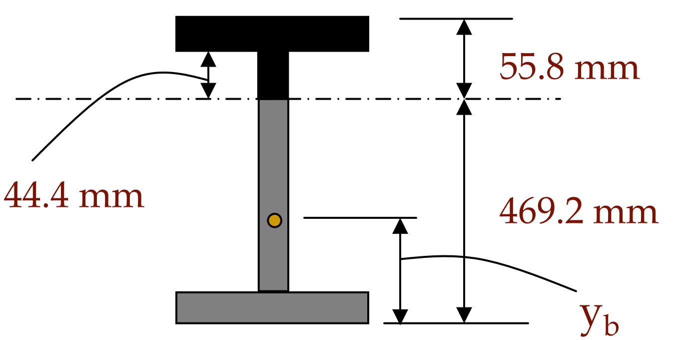
$$C_w = t_w d_w F_y$$

$$136.36 = 8.9 \times d_w \, 345/1000$$

$$d_w = 44.4mm$$

Area in Tension = 6113.84 mm^2

y_b = 168.7 mm



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Solution: (contd…)

$$d_1 = d + t_s - y_b - \frac{a}{2} = 465.7mm$$

$$d_2 = d - y_b - \frac{t_f}{2} = 350.6mm$$

$$d_3 = d - y_b - t_f - \frac{d_w}{2} = 322.7mm$$

$$\phi_b M_n = \phi_b \left[C_c \times d_1 + C_f \times d_2 + C_w \times d_3 \right]$$

$$= \frac{0.90 \times [1323.92 \times 465.7 + 648.95 \times 350.6 + 136.36 \times 322.7]}{1000}$$

$$= 799.3kN - m < M_u = 794.61kN - m$$

O.K.

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Solution: (contd…)

Check at the Construction Stage

Self weight of beam = 66 kg/m = 26.4 kg/m^2

Self weight of slab $= \dfrac{125}{1000} \times 2400 \times 2.5 = 750 \text{kg}/\text{m}$

Construction Live Load = 100 kg/m^2 = 100 x 2.5 = 250 kg/m

$$w_u = \left[1.2 \times 66 + 1.6 \times (750 + 250)\right] \times \frac{9.81}{1000}$$

$$= 16.47 kN/m$$

$$M_u = \frac{16.47 \times 12^2}{8} = 296.51 kN - m$$

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Solution: (contd…)

Check at the Construction Stage

$$\phi_b M_p = 0.9 Z_x F_y$$

$$= 0.9 \times 1563 \times 10^3 \times 345 \big/ 10^6$$

$$= 485.31 \text{ kN} - \text{m} \quad > M_u$$ **O.K.**

Final Selection W 530 x 66

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Composite Columns

“A steel column fabricated from rolled or built-up steel shapes and encased in structural concrete or fabricated from steel pipes or HSS and filled with structural concrete”

Encased Composite Column

“A steel column fabricated from rolled or built-up shapes and encased in structural concrete”

Filled Composite Column

“Structural Steel HSS or pipe that are filled with structural concrete”

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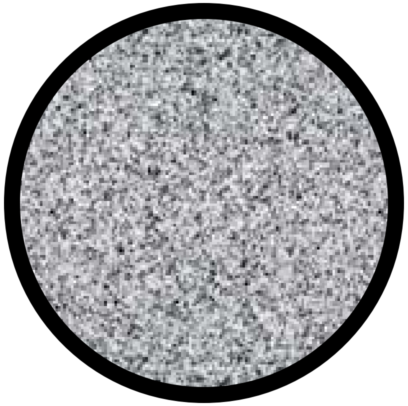
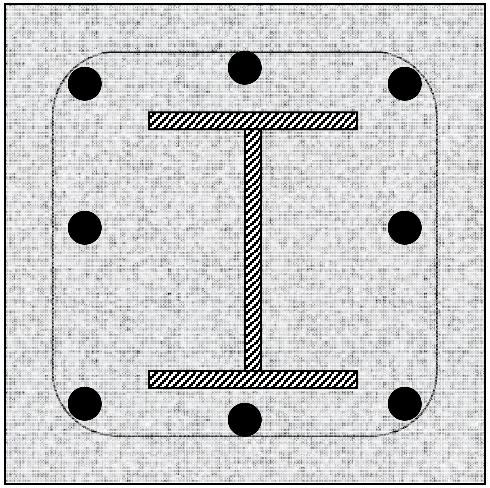
Composite Columns (contd…)



