A COMPUTER PROGRAM ON COMPOSITE BEAM DESIGN

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My sincere appreciation and admiration go to the people on my committee who, through the years, have shared their knowledge and expertise with me and assisted me in this project.

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DESCRIPTION OF COMPOSITE BEAM PROGRAM

This program is structured to aid in the design of composite beams. It is primarily written for simply supported beams, since this design, with only positive bending, makes much better use of the materials used in composite design—that is steel in tension and concrete in compression. The program will however aid in the design of continuous beam segments.

The sequence of input is the same as in the example problems on composite design in the AISC manual and is as follows:

1. Loads Input

At least two load conditions must be input. The first load condition is loads applied before the concrete reaches 75% of its required strength.

Loads applied after the concrete reaches 75% of its compression strength are input next. Up to five load~~s~~ conditions (after load condition 1) can be input. The program determines the most severe moments and shears at 1/20th points resulting from these load conditions.

Only uniform, concentrated, and axial loads are accepted. The axial load is only used to evaluate the allowable bending stress.

For continuous beam spans, the left end shear and moment must first be obtained by other means and is input data for the composite beam program.

2. Design Constraints

In this segment of the program several questions are asked:

Design Constraints:

1. Unit weight of concrete (Kips/cubic ft)
2. Yield strength of steel (KSI)
3. Concrete compressive strength (KSI)
4. Beam spacing (ft)
5. Slab thickness (in)
6. Maximum and minimum permissible depth of beam (in)

With the given data the program, at the user's option, will:

*1. Choose the lightest section
2. Allow the user to select qualifying sections
3. Allow the user to select sections based on depth requirements for cover plate design in region of positive bending

If an acceptable preliminary beam is found, the user then inputs:

1. Whether or not formed steel deck (FSD) is used. If formed steel deck is used, the user inputs:

    a. Rib height
    b. Rib width
    c. Whether or not FSD runs parallel or perpendicular to beam (if ribs run perpendicular to beam, then rib spacing must be input).

From this data the transformed section modulus is again calculated based on the above information and actual design constraints.

At this time, if the beam is continuous, the user must input, if required, cover plate sizes and reinforcing steel.

With the actual section modulus calculated for the positive (negative) moment region, actual stresses are calculated based on one of the following two conditions:

1. <u>Unshored construction</u>
   Bending in composite section based on total load
   Bending in steel beam alone based on initial load
   Shear in steel beam based on total load
   Compression in concrete based on live loads.

2. <u>Shored construction</u>
   Bending in composite section based on total load
   Bending in steel beam alone based on initial load
   Shear in steel beam based on total load
   Compression in concrete based on total load

Assuming the stresses are okay, (if they are not, the program goes back to "Design Constraints"), the program will calculate the magnitude and location of the maximum deflection.

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* Available beam sizes in this program are the same as those listed on pages 2-108 and 2-109 of the AISC manual

FLOW CHARTS

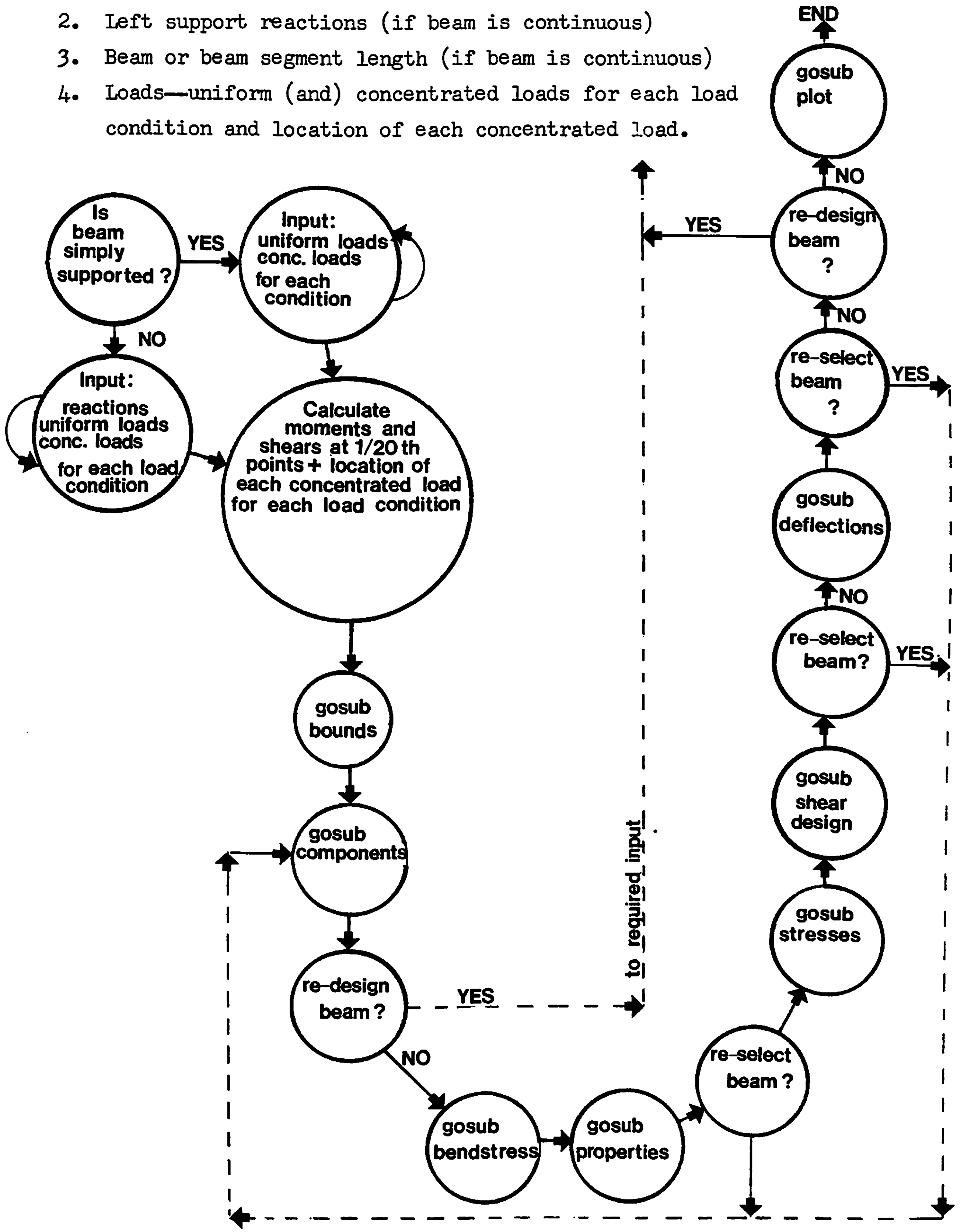
The following pages show the general sequence of program operations.

The first page diagrams the main program. The following pages graphically describes the subroutines. Many subroutines were used to allow for a very flexible program as will be apparent from the flow chart of the main program. Following is a brief verbal description of the main program and each subroutine. A more thorough description of equations and procedures used are explained later in this report.

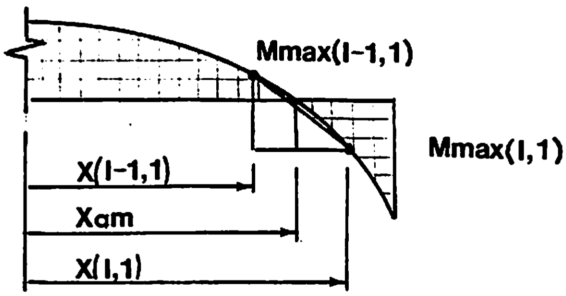
NAME	DESCRIPTION
1 Lillod	Main program which finds shears and moments at 1/20th points plus each point of concentrated load for each load condition.
1(a) Components	Subroutine that selects a preliminary composite section. Cover plated beams can be designed if necessary. Calculates preliminary section moduli.
1(b) Bendstress	Subroutine that calculates allowable bending stress for load condition 1 when concrete has not reached 75% strength.
1(c) Properties	Subroutine that calculates exact section properties (Y, I_t, and Str) based on whether or not formed steel deck is used, actual effective width of concrete, etc.
1(d) Stresses	Calculates actual stresses for steel beam alone, composite beam, and concrete stress. Warning: The program leaves it up to the discretion of the user whether or not to continue design.
2(e) Sheardesign	Calculates number of shear stud connectors required for both full and composite action.
1(f) Deflections	Calculates deflections for beams at 1/20th points plus points of concentrated loads

Required input:

1. Number of load conditions
2. Left support reactions (if beam is continuous)
3. Beam or beam segment length (if beam is continuous)
4. Loads—uniform (and) concentrated loads for each load condition and location of each concentrated load.



Case 1 Moment going from positive to negative



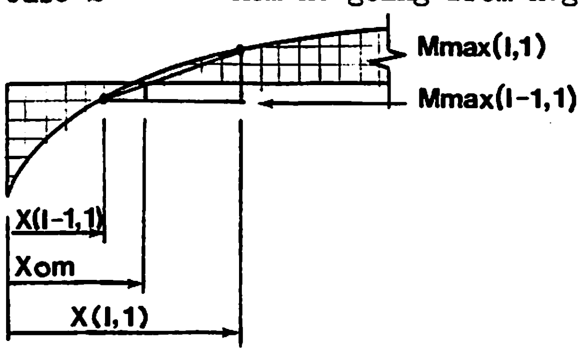
From similar triangles

$$\frac{X(I,1)-Xom}{ABS(Mmax\ (I,1))} = \frac{X(I,1)-X(I-1,1)}{Mmax(I-1,1)-Mmax(I,1)}$$

$$-Xom = \frac{ABS(Mmax(I,1))[X(I,1)-X(I-1,1)]}{Mmax(I-1,1)-Mmax(I,1)} - X(I,1)$$

