COMPOSITE BEAMS

General

A composite beam, in general, consists of a steel beam and concrete slab so inter-connected that both the steel beam and the slab act jointly to resist bending. In practice there are several combinations which effectively act as composite beams: a steel beam or girder with a concrete slab inter-connected with mechanical shear connectors, a steel beam or girder fully encased by the concrete in such a way that the encased beam and the concrete slab behave monolithically, or a steel beam or girder with a steel deck and concrete cover slab inter-connected by mechanical shear connectors. Clause 17 of CAN3-S16.1-M84, "Steel Structures for Buildings — Limit States Design", contains requirements for composite beams.

Some of the advantages offered by composite construction are:

1. Reduction in weight of steel members required.
2. Reduction in depth of steel members required.
3. Reduction in deflections.
4. Ease of modification of electrical utilities when steel deck is used.

The most common type of composite construction utilizes some form of mechanical shear connector welded to the top flange of the beam. In any economic analysis, the weight savings and other advantages of composite construction must be weighed against the added cost of these connectors.

Composite construction is most advantageous when heavy loads and long spans are involved. For this reason composite construction is widely used in bridge construction. With the reduction in the weight of the floor assembly when steel deck is used, composite beams consisting of steel beams with steel deck and concrete cover slab are frequently used in buildings.

A comprehensive examination of the design and behaviour of steel-concrete composite floor systems and their components is contained in the CISC's "Design and Construction of Composite Floor Systems". This book contains numerous design aids and examples for composite beams, girders, trusses, and stub-girders. Design may be further facilitated by the use of CISC/CSCC developed computer programs such as GFD2 for the HP9000 Series 200 or component design program for personal computers. Programmable calculators can also be used very effectively in the design of composite beams. Programs for the HP41C programmable pocket calculators are also available from the CISC. Further information on computer-aided design may be obtained from the CISC.

Tables

The Composite Beam Trial Selection Tables, pages 5—28 to 5—87 are based on the use of CSA G40.21-M 300W steel and list composite beams using W shapes manufactured in Canada from 200 mm to 610 mm nominal depth. Tables are provided for the following:

a) 130 mm solid slab with a 28-day specified concrete strength, f'_c, of 20 MPa.

b) 38, 51 and 76 mm steel deck and 65 mm concrete cover slabs with f'_c of 20 MPa.

c) 51 and 76 mm steel deck and 85 mm concrete cover slab with f'_c of 25 MPa.

The shapes are listed in descending order of nominal depth and mass. In the tables the following properties and resistances are listed:

b = flange width of steel shape, millimetres

t = flange thickness of steel shape, millimetres

b_1 = Effective width of the concrete used in computing values of M_{rc}, Q_r, I_1 and S_{s_1} less than or equal to $b + 16t_s$ where t_s equals the overall slab thickness or overall cover slab plus steel deck thickness. Refer to Clause 17.3.2 of CAN3-S16.1-M84, millimetres.

M_{rc} = Factored moment resistance of the composite beam for percentages of shear connection from 50 to 100 (Clause 17.4.3, CAN3-S16.1-M84), kilonewton metres.

$Q_{r100\%}$ = Required sum of all the factored resistances of the shear connectors between points of maximum and zero moment for 100 per cent shear connection; equal to either $\phi A_s F_y$ or $0.85\ \phi_c\ btf'_c$, (Clause 17.4.3, 17.4.5 and 17.4.6, CAN3-S16.1-M84), kilonewtons

I_1 = Moment of inertia of the composite section computed using a mass density of 2300 kg/m³ for f'_c = 20 MPa and a mass density of 1850 kg/m³ for f'_c = 25 MPa, 10^6 mm⁴.

S_{s_1} = Section modulus of the composite section related to the extreme fibre of the bottom flange of the steel beam based on the value of I_1, 10^3 mm³.

For the bare steel shape:

M_r = Factored moment resistance of the steel beam alone, kilonewton metres.

V_r = Factored shear resistance of the steel beam alone, kilonewtons.

L_u = Maximum unsupported length of compression flange of the steel beam alone for which no reduction in M_r is required, millimetres.

I_x = Moment of inertia about the X—X axis of the steel beam alone, 10^6 mm⁴.

S_x = Section modulus of the steel beam alone, 10^3 mm³.

Since the concrete slab and/or the steel deck inhibit movement of the top flange, lateral buckling is not a consideration under composite action. During construction however certain beams or girders may have some length of unsupported compression flange greater than L_u. The moment resistance of these members can be found in the "Beam Selection Tables" on pages 5—98 to 5—109 for the appropriate unsupported length of compression flange.

The factored shear resistance, V_r, tabulated is computed according to Clause 13.4.1, CAN3-S16.1-M84 for the appropriate h/w ratio.

Shear Connectors

Clauses 17.4.5 and 17.4.6, CAN3-S16.1-M84 stipulate the requirements for total factored horizontal shear force that must be resisted by shear connectors.

For full (i.e. 100%) shear connection the total factored horizontal shear force, V_h, to be transferred between the point of maximum positive moment and the points of zero moment is either

= $\phi A_s F_y$ when the plastic neutral axis is in the slab, or

= $0.85\ \phi_c\ btf'_c$ when the plastic neutral axis is in the steel section

For partial shear connection the total factored horizontal shear force, V_h, is the sum of the factored resistances of all the shear connectors between points of maximum positive and zero moment. Clause 17.4.4, CAN3-S16.1-M84, limits the minimum amount of partial shear connection to 50 percent of $\phi A_s F_y$ or $0.85\ \phi_c\ btf'_c$ whichever is the lesser when computing flexural strength.