Filled Composite Column



Encased Composite Column

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I2.1 Limitations

To qualify as Encased Composite Column, the following limitations shall be met

1. Steel section area ≥ 1% of A_g.
2. Concrete encasement of a steel core shall be reinforced with longitudinal load-carrying bars to restrain concrete and lateral ties. Minimum longitudinal reinforcement must be 0.4% of the gross area of the composite member.

$$\rho_{sr} = \frac{A_{sr}}{A_g}$$

where

A_{sr} = area of continuous reinforcing bars, mm2

A_g = gross area of composite member, mm2

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3. The minimum transverse reinforcement is to be 0.23 mm2/mm of tie spacing.
4. 21 MPa ≤ fc′ ≤ 70 MPa
5. Fy ≤ 525 MPa

Compressive Strength

The *design compressive strength*, $\phi_c P_n$, and *allowable compressive strength*, P_n/Ω_c, for axially loaded *encased composite columns* for the limit state of *flexural buckling* based on column slenderness is evaluated as follows:

Steel Structures

(a) When $P_e \geq 0.44 P_o$

$$P_n = P_o \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right]$$

(b) When $P_e < 0.44 P_o$

$$P_n = 0.877 P_e$$

$\phi_c = 0.75$ (LRFD) $\Omega_c = 2.00$ (ASD)

$$P_o = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f'_c$$

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$$P_e = \frac{\pi^2 (EI_{eff})}{(KL)^2}$$

A_s = area of the steel section, mm^2

A_c = area of concrete, mm^2

A_{sr} = area of continuous reinforcing bars, mm^2

E_c = modulus of elasticity of concrete

$$= 0.043 w_c^{1.5} \sqrt{f_c'} \quad \text{, MPa}$$

E_s = modulus of elasticity of steel = 200, GPa

f_c' = specified compressive strength of concrete, MPa

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F_y = specified minimum yield stress of steel section, MPa

F_{yr} = specified minimum yield stress of reinforcing bars, MPa

I_c = moment of inertia of the concrete section, mm^4

I_s = moment of inertia of steel shape, mm^4

I_{sr} = moment of inertia of reinforcing bars, mm^4

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$K =$ the effective length factor

$L =$ laterally unbraced length of the member, mm

$w_c =$ weight of concrete per unit volume

$(1500 \leq w_c \leq 2500 \text{ kg/m}^3)$

$EI_{eff} =$ effective stiffness of composite section, N-mm^2

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c$$

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.3$$

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Shear Strength

The *available shear strength* is calculated based on either the shear strength of the steel section alone plus the shear strength provided by tie reinforcement, if present, or the shear strength of the reinforced concrete portion alone.

Load Transfer

*Load*s applied to axially loaded encased composite columns must be transferred between the steel and concrete. The shear connectors must satisfy the following requirements:

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(a) When the external *force* is applied directly to the steel section, *shear connectors* are provided to transfer the required shear force, V', as follows:

$$V' = V(1 - A_s F_y / P_o)$$

$V =$ required shear force introduced to *column*, N

$A_s =$ area of steel cross section, mm^2

$P_o =$ nominal axial compressive strength without consideration of *length effects*, N

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(b) When the external force is applied directly to the concrete encasement, shear connectors must be provided to transfer the required shear force, V', as follows:

$$V' = V(A_s F_y / P_o)$$

(c) When load is applied to the concrete of an encased composite column by direct bearing the *design bearing strength*, $\phi_B P_p$, and the *allowable bearing strength*, P_p/Ω_B, of the concrete is as given below:

Steel Structures

$$P_p = 1.7 f_c{}' A_B$$

$$\phi_B = 0.65 \text{ (LRFD)} \qquad \Omega_B = 2.31 \ (ASD)$$

A_{B} = loaded area of concrete, mm^2

Detailing Requirements

At least four continuous longitudinal reinforcing bars are to be used in encased composite columns.