$$Xom = X(I,1) - \frac{ABS(Mmax(I,1))[X(I,1)-X(I-1,1)]}{Mmax(I-1,1)-Mmax(I,1)}$$

Case 2 Moment going from negative to positive

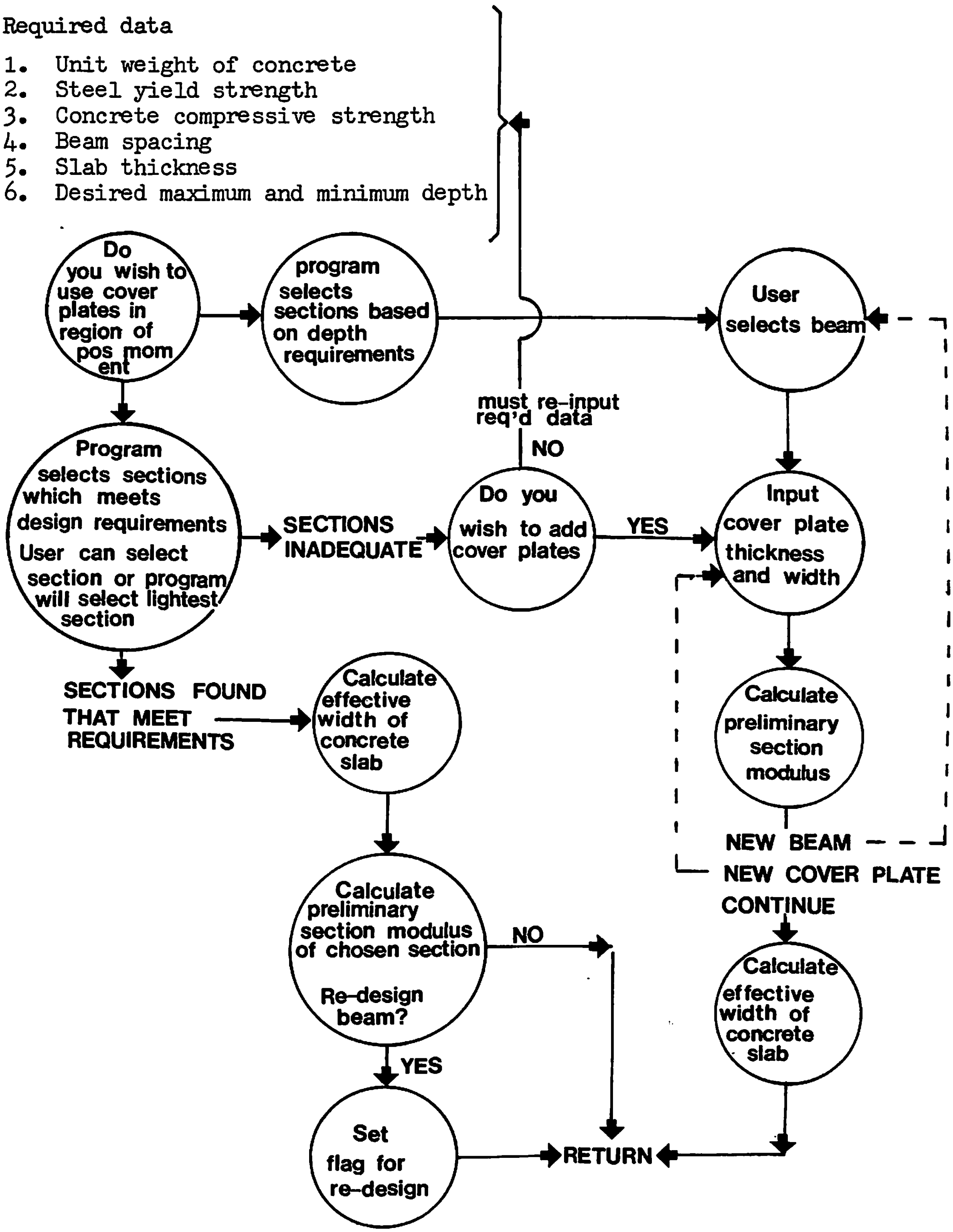


$$\frac{Xom-X(I-1,1)}{ABS(Mmax(I-1,1))} = \frac{X(I,1)-X(I-1,1)}{Mmax(I,1)-Mmax(I-1,1)}$$

$$Xom = \frac{ABS(Mmax(I-1,1))(X(I,1)-X(I-1,1))}{Mmax(I,1)-Mmax(I-1,1)} + X(I-1,1)$$

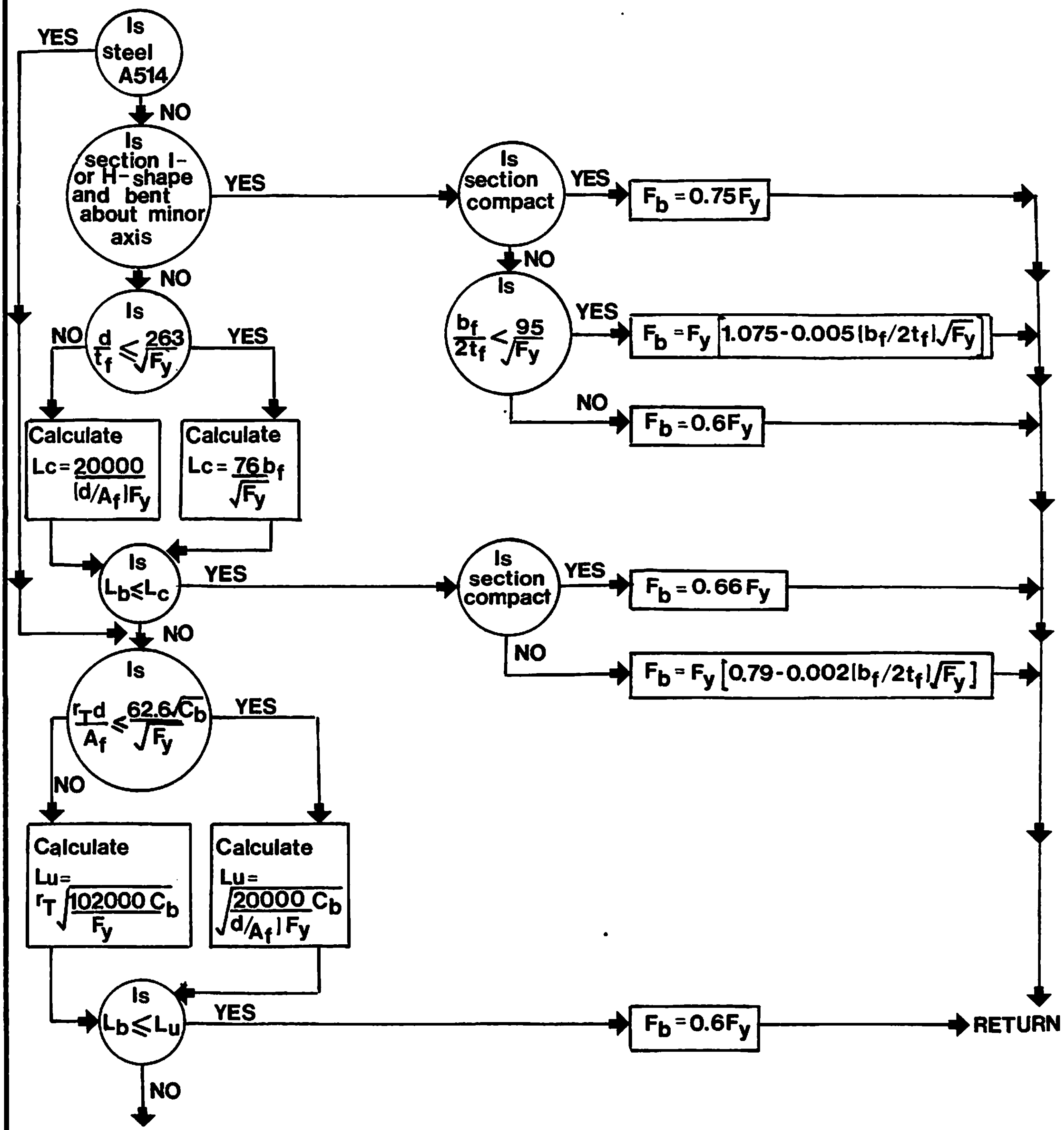
Required data

1. Unit weight of concrete
2. Steel yield strength
3. Concrete compressive strength
4. Beam spacing
5. Slab thickness
6. Desired maximum and minimum depth



Required data

1. Location of laterally braced points (ft)



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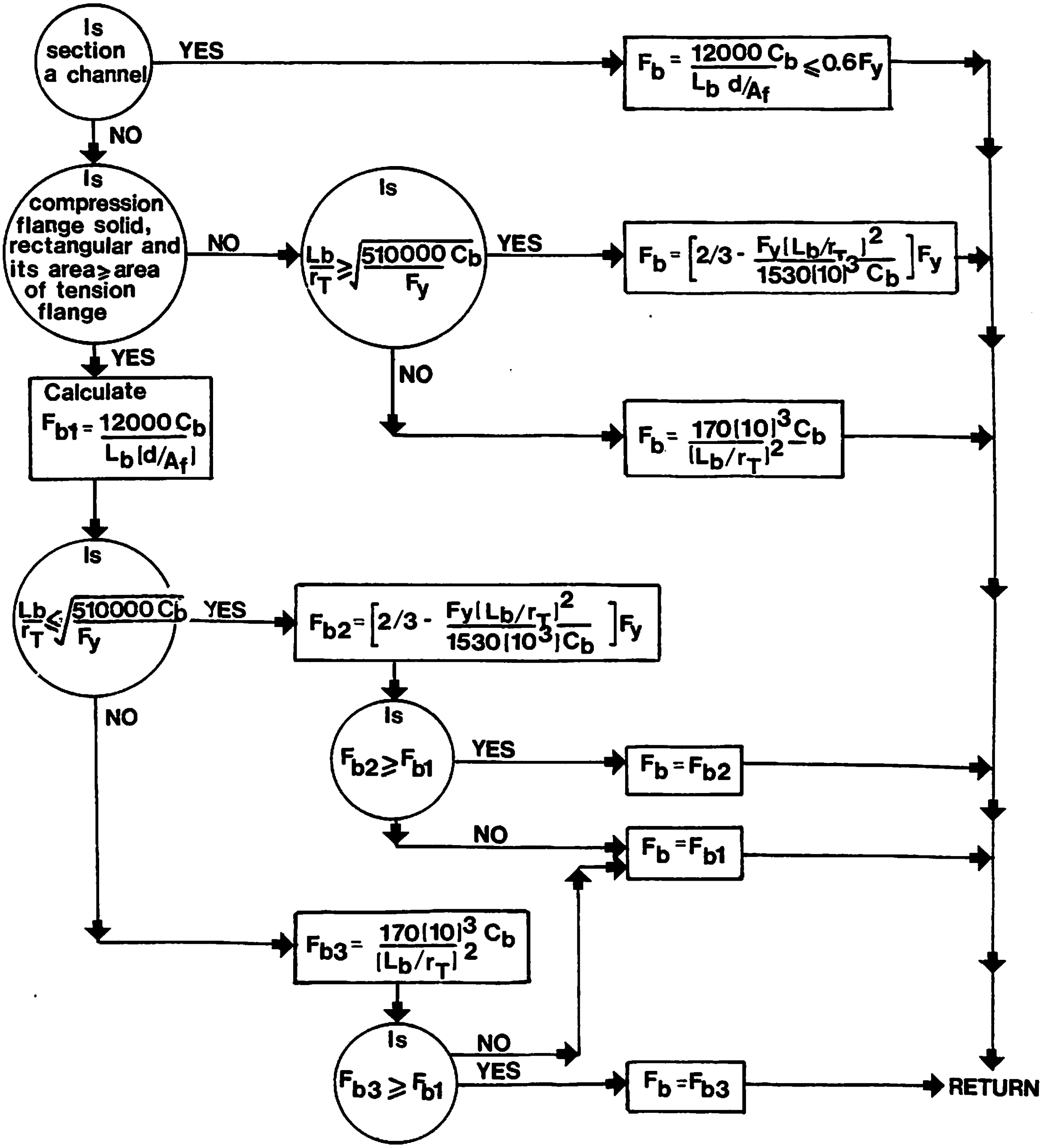
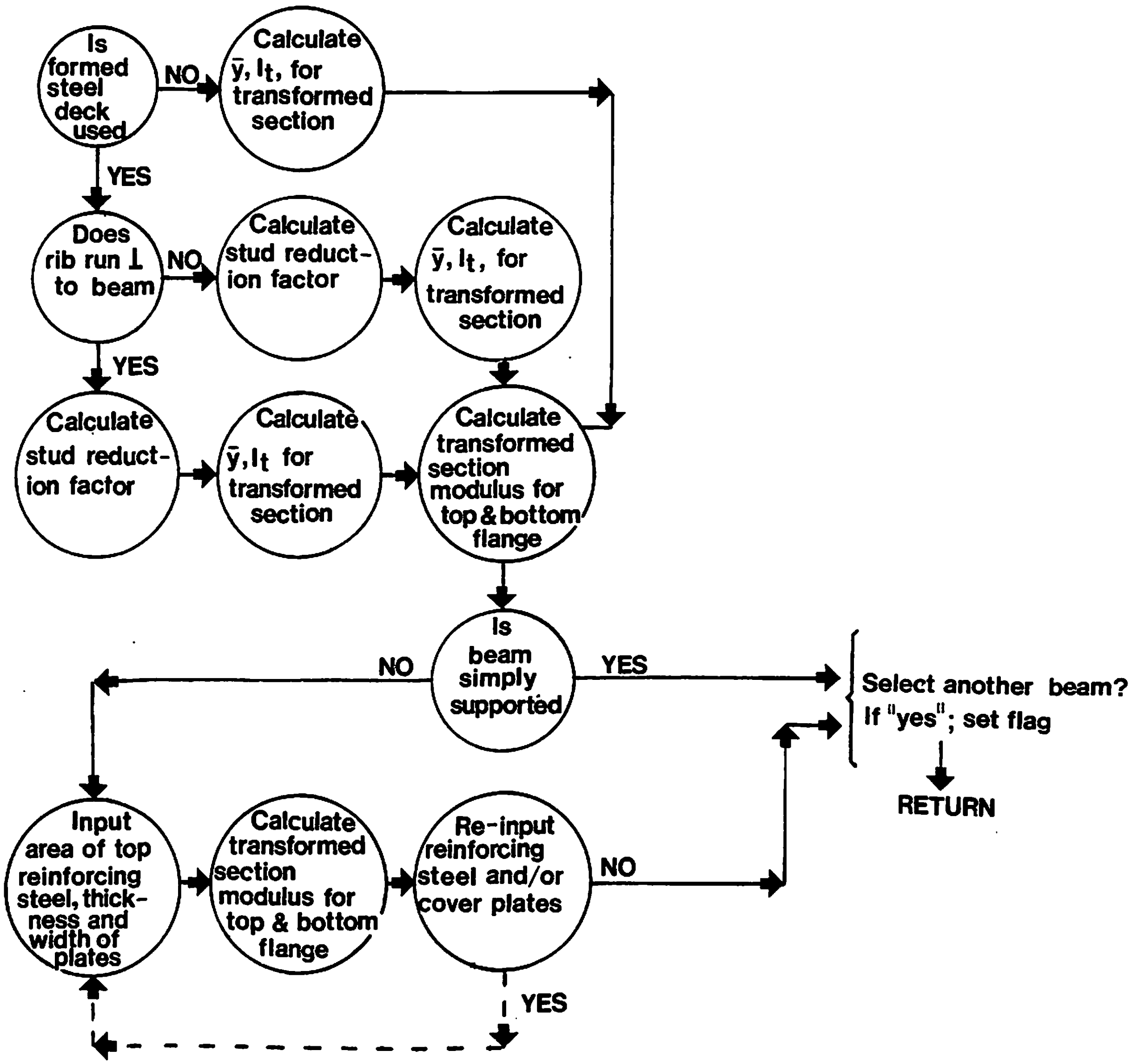
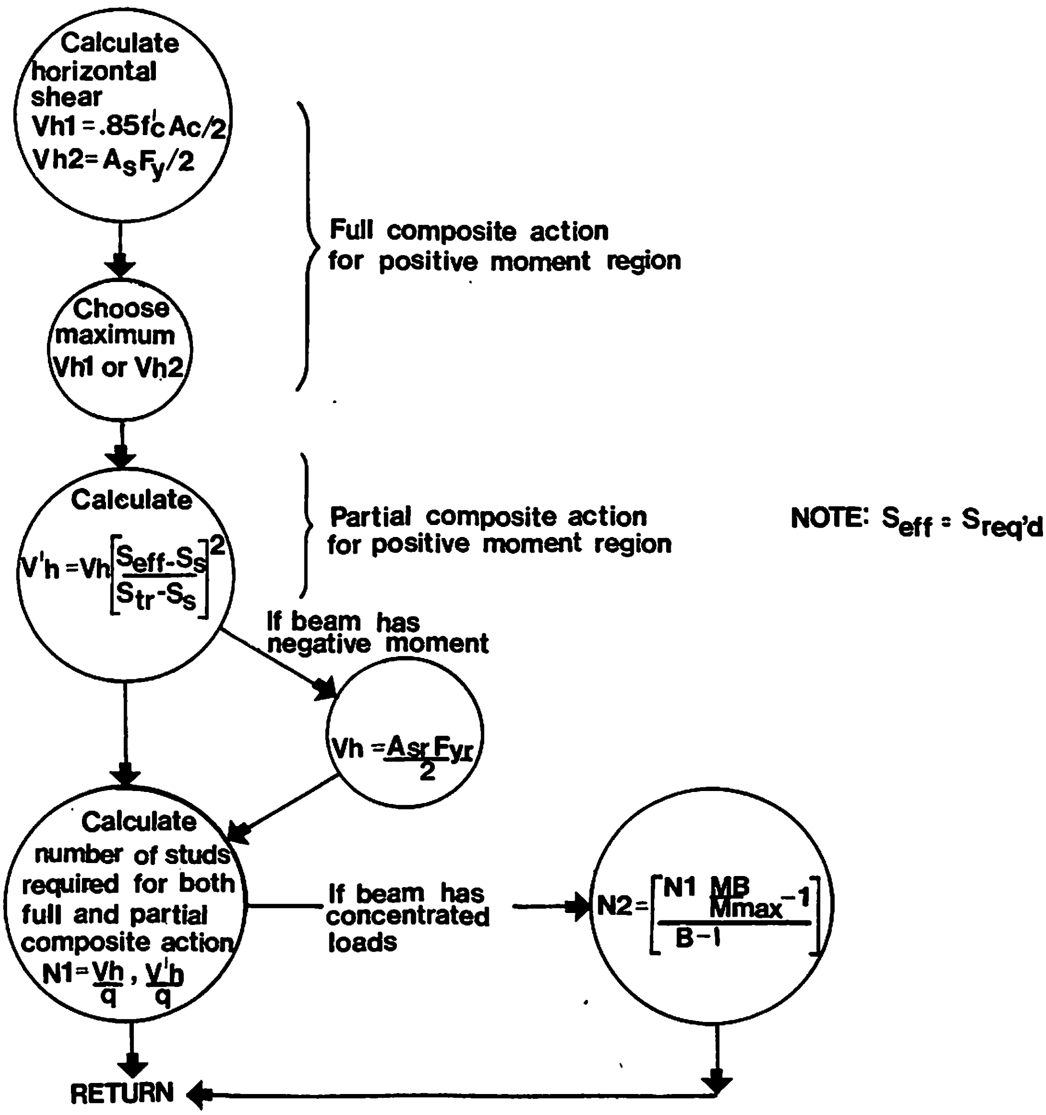
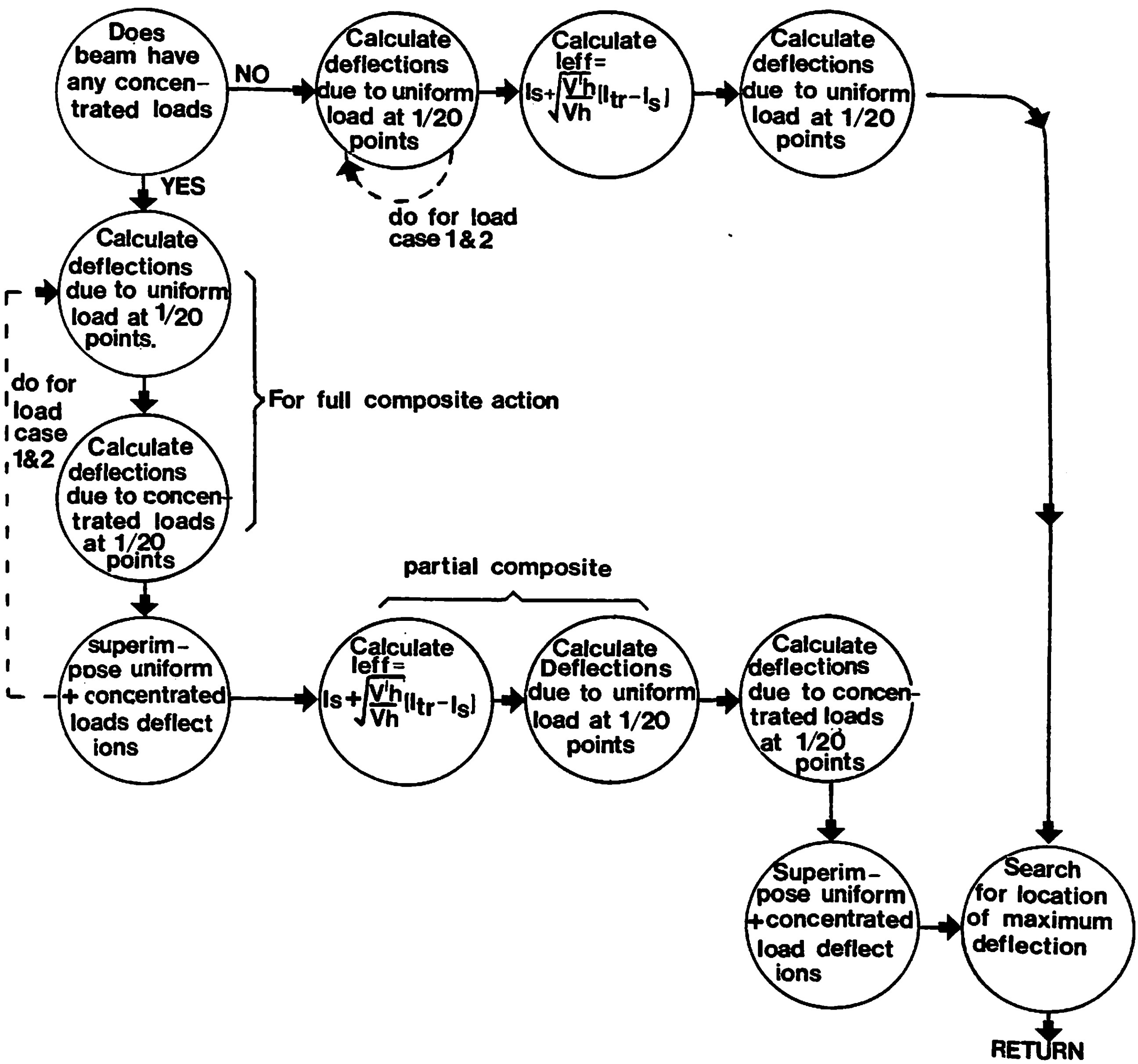