Transverse reinforcement must be spaced at the smallest of 16 longitudinal bar diameters, 48 tie bar diameters or 0.5 times the least dimension of the composite section.

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A minimum clear cover of 38 mm to the reinforcing steel must be maintained.

Shear connectors are provided to transfer the required shear *force*.

The shear connectors are distributed along the length of the member at least a distance of 2.5 times the depth of the encased composite column above and below the load transfer region.

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The maximum connector spacing is to be 405 mm. Connectors to transfer axial load are placed on at least two faces of the steel shape in a configuration symmetrical about the steel shape axes.

If the composite cross section is built up from two or more encased steel shapes, the shapes are interconnected with *lacing*, *tie plates*, *batten plates* or similar components to prevent buckling of individual shapes due to loads applied prior to hardening of the concrete.

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Strength of Stud Shear Connectors

The *nominal strength* of one stud shear connector embedded in solid concrete is:

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \quad \leq \quad A_{sc} F_u$$

A_{sc} = cross-sectional area of stud shear connector, mm2

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

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Composite Columns (contd…)

I2.1 Limitations

To qualify as Filled Composite Column, the following limitations shall be met

1. Steel section area ≥ 1% of A_g.
2. The maximum $b\,/\,t$ ratio for a rectangular HSS is to be $2.26\sqrt{E/F_y}$.
3. The maximum $D\,/\,t$ ratio for a round HSS is to be $0.15\sqrt{E/F_y}$.
4. 21 MPa $\leq f_c' \leq$ 70 MPa
5. Fy ≤ 525 MPa

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Compressive Strength

The *design compressive strength*, $\phi_c P_n$, and *allowable compressive strength*, P_n/Ω_c, for axially loaded *encased composite columns* for the limit state of *flexural buckling* based on column slenderness is evaluated as follows:

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(a) When $P_e \geq 0.44 P_o$

$$P_n = P_o \left[0.658^{\left(\frac{P_o}{P_e}\right)} \right]$$

(b) When $P_e < 0.44 P_o$

$$P_n = 0.877 P_e$$

$\phi_c = 0.75$ (LRFD) $\Omega_c = 2.00$ (ASD)

$$P_o = A_s F_y + A_{sr} F_{yr} + C_2 A_c f'_c$$

$$P_e = \frac{\pi^2 (EI_{eff})}{(KL)^2}$$

$C_2 =$ 0.85 for rectangular section and 0.95 for circular sections.

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EI_{eff} = effective stiffness of composite section, N-mm^2

$$EI_{\text{eff}} = E_{\text{s}}\, I_{\text{s}} + E_{\text{s}}\, I_{\text{sr}} + C_3 E_{\text{c}}\, I_{\text{c}}$$

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \quad \leq \quad 0.9$$

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Shear Strength

The *available shear strength* is calculated based on either the shear strength of the steel section alone plus the shear strength provided by tie reinforcement, if present, or the shear strength of the reinforced concrete portion alone.

Load Transfer

Loads applied to filled composite columns are to be transferred between the steel and concrete.

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When the external force is applied either to the steel section or to the concrete infill, transfer of force from the steel section to the concrete core is required from *direct bond interaction*, shear *connection* or direct bearing.

The force transfer *mechanism* providing the largest *nominal strength* may be used. These force transfer mechanisms should not be superimposed.

When load is applied to the concrete of an encased or filled composite column by direct bearing the *design bearing strength*, $\phi_B P_p$, and the *allowable bearing strength*, P_p/Ω_B, of the concrete is as follows:

Steel Structures

$$P_p = 1.7 f_c' A_B$$

$$\phi_B = 0.65 \text{ (LRFD)} \qquad \Omega_B = 2.31 \ (ASD)$$

A_B = loaded area of concrete, mm^2

Detailing Requirements

Where required, shear connectors transferring the required shear force are distributed along the length of the member at least a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS both above and below the load transfer region.