Is section a channel
YES
NO
$F_b = \frac{12000\,C_b}{L_b\,d/A_f} \leq 0.6F_y$
Is compression flange solid, rectangular and its area ≥ area of tension flange
NO
YES
Is $\frac{L_b}{r_T} \geq \sqrt{\frac{510000\,C_b}{F_y}}$
$F_b = \left[2/3 - \frac{F_y(L_b/r_T)^2}{1530(10)^3\,C_b}\right]F_y$
$F_b = \frac{170(10)^3\,C_b}{(L_b/r_T)^2}$
Calculate $F_{b1} = \frac{12000\,C_b}{L_b(d/A_f)}$
Is $\frac{L_b}{r_T} \leq \sqrt{\frac{510000\,C_b}{F_y}}$
$F_{b2} = \left[2/3 - \frac{F_y(L_b/r_T)^2}{1530(10^3)C_b}\right]F_y$
Is $F_{b2} \geq F_{b1}$
$F_b = F_{b2}$
$F_b = F_{b1}$
$F_{b3} = \frac{170(10)^3\,C_b}{(L_b/r_T)^2}$
Is $F_{b3} \geq F_{b1}$
$F_b = F_{b3}$
RETURN

Required data:
FSD orientation (if used)
Rib height
Rib width
Stud height
Rib spacing (if FSD runs parallel to beam)




Calculate horizontal shear
Vh1 = .85f'c Ac/2
Vh2 = As Fy/2
Full composite action for positive moment region
Choose maximum Vh1 or Vh2
Calculate V'h = Vh [Seff-Ss / Str-Ss]^2
Partial composite action for positive moment region
NOTE: Seff = Sreq'd
If beam has negative moment
Vh = Asr Fyr / 2
Calculate number of studs required for both full and partial composite action
N1 = Vh/q , V'h/q
If beam has concentrated loads
N2 = [N1 (MB/Mmax) - 1 / B-1]
RETURN


Does beam have any concentrated loads
NO
YES
Calculate deflections due to uniform load at 1/20 points
do for load case 1&2
Calculate $I_{eff} = I_s + \sqrt{\frac{V'h}{Vh}}(I_{tr} - I_s)$
Calculate deflections due to uniform load at 1/20 points
Calculate deflections due to uniform load at 1/20 points.
do for load case 1&2
For full composite action
Calculate deflections due to concentrated loads at 1/20 points
partial composite
superimpose uniform + concentrated loads deflections
Calculate $I_{eff} = I_s + \sqrt{\frac{V'h}{Vh}}(I_{tr} - I_s)$
Calculate Deflections due to uniform load at 1/20 points
Calculate deflections due to concentrated loads at 1/20 points
Superimpose uniform + concentrated load deflections
Search for location of maximum deflection
RETURN

OUTLINE OF AISC SPECIFICATIONS FOR COMPOSITE BEAM DESIGN

General Notes

1. The design of the steel beam is based on the assumption that composite action resists the total design moment. There are two general types of construction:

a. Shored construction
Shoring is used during the time the concrete hardens to aid in reducing initial load deflections. The flexural stress in the concrete slab due to composite action is determined from the total moment.

b. Unshored construction
Flexural stress in the concrete slab due to composite action is determined from moment due to loads after the concrete reaches its 28 day compressive strength.

Note: Shored construction must be used if

$Str > (1.35+0.35 Ml/Md) Ss$ (1.11-2)

Where Str = Section modulus of transformed section

Ml = Moment due to loads applied after concrete reaches 75% of its required strength

Md = Moment due to loads applied before concrete reaches 75% of its required strength

General Considerations

1. Composite construction is most efficient with heavy loading, relatively long spans and beams spaced as far apart as permissible. A comparrison of the cost of shear connectors versus the savings in beam weight should be made to economically base a decision of using composite design.

2. Concrete compressive stress will seldom be critical for beams if a full width slab and Fy = 36 KSI steel are used in unshored construction. It is more likely to be critical when a narrow concrete flange or Fy = 50 KSI steel is used, and is frequently

critical if both Fy = 50 KSI steel and narrow concrete flange are used.

Deflections

1. In general deflection of composite beams will usually be about 1/3 to 1/2 less than deflection of non-composite beams.

2. In practice, deflections, particularly of the steel section alone under construction loads, should be calculated and listed on the contract documents as a guide for cambering or estimating slab quantities.

3. If it is desired to investigate long term creep deflection, Itr should be based on a modular ratio, n, double that used for stress calculations.

Use of Cover Plates

1. Bottom cover plates are an effective means of increasing the strength or reducing the depth of composite beams when deflections are not critical.

Use of Formed Steel Deck (FSD)

Limitation on parameters

1. Deck rib height = maximum 3 in.
2. Average width of concrete rib = minimum 2 in.
3. Shear connectors = welded studs only, maximum 3/4 in. diameter
4. Stud length [minimum] = rib height + 1 1/2 in.
5. Effective width of concrete flange = [to be determined using total slab thickness, including ribs]
6. Slab thickness above deck = minimum 2 in.

Other Considerations

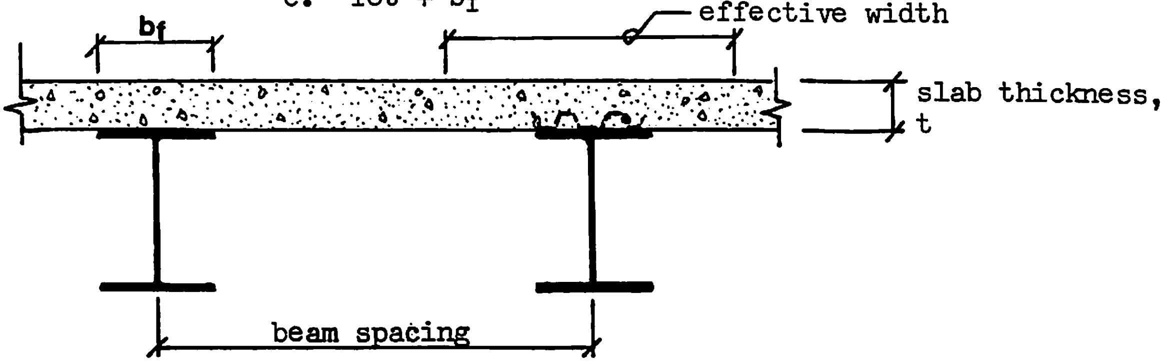
The AISC specification provisions for the design of composite beams are based on ultimate load considerations, even though they are presented in terms of working stresses. Because of this, for unshored construction, actual stresses in the tension flange of the steel beam under working load are higher than calculated stresses. The effect of formula (1.11-2) in section 1.11.2.2 is to limit the tension flange stress to 0.89 Fy.

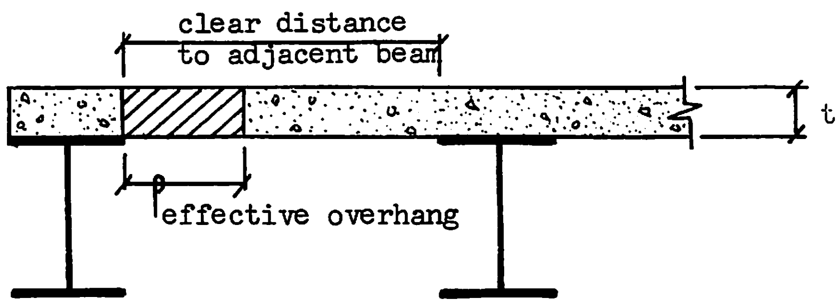
SECTION 1.11.1 AISC CODE

Effective width of concrete flange

1. When the slab extends on both sides of the beam, the effective width shall not exceed:

a. 1/4 the beam span, (1/4L)
b. The beam spacing, (S)
c. $16t + b_f$





2. For beams having a flange on one side only, the effective overhanging flange width shall not exceed:

a. 1/12 the span length of the beam (L/12)
b. 6 times the thickness of the slab (6t)
c. 1/2 the clear distance to the next beam

If $d/t_f > \frac{263}{\sqrt{Fy}}$ then $\frac{20000}{(d/Af)Fy}$ controls

If $d/t_f \leqslant \frac{263}{\sqrt{Fy}}$ then $\frac{76bf}{\sqrt{Fy}}$ controls

Controling value must be greater than unsupported length to qualify

6-7 These subparagraphs do not apply to W-sections

*Note: In calculations involving composite sections in positive moment areas the steel cross section is exempt from the compactness requirements of subparagraphs 2,3, and 5 of section 1.5.1.4.1. These subparagraphs are only applicable to negative moment areas or to the steel beam before concrete reaches 75% strength.

1.5.1.4.2 members (except hybrid girders and members of A514 steel) which meet the requirements of section 1.5.1.4.1, except that $b_f/2t_f$ exceeds $65/\sqrt{Fy}$ but is less than $95/\sqrt{Fy}$, may be designed on the basis of an allowable bending stress:

$$Fb = Fy\ [0.79-0.002(\frac{bf}{2t_f})\sqrt{Fy}] \qquad (1.5\text{-}5a)$$

$\begin{bmatrix}1.5.1.4.3\\1.5.1.4.4\end{bmatrix}$ Not applicable to this program

1.5.1.4.5 On extreme fibers of flexural members not covered in section 1.5.1.4.1 and 1.5.1.4.2

1. Tension

$$Fb = .60Fy$$

2. Compression

a. For members meeting the requirements of section 1.9.1.2, having an axis of symmetry in, and loaded in, the plane of their web, and compression on extreme fibers of channels bent about their major axis:

The larger value computed by formulas (1.5-6a) or (1.5-6b) and (1.5-7), as applicable, but not more than 0.6Fy.

When $\sqrt{\frac{102x10^3\ Cb}{Fy}} \leqslant \frac{1}{rt} \leqslant \sqrt{\frac{510x10^3\ Cb}{Fy}}$

$$Fb = \left[\frac{2}{3} - \frac{Fy\ (1/r_T)^2}{1530x10^3Cb}\right] Fy \qquad (1.5\text{-}6a)$$

SECTION 1.11.2 AISC CODE

When shear connectors are used in accordance with section 1.11.4, the composite section shall be proportioned to support all of the loads without exceeding the allowable stress prescribed in section 1.5.1.4, even when the steel section is not shored during construction.

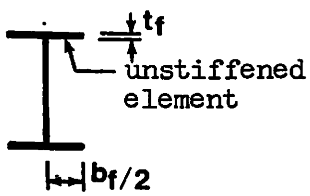
<u>Outline of section 1.5.1.4</u>

Tension and compression of extreme fibers of compact hot-rolled or built-up members symmetric about, and loaded in, the plane of their minor axis and meeting the requirements of this section:

➡ $F_b = 0.66F_y$

1. Flanges continuously connected to web.

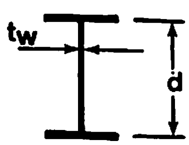
* 2. Compactness requirements: The width-thickness ratio of unstiffened projecting elements of the compression flange shall not exceed $65/\sqrt{F_y}$



$$\frac{b_f}{2t_f} \leq \frac{65}{\sqrt{F_y}}$$

* 3. Subparagraph does not apply to W-sections

4. The depth-thickness ratio of the web shall not exceed the value given by formula (1.5-4a) or (1.5-4b) as applicable. (this subparagraph intended for compression members)



$$\frac{d}{t_w} \leq \frac{640}{\sqrt{F_y}}\left(1-3.74\frac{f_a}{F_y}\right) \quad \text{when } \frac{f_a}{F_y} \leq 0.16$$

$$\frac{d}{t_w} \leq \frac{257}{\sqrt{F_y}} \quad \text{when } \frac{f_a}{F_y} > 0.16$$

* 5. The laterally unsupported length of the compression flange of members other than circular or box members shall not exceed the value: $76\, b_f/\sqrt{F_y}$ or $20000/[(d/A_f)(F_y)]$
Which one controls?

$$\frac{76b_f}{\sqrt{F_y}} = \frac{20000}{(d/A_f)F_y} = \frac{20000}{(d/(b_f t_f))F_y} \qquad \frac{d}{t_f} = \frac{263}{\sqrt{F_y}}$$

When $\frac{l}{r_T} \geq \sqrt{\frac{510\mathrm{x}10^3 Cb}{Fy}}$

$$Fb = \frac{170\mathrm{x}10^3 Cb}{Fy} \qquad (1.5\text{-}6b)$$

Or, when the compression flange is solid and approximately rectangular in cross section and its area is not less than that of the tension flange:

$$Fb = \frac{12\mathrm{x}10^3 Cb}{ld/A_f} \qquad (1.5\text{-}7)$$

Nomenclature

l = Distance between cross sections braced against twist or lateral desplacement of the compression flange, inches

r_T = Radius of gyration of a section comprising the compression flange + 1/3 of the compression web area, taken about an axis in the plane of the web, inches

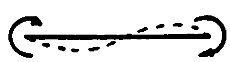
A_f = Area of the compression flange, square inches

Cb = $1.75 + 1.05\ (M_1/M_2) + 0.3\ (M_1/M_2)^2 \leq 2.3$

M_1 = The smaller bending moment at one end of the unbraced length

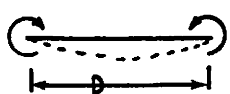
M_2 = The larger bending moment at one end of the unbraced length

M_1/M_2: Positive when M_1 and M_2 cause reverse curvature bending



Reverse curvature bending

M_1/M_2: Negative when M_1 and M_2 cause single curvature bending.



Unbraced length

When the bending moment at any point within an unbraced length is larger than that at both ends of this length, the value of Cb shall be taken as unity.

b. For members meeting the requirements of section 1.9.1.2, but not included in sub-paragraph 2a of this section:

$Fb = 0.6Fy$

Provided that sections bent about their major axis are braced laterally in the region of compression stress at intervals not exceeding $76\ b_f/\sqrt{Fy}$

For this requirement to be satisfied from equation 1.5-7 $Fb = \dfrac{12x10^3\ Cb}{ld/A_f}$, let $Fb = 0.6Fy$

$$l \leqslant \frac{12x10^3\ Cb}{d/Af\ (0.6Fy)} = \frac{20000\ Cb}{(d/Af)Fy} \quad (1.)$$

Also $\dfrac{l}{r_T} = \sqrt{\dfrac{102x10^3\ Cb}{Fy}}$

$$l \leqslant \frac{r_T\ (319.375\sqrt{Cb})}{\sqrt{Fy}} \quad (2.)$$

Which equation controls?
Setting (1.) = (2.)

$$\frac{r_T\ (319.375\sqrt{Cb})}{\sqrt{Fy}} = \frac{20000\ \ Cb}{(d/A_f)Fy}$$

$$\frac{r_T\ d}{A_f} = \frac{62.6}{\sqrt{Fy}}\sqrt{Cb}$$

If $\dfrac{r_T d}{A_f} > \dfrac{62.6\sqrt{Cb}}{\sqrt{Fy}}$ then $r_T\sqrt{\dfrac{102x10^3\ Cb}{Fy}}$ controls

If $\dfrac{r_T d}{A_f} \leqslant \dfrac{62.6\sqrt{Cb}}{\sqrt{Fy}}$ then $\sqrt{\dfrac{20000\ \ Cb}{(d/A_f)\ Fy}}$ controls

In other words in order to quality for the 0.6Fy allowable bending stress we need check the following:

Check to see if $\dfrac{r_T d}{A_f} \leqslant \dfrac{62.6\sqrt{Cb}}{\sqrt{Fy}}$

If this requirement is met, we need to check

$$lb \leqslant \sqrt{\frac{20000\ Cb}{(d/A_f)Fy}} \quad \Rightarrow \text{use } 0.6Fy$$

If this requirement is not met then we need to check

$$lb \leqslant r_T\sqrt{\frac{102x10^3\ Cb}{Fy}} \Rightarrow \text{use } 0.6Fy$$

Refer to the bendstress flow chart for a graphic description on allowable bending stress

MOMENT GRADIENT MULTIPLIER/RADIUS OF GYRATION

Cb Ranges from 1.0 to 2.3

Cb Can only be applied if the beam is loaded in the plane of its web, is bent about its major axis, and the rectangular compression flange has an area not less than that of the tension flange

Cb = 1.0 This assumes that the compressive bending stress remains constant between braced points or that its maximum value is located somewhere between the braced points and not at the ends

AISC provides the following formula for obtaining Cb

$$Cb = 1.75 + 1.05\left[\frac{M_1}{M_2}\right] + 0.3\left[\frac{M_1}{M_2}\right]^2 \leqslant 2.3$$

Where -
M_1 is the smaller and M_2 the larger bending moment at the ends of the unbraced length taken about the strong axis of the member

The ratio (M_1/M_2) is positive for reverse curvature bending and negative for single curvature bending.

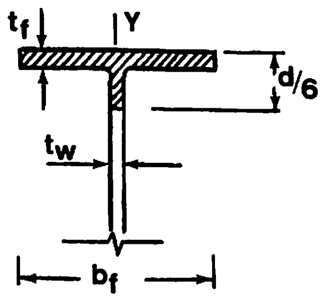
Radius of Gyration

The proportions of the compression flange when referenced to the Y axis of the beam, is important when considering a shape's resistance to buckling.

For the purposes of calculating allowable bending stress

$$r_T = \sqrt{\frac{I}{A}}$$

Where I = Moment of inertia of the compression flange and 1/3 of the compression web, with respect to the Y axis



$$I_{flange} = 2\,\frac{(t_f)(b_f/2)^3}{12} \qquad I_{web} = \frac{2(d/6)(t_w/2)^3}{12}$$

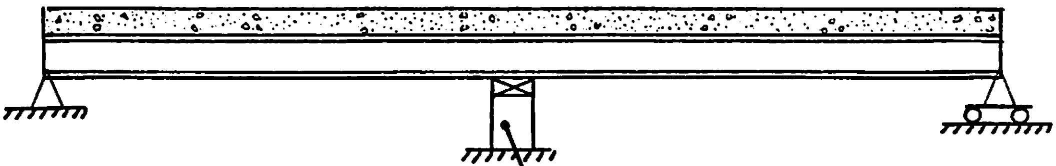
$$A_{flange} = (b_f)(t_f) \qquad A_{web} = (d/6)(t_w)$$

$$I = I_{flange} + A_{flange}\,(b_f/4)^2 + I_{web} + A_{web}\left[\frac{t_w}{4}\right]^2$$

$$A = A_{flange} + A_{web}$$

Note: The fillet at the flange/web intersection is neglected.

Shored Construction



shoring used between structural supports before concrete reaches 75% strength.