The maximum connector spacing is to be kept equal to 405 mm.

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Composite Columns (contd…)

Advantages of Composite Columns

1. Better fire rating, less rusting.
2. Better appearance, improved by concrete.
3. Better performance than steel column against impact load.
4. Composite sections are smaller than RCC sections, specially important in high rise building.
5. Light columns so small foundations.
6. Advantage of both materials is utilized, M.O.I increases.
7. Construction time can be reduced.

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Composite Columns (contd…)

Disadvantages of Composite Columns

1. To predict shortening is more difficult because of two different materials in the same member.
2. Differential shortening of different columns may cause complete failure of structure.
3. If some columns are of steel and some are composite, concrete in composite section has creep while in steel columns there is no creep. This causes differential settlement.
4. Lack of knowledge about bond between steel and concrete.

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Encased Beams

"A beam totally encased in concrete cast integrally with the slab may be assumed to be interconnected to the concrete by natural bond, without additional anchorage" provided that

1. Concrete cover over the beam sides and soffit is at least 50 mm.
2. The top of the beam is at least 38 mm below the top and 50 mm above the bottom of the slab.
3. Concrete encasement contains adequate mesh or other reinforcing steel to prevent spalling of concrete

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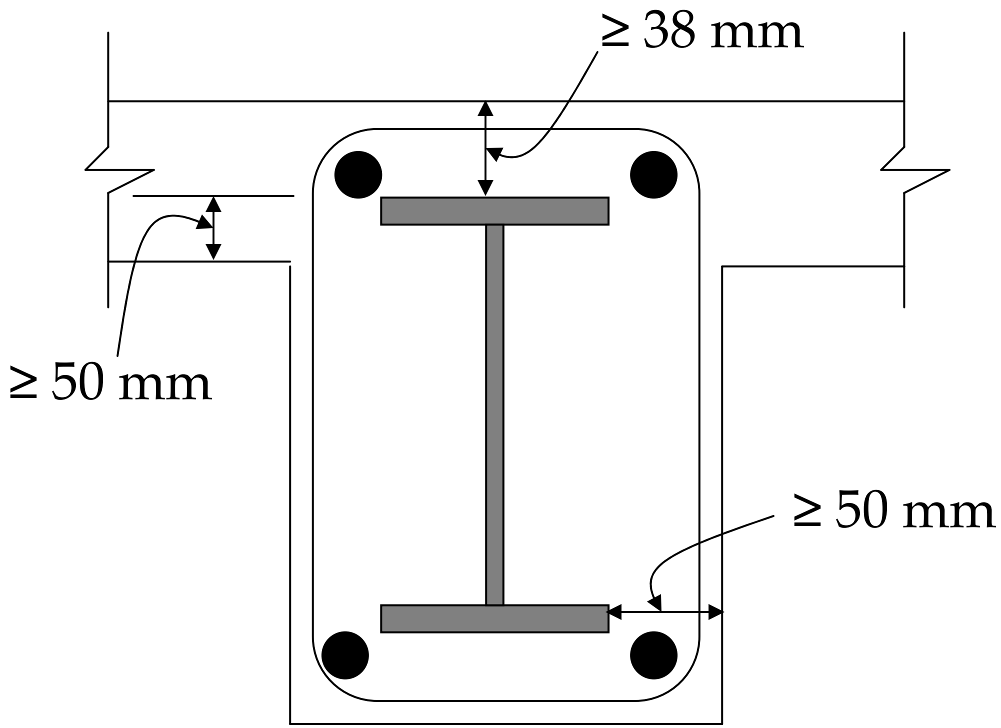
Encased Beams



Also study ACI 318-05 Chapter # 17 and ACI 318-10.16.8

Concluded