When the shoring is removed after concrete reaches 75% strength, the concrete compressive stress results from initial loads + loads applied after 75% hardening.

$$fc = \frac{Mtotal}{n\ Stop} \leqslant 0.45f'c$$

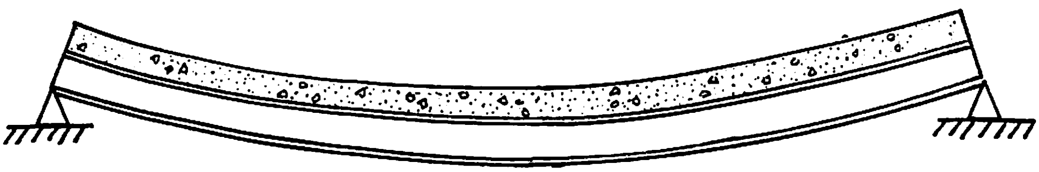
Where

Mtotal = Moment resulting from all loads (before and after concrete reaches 75% strength)

Stop = Section modulus with respect to the top

Note: Shored construction must be used if:
$$Str > \left(1.35 + 0.35\frac{Mlive}{Mdead}\right) Ss$$

Unshored Construction



The concrete, when shoring is not used, only takes load after it reaches 75% strength and load is applied to it.

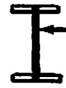
$$fc = \frac{Mlive}{n\ Stop} \leqslant 0.45\ f'_c$$

Where

Mlive = Moment resulting from loads applied after concrete reaches 75% strength

The steel beam alone must resist bending stresses resulting from initial loads. $fb = \frac{Mdead}{Ss} \leqslant Fb$ Where Ss = Section modulus of steel beam

It must also resist the total shear stress caused by live + dead loads. $fv = \frac{Vtot}{Aweb} \leqslant Fv$


Area used in shear

The composite section must resist bending stresses resulting from total (live + dead) loads. $fb = \frac{Mtot}{Str}$ Fb

The actual section modulus of the transformed section shall be used in calculating the concrete flexural compression stress and this stress shall be based upon loading applied before and after concrete has reached 75% of its required strength and shall not exceed $0.45f'_c$.

Construction Without Temporary Shoring

The actual section modulus of the transformed section shall be used in calculating the concrete flexural compression stress and this stress shall be based upon loading applied after concrete has reached 75% of its required strength and shall not exceed $0.45f'_c$.

1.11.3 End Shear

The web and the end connections of the steel beam shall be designed to carry the total reaction.

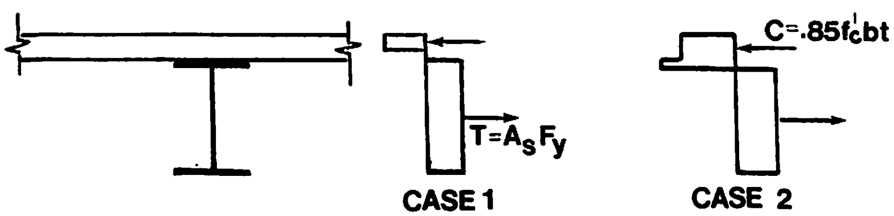
1.11.4 Shear Connectors

The entire horizontal shear at the junction of the steel beam and the concrete slab shall be assumed to be transferred by shear connectors welded to the top flange of the beam and embedded in the concrete. For full composite action with concrete subject to flexural compression, the total horizontal shear to be resisted between the point of maximum positive moment and points of zero moment shall be taken as the smaller value using formulas (1.11-3) and (1.11-4)

$$Vh = \frac{0.85f'_c Ac}{2}$$

$$Vh = \frac{A_s F_y}{2}$$

These equations were derived using ultimate strength theory. Their derivation will be briefly outlined. (Wang and Solmon), (Tall) Consider positive bending:



Case 1: Concrete slab is adequate to resist the total compressive force at ultimate load.

Case 2: Concrete slab is not adequate to resist the total compressive force at ultimate load and a portion of it must be resisted by the flange of the steel beam.

The total horizontal resisting shear at the slab-beam interface must equal the total compressive force in the slab.

For Case 1

$V_h = C = A_sF_y$

Assuming that the compressive force at design load will be 1/2 that at ultimate load.

$$V_h = \frac{A_sF_y}{2} \qquad (1.11\text{-}4)$$

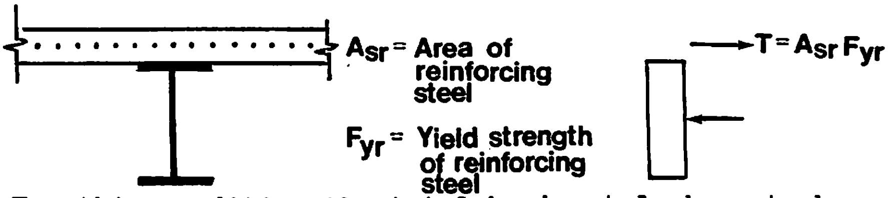
For Case 2

$V_h = C = 0.85f'_cbt$ bt = Area of the transformed concrete. Again, assuming that the compressive force at design load will be 1/2 that at ultimate load.

$$V_h = \frac{0.85f'_cA_c}{2} \qquad (1.11\text{-}3)\ *$$

The smaller horizontal shear causing either Case 1 or Case 2 will control.

Consider negative bending:



For this condition the total horizontal shear to be resisted at the slab-beam interface is equal to the tensile force in the reinforcing steel.

$V_h = T = A_{sr} F_{yr}$

Assuming that the tensile force at design load will be 1/2 that at ultimate load.

$$V_h = \frac{A_{sr} F_{yr}}{2} \qquad (1.11\text{-}5)$$

This equation is applicable in continuous beams where longitudinal reinforcing steel is considered to act compositely with the steel beam in the negative moment regions. This shear is to be resisted by shear connectors between an interior support and each adjacent point of contraflexure.

* The term 1/2 $A_{sr}F_{yr}$ should be added to the right hand side of formula (1.11-3) if longitudinal steel is located within the effective width of concrete flange.

Required number of shear connectors:

For full composite action, the number of connectors required to resist the horizontal shear, shall not be less than that determined by the relationship Vh/q where q is the allowable shear load for one connector.

For partial composite action with concrete subject to flexural compression, the horizontal shear, $V'h$, to be used in computing Seff shall be taken as the product q times the number of connectors furnished between the point of maximum moment and the nearest point of zero moment.

Determine the horizontal shear to be resisted to qualify for partial composite action. Solve equation 1.11-1 for $V'h$

$$Seff = Ss + \sqrt{\frac{V'h}{Vh}}\ (Str - Ss)$$

$$\sqrt{\frac{V'h}{Vh}} = \frac{Seff - Ss}{Str - Ss}$$

$$V'h = \left[\frac{Seff - Ss}{Str - Ss}\right]^2 (Vh)$$

Vh = Horizontal shear to be resisted using full composite action. Eqn. (1.11-3) or (1.11-4), whichever is smaller

Ss = Section modulus of steel beam

Str = Transformed section modulus of transformed section

Seff = Effective section modulus of transformed section. In this relationship, this section modulus must be at least equal to the required section modulus determined from actual loading conditions, that cause the severest condition

$$Sreqd = \frac{M}{Fb}$$

Rewriting ...

$$V'h = \left[\frac{Sreqd - Ss}{Str - Ss}\right]^2 (Vh)$$

The value of $V'h$ shall not be less than 1/4 the smaller value of formula (1.11-3), using the maximum permitted effective width of the concrete flange, or formula (1.11-4).

The effective moment of inertia for deflection computations shall be determined by:

$$I_{eff} \quad I_s + \sqrt{\frac{V'_h}{V_h}}\,(I_t - I_s)$$

Is = Moment of inertia of the steel beam

It = Moment of inertia of the transformed composite section

Effect of Concentrated Loads on Horizontal Shear

The connectors required each side of the point of maximum moment in an area of positive bending may be uniformly distributed between that point and adjacent points of zero moment, except that N2, the number of shear connectors required between any concentrated load in that area and the nearest point of zero moment, shall be not less than that determined by formula (1.11-7)

$$N_2 = \frac{N_1\left[\frac{M\beta}{M_{max}} - 1\right]}{\beta - 1} \qquad (1.11\text{-}7)$$

M = Moment (less than maximum moment) at a concentrated load point

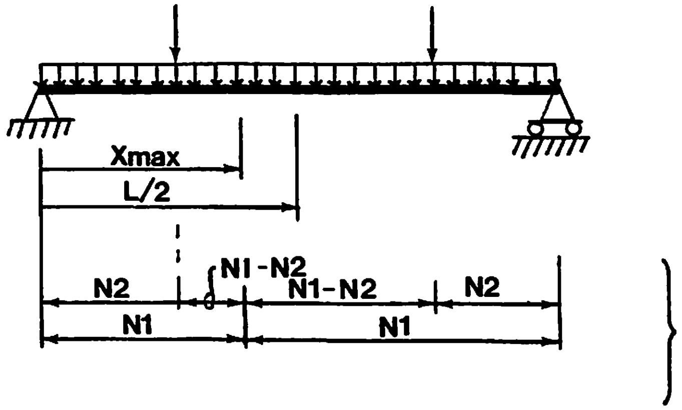
N_1 = Number of connectors required between point of maximum moment and point of zero moment, determined by the relationship V_h/q (full composite action) or V'_h/q (partial composite action), as applicable.

β = $\frac{S_{tr}}{S_s}$ (full composite action) or $\frac{S_{eff}}{S_s}$ (partial composite action)

For a continuous beam, connectors required in the region of negative bending may be uniformly distributed between the point of maximum moment and each point of zero moment.

Shear connectors shall have at least 1 inch of lateral cover, except for connectors installed in the ribs of formed steel decks. Unless located directly over the web, the diameter of studs shall not be greater than 2 1/2 times the thickness of the flange to which they are welded. The minimum center to center spacing of stud connectors shall be 6 diameters along the longitudinal axis of the supporting composite beam and 4 diameters transverse to the longitudinal axis of the supporting composite beam. The maximum center to center spacing of stud connectors shall not exceed 8 times the total slab thickness.

After the stud spacing has been adjusted to place the studs between the applicable concentrated load and the end of the beam (zero moment point), the balance ($N_1 - N_2$) studs are placed between the concentrated load and the point of maximum moment.

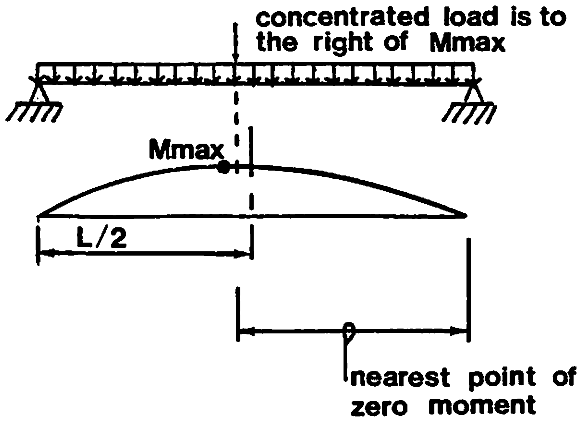


Xmax = Location of maximum moment

Distribution of shear studs along beam

N1 = Number of studs required between point of maximum moment and adjacent points of zero moment.

N2 = Number of studs required between concentrated load(s) and nearest point of zero moment. Note that the nearest point of zero moment is that point on the same side of the maximum moment as the concentrated load.

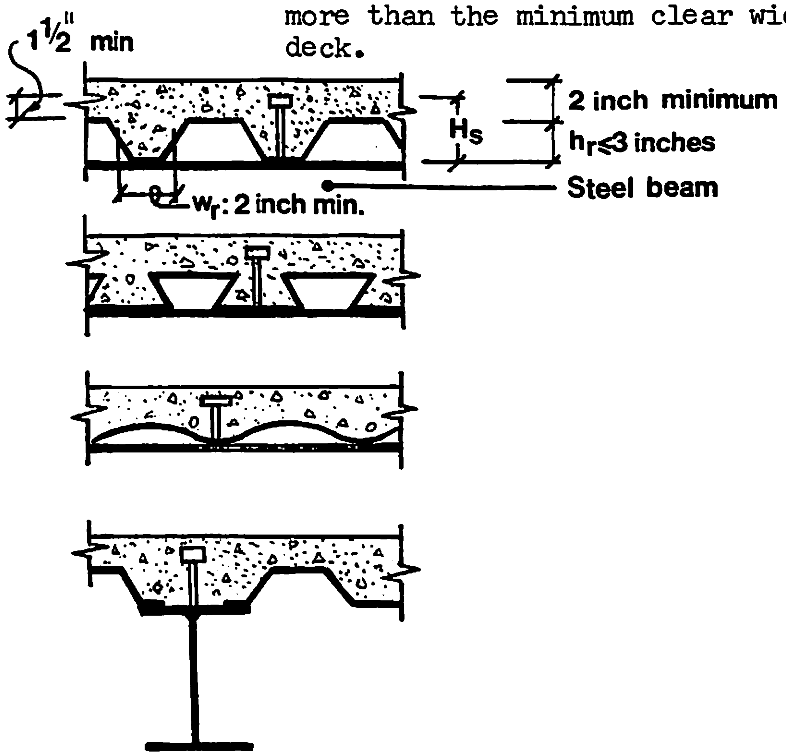


1.11.5
Composite Beams or Girders With Formed Steel Deck

1.11.5.1 General

1. Section 1.11.5 is applicable to decks with nominal rib height not greater than 3 inches.

2. The average width of concrete rib, Wr, shall be not less than 2 inches, but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.



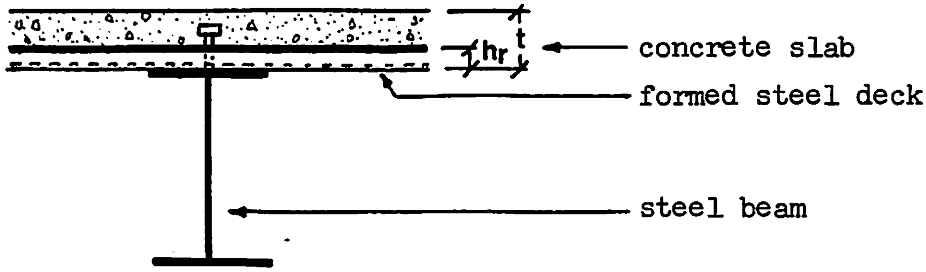
3. The concrete slab shall be connected to the steel beam or girder with welded stud shear connectors 3/4 inch or less diameter. Studs may be welded through the deck or directly to the steel member.

4. Stud shear connectors shall extend not less than 1 1/2 inches above the top of the steel deck after installation.

5. The total slab thickness, including ribs, shall be used in determining the effective width of concrete flange.

6. Slab thickness above steel deck shall be not less than 2 inches.

1.11.5.2 Deck Ribs Oriented Perpendicular to Steel Beam or Girder



1. Concrete below the top of the steel deck shall be neglected when determining section properties and in calculating A_c for formula (1.11-3).

2. The spacing of stud shear connectors along the length of a supporting beam or girder shall not exceed 32 inches.

3. The allowable horizontal shear load per stud connector, q, shall be the value stipulated in sect. 1.11.4 multiplied by the following reduction factor:

$$\left[\frac{0.85}{\sqrt{Nr}}\right]\left[\frac{Wr}{hr}\right]\left[\frac{Hs}{hr} - 1.0\right] \leqslant 1.0$$

Where

hr = Nominal rib height, inches

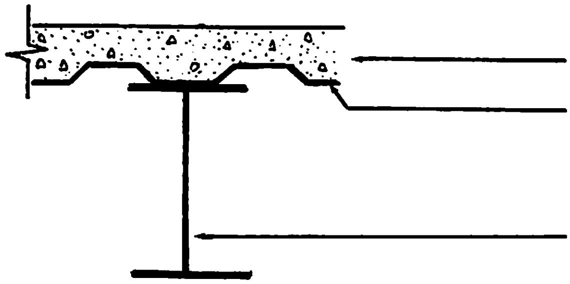
Hs = Length of stud connector after welding, inches, not to exceed the value (hr + 3) in computations, although the actual length may be greater.

Nr = Number of stud connectors on a beam in one rib, not to exceed 3 in computations although more than 3 studs may be installed.

Wr = Average width of concrete rib, inches.

4. To resist uplift, the steel deck shall be anchored to all compositely designed steel beams or girders at a spacing not to exceed 16 inches. Such anchorage may be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other devices specified by the designer.

1.11.5.3 Deck Ribs Oriented Parallel to Steel Beam or Girder



1. Concrete below the top of the steel deck may be included when determining section properties and shall be included in calculating A_c for formula (1.11.3)

2. Steel deck ribs over supporting beams or girders may be split longitudinally and seperated to form a concrete haunch.

3. When the nominal depth of steel deck is 1 1/2 inches or greater, the average width, Wr, of the supported haunch or rib shall be not less than 2 inches for the first stud in the transverse row plus 4 stud diameters for each additional stud.

4. The allowable horizontal shear load per stud connector, q, shall be the value stipulated in sect. 1.11.4, except that when the ratio Wr/hr is less than 1.5, the allowable load shall be multiplied by the following reduction factor:

$$0.6\left[\frac{Wr}{hr}\right]\left[\frac{Hs}{hr} - 1.0\right] \leq 1.0$$

   Where hr and Hs are as defined in sect. 1.11.5.2 and Wr is the average width of concrete rib or haunch.

1.11.6 Special Cases

When composite construction does not conform to the requirements of sects. 1.11.1 through 1.11.5, allowable load per shear connector must be established by a suitable test program.

The Following Section Contains Equations Used to Locate the Neutral Axis of the Composite Section and Calculate the Moment of Inertia

Compression in the Top Flange for the Given Conditions:

1. Concrete slab only
2. Formed steel deck running parallel to beam
3. Formed steel deck running perpendicular to beam

Assumptions:

1. Concrete in tension is ineffective
2. The formed steel deck does not contribute to the transformed section
3. Width of formed steel deck is constant

Note: Section properties are calculated assuming cover plates are used. If cover plates are not used, their contribution simply drops out of the equation.

Momenclature

1. Areas

As	=	Area of steel section
Acp	=	Area of cover plate
Atc	=	Area of transformed concrete. (for sections with formed steel deck this area is divided into A1tc and A2tc)

2. Lengths

d	=	Depth of steel section
t	=	Thickness of slab from top of beam flange
hr	=	Height of formed steel deck
wr	=	Width of formed steel deck
be	=	Effective width of concrete slab
tcp	=	Thickness of cover plate
wcp	=	Width of cover plate

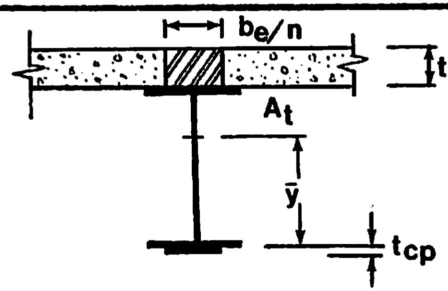
3. Properties

$\bar{y}$	=	Distance to neutral axis of composite section with respect to bottom of steel beam
$\bar{y}t$	=	Distance to neutral axis of composite section with respect to top of concrete slab
It	=	Moment of inertia of composite section with respect to its centroid. (for sections with formed steel deck this moment of inertia is

divided into I1tc and I2tc)

Icp = Moment of inertia of cover plate with respect to its centroid

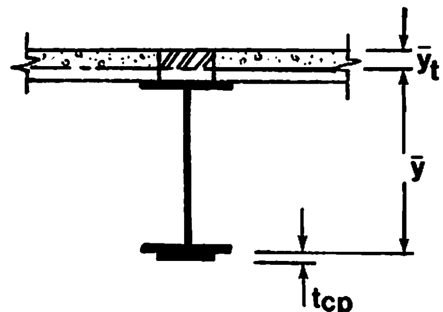
Is = Moment of inertia of steel beam with respect to its centroid



$A_{tc} = b_e/n$

(1.) Neutral axis of composite section in steel beam

$$\bar{y} = \frac{\Sigma A\bar{y}}{\Sigma A} = \frac{Atc(d+t/2)+As(d/2)+Acp(-tcp/2)}{Atc+As+Acp}$$



(2.) Neutral axis of composite section in concrete slab

$$\bar{y}t = \frac{\Sigma A\bar{y}}{\Sigma A} = \frac{\bar{y}t(be/n)(\bar{y}t/2)+As(t+d/2)+Acp(t+d+tcp/2)}{\bar{y}t(be/n)+As+Acp}$$

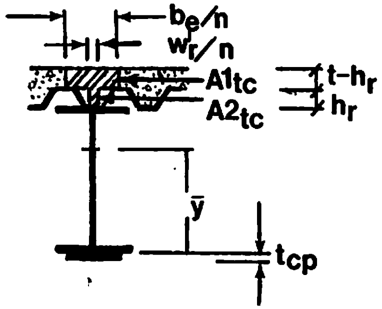
$$\bar{y}t[\bar{y}t(be/n)+As+Acp] = \frac{\bar{y}t^2 be}{2n}+As(t+d/2)+Acp(t+d+tcp/2)$$

$$\bar{y}t^2\left(\frac{be}{2n}\right)+\bar{y}t(As+Acp)-As(t+d/2)-Acp(t+d+tcp/2)=0$$

Let $a=\frac{be}{2n}$ $\quad b=As+Acp$ $\quad c=-As(t+d/2)-Acp(t+d+tcp/2)$

$$\bar{y}t=\frac{-b+\sqrt{b^2-4ac}}{2a}$$

$$\bar{y}=t+d-\bar{y}t$$



Procedure for determing A2tc:

# of rib widths ($w'r$) in effective width= be/(Rib spacing)

$$A2tc = \frac{hr}{n}\left[\frac{be(wr)}{\text{Rib spacing}}\right]$$

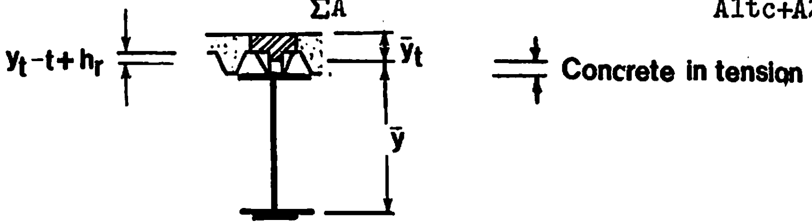
$$\text{LET } w'r = \left[\frac{be(wr)}{\text{Rib spacing}}\right]$$

$$A2tc = \frac{w'r(hr)}{n}$$

1. Neutral axis of composite section in steel beam

$$A1tc = (be/n)(t-hr)$$

$$\bar{y} = \frac{\Sigma A\bar{y}}{\Sigma A} = \frac{A1tc(d+hr+(t-hr)/2)+A2tc(d+hr/2)+As(d/2)+Acp(-tcp/2)}{A1tc+A2tc+As+Acp}$$



$$\bar{y}t = \frac{-b+\sqrt{b^2-4ac}}{2a}$$

2. Neutral axis of composite section within rib height of FSD

$$A1tc = (be/n)(t-hr)$$

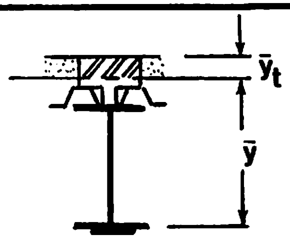
$$yt = \frac{\Sigma A\bar{y}}{\Sigma A} = \frac{A1tc(t-hr)/2+(\bar{y}t-(t-hr))(w'r/n)(\bar{y}t-(\bar{y}t-(t-hr))/2)+As(t+d/2)+Acp(t+d+tcp/2)}{A1tc+(\bar{y}t-(t-hr))(w'r/n)+As+Acp}$$

Solving the quadratic...

$$a = w'r/2n$$

$$b = A1tc+As+Acp+w'r(hr)/n-(w'r)t/n$$

$$c = \frac{t^2w'r}{2n} + \frac{hr^2w'r}{2n} - \frac{t(hr)w'r}{n} - \frac{A1tc(t-hr)}{2} - As(t+d/2) - Acp(t+d+tcp/2)$$



(3) Neutral axis of composite section above rib height of FSD

$$\bar{y}t = \frac{\Sigma A\bar{y}}{\Sigma A} = \frac{\bar{y}t(be/n)(\bar{y}t/2) + As(t+d/2) + Acp(t+d+tcp/2)}{\bar{y}t(be/n) + As + Acp}$$

$$\bar{y}t[\bar{y}t(be/n) + As + Acp] = \bar{y}t^2(be/2n) + As(t+d/2) + Acp(t+d+tcp/2)$$

$$\bar{y}t^2(be/2n) + \bar{y}t(As+Acp) - As(t+d/2) - Acp(t+d+tcp/2) = 0$$

Setting up the quadratic equation...

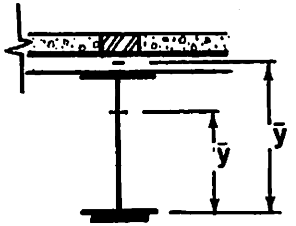
$a = be/2n$

$b = As+Acp$

$c = -As(t+d/2) - Acp(t+d+tcp/2)$

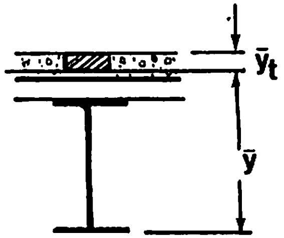
$$\bar{y}t = \frac{-b + \sqrt{b^2 - 4ac}}{2a}$$

$\bar{y} = d+t-\bar{y}t$



(1.) Neutral axis of composite section in steel beam or within height of FSD

$$\bar{y} = \frac{Atc(d+hr+(t-hr)/2)+As(d/2)+Acp(-tcp/2)}{Atc+As+Acp}$$



(2.) Neutral axis of composite section above rib height of FSD

$$\bar{y}t = \frac{(be/n)\bar{y}t(\bar{y}t/2)+As(t+d/2)+Acp(t+d+tcp/2)}{(be/n)\bar{y}t+As+Acp}$$

$$\bar{y}t[(be/n)\bar{y}t+As+Acp] = \bar{y}t^2(be/2n)+As(t+d/2)+Acp(t+d+tcp/2)$$

$$\bar{y}t^2(be/2n)+\bar{y}t(As+Acp)-As(t+d/2)-Acp(t+d+tcp/2)$$

$a = be/2n$

$b = As+Acp$

$c = -As(t+d/2)-Acp(t+d+tcp/2)$

$$\bar{y}t = \frac{-b + \sqrt{b^2-4ac}}{2a}$$

$\bar{y} = d+t-\bar{y}t$

1. Concrete slab only (Neutral axis in steel beam)

$$Itc = \frac{(be/n)t^3}{12}$$

$$It = Itc + Atc(d+t/2-\bar{y})^2 + Is + As(\bar{y}-d/2)^2 + Icp + Acp(\bar{y}+tcp/2)^2$$

2. Concrete slab only (Neutral axis (NA) in concrete slab)

$$Atc = (be/n)(\bar{y}t)$$

$$Itc = \frac{(be/n)(\bar{y}t)^3}{12}$$

$$It = Itc + Atc(\bar{y}t/2)^2 + Is + As(\bar{y}-d/2)^2 + Icp + Acp(\bar{y}+tcp/2)^2$$

3. Formed steel deck running parallel to beam (NA in steel beam)

$$A1tc = (t-hr)(be/n)$$

Note: $w'r = \dfrac{be(wr)}{\text{Rib spacing}}$

$$A2tc = (hr)(w'r/n)$$

$$I1tc = \frac{(be/n)(t-hr)^3}{12}$$

$$I2tc = \frac{(w'r/n)(hr)^3}{12}$$

$$It = I1tc + A1tc(d+hr+(t-hr)/2 - \bar{y})^2 + I2tc + A2tc(d+hr/2 - \bar{y})^2 + Is + As(\bar{y}-d/2)^2 + Icp + Acp(\bar{y}+tcp/2)^2$$

4. FSD running parallel to beam (NA within rib height of FSD)

$$A1tc = (t-hr)(be/n)$$

$$A2tc = (\bar{y}t-(t-hr))(be/n)$$

$$I1tc = (be/n)(t-hr)^3/12$$

$$I2tc = (w'r/n)(\bar{y}t-t+hr)^3/12$$

$$It = I1tc + A1tc(\bar{y}t-t/2+hr/2)^2 + I2tc + A2tc\,((\bar{y}t-t+hr)/2)^2 + Is + As(\bar{y}-d/2)^2 + Icp + Acp(\bar{y}+tcp/2)^2$$

5. FSD running parallel to beam (NA above rib height of FSD)

$$Atc = (be/n)(\bar{y}t)$$

$$Itc = (be/n)(\bar{y}t)^3/12$$

$$It = Itc + Atc(\bar{y}t/2)^2 + Is + As(\bar{y}-d/2)^2 + Icp + Acp(\bar{y}+tcp/2)^2$$

6. FSD running perpendicular to beam (NA below height of FSD)

$Atc = (be/n)(t-hr)$

$Itc = (be/n)(t-hr)^3/12$

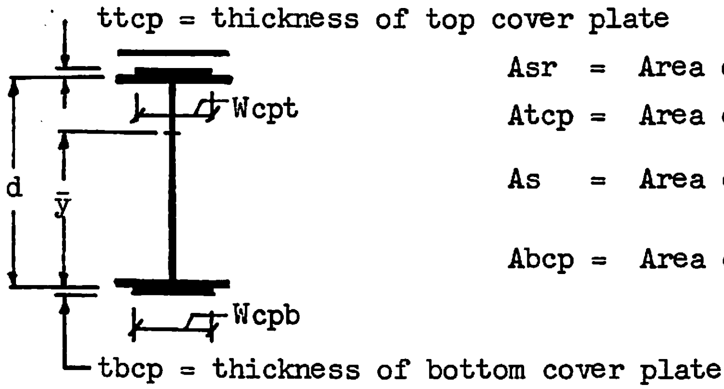
$It = Itc + Atc(d+hr+(t-hr)/2 - \bar{y})^2 + Is + As(\bar{y}-d/2)^2 + Icp + Acp(\bar{y}+tcp/2)^2$

7. FSD running perpendicular to beam (NA above height of FSD)

$Atc = (be/n)(\bar{y}t)$

$Itc = (be/n)(\bar{y}t)^3/12$

$It = Itc + Atc(\bar{y}t/2)^2 + Is + As(\bar{y}-d/2)^2 + Icp + Acp(\bar{y}+tcp/2)^2$



Asr = Area of reinforcing steel

Atcp = Area of top cover plate

As = Area of steel beam

Abcp = Area of bottom cover plate

Assumptions: Concrete does not take tension

Reinforcing steel is adequately anchored to develope tensile stress

$$\bar{y} = \frac{\Sigma A\bar{Y}}{\Sigma A} = \frac{Abcp\ (-tbcp)/2 + As(d/2) + Atcp\ (d+ttcp/2) + Asr(d+t/2)}{Abcp + As + Atcp + Asr}$$

Icpt = Moment of inertia of top cover plate = $\frac{Wcpt\ (tcpt)^3}{12}$

Icpb = Moment of inertia of bottom cover plate = $\frac{Wcpb\ (tcpb)^3}{12}$

Itnm = Moment of inertia of transformed section in the negative moment region

Itnm = $I + Ad^2$

Itnm = $Asr\ (d+t/2-y)^2 + Icpt + Acpt(d+tcpt/2-y)^2 + Is + As(y-d/2)^2 + Icpb + Acpb(y+tcpb/2)^2$

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