Generally, shear connectors may be evenly spaced in regions of positive or negative bending. However, where there is a concentrated load within a region of positive bending, the number of shear connectors and, hence, the shear connector spacing, is determined by Clause 17.4.8, CAN3-S16.1-M84.

The factored shear resistance, q_r, of end-welded studs for solid slabs where the stud height is at least 4 stud diameters and steel deck with concrete cover slabs where the flute average width is at least twice the height of the steel deck and the projection of the stud is at least 2 stud diameters above the top surface of the steel deck are given in the Table 5—3 on page 5—21 and have been computed according to Clause 17.3.6(a)of CAN3-S16.1-M84.

Factored shear resistances of studs for selected cases of steel deck with cover slab are given in the Table 5—2 on page 5—20 (Table 8, CAN3-S16.1-M84).

Deflections

A composite beam is stiffer than a similar non-composite beam, and deflections are reduced when composite construction is used. Due to creep of the concrete slab, the maximum deflection may increase over a period of time, especially if the full load is sustained. Appendix L of CAN3-S16.1-M84 provides guidance in estimating deflections caused by shrinkage of the concrete slab. Deflection during construction, when the steel beam alone supports the loads, should be checked. Cambering or the use of temporary shoring will reduce the final total deflection.

If the steel beam is unshored during construction, Clause 17.6, CAN3-S16.1-M84 limits the sum of the stresses in the tension flange, due to the specified loads applied before the concrete strength reached 0.75 f_c' plus the stresses at the same location due to the remaining specified loads considered to act on the composite section, to 0.90 F_y.

References

Additional information on composite beams can be obtained from the following references:

1. Chien, E.Y.L., and Ritchie, J.K., "Design and Construction of Composite Floor Systems", CISC, 1984.
2. Adams, P. F., Krentz, H. A., and Kulak, G. L., "Limit States Design in Structural Steel — SI Units", CISC, 1979.
3. Picard, A., Beaulieu, D., "Calcul aux états limites des charpentes d'acier", CISC, 1981.
4. Report of ASCE Task Committee on Composite Construction, "Composite Steel-Concrete Construction" ASCE Proc. J. of Struct. Div. Vol. 100, ST5, May 1974.
5. Kennedy, D.J.L. et al., "Limit States Design", Proc. of Canadian Structural Engineering Conference, 1976, Can. Steel Const. Council, Willowdale, 1976.

Table 5-2
Factored Shear Resistances of Studs for Selected Cases of Steel Deck and Cover Slabs

Depth Steel Deck (mm)	Average Rib Width Minimum (mm)	Depth of Cover Slab (mm)	Stud Size d x h (mm x mm)	No. of Studs per Rib	Factored Shear Resistance* q_r (newtons)
38/43	50	65	14 x 75	1	19 200
38/43	50	65	20 x 75	1	40 000
				2	60 800 per pair
38/43	50	90	20 x 100	1	63 200
				2	90 400 per pair

*Factored shear resistances given in Table 5-2 are derived from test and reflect the influence of rib geometry and stiffness on the useful capacity of studs.

Table 5-3
Factored Shear Resistances of End Welded Headed Studs, q_r, kilonewtons
For Solid Slabs and for Cover Slabs with Wide Rib Deck*

Stud Diameter		Mass Density of Concrete w_c	Factored Shear Resistance, q_r, in kilonewtons for various concrete strengths f'_c, MPa		
in.	mm	kg/m^3	20 MPa	25 MPa	30 MPa
	12	2300	29.5	34.8	37.5
		1850	25.0	29.6	33.9
1/2		2300	33.0	39.0	42.2
		1850	28.0	33.1	38.0
5/8		2300	51.6	61.0	65.7
		1850	43.8	51.8	59.4
	16	2300	52.4	61.9	66.7
		1850	44.5	52.6	60.3
3/4		2300	74.3	87.8	94.6
		1850	63.1	74.6	85.5
	20	2300	81.9	96.8	104
		1850	69.5	82.2	94.2
	22	2300	99.0	117	126
		1850	84.1	99.4	114
7/8		2300	101	119	127
		1850	85.9	101	116

*$q_r = 0.5\, \phi_{sc} A_{sc} \sqrt{f'_c E_c} \leqslant F_u \phi_{sc} A_{sc}$, F_u taken as 415 MPa.

where $E_c = w_c^{1.5}\, 0.043 \sqrt{f'_c}$ and $\phi_{sc} = 0.80$

Examples

1. **Given:**

   Select a simple span composite beam to span 12 m carrying a uniformly distributed specified load of 20 kN/m live load and 7 kN/m dead load. Beam spacing is 3 m. Live load deflection is limited to L/360. Use G40.21-M 300W steel, 76 mm steel deck, 65 mm cover slab of 20 MPa concrete.

Solution

Effect of factored loads = $\alpha_D D + \alpha_L L = 1.25 \times 7 + 1.50 \times 20 = 38.8$ kN/m

Therefore, w_f

$$M_f = \frac{w_f L^2}{8} = \frac{38.8 \times 12^2}{8} = 698 \text{ kN}\cdot\text{m}$$

$$R = V_f = \frac{w_f L}{2} = \frac{38.8 \times 12}{2} = 233 \text{ kN}$$

To meet the deflection limit of L/360 use Figure 5—1 and Table 5—4, pages 5—90 to 5—91, to compute the minimum I required. See page 5—88 on Deflections for an explanation of this method.

Total specified live load, $W = 20 \times 12 = 240$ kN

$B_d = 1.0$ (standard case)

$I_{req'd} = W \times C_d \times B_d = 240 \times 3.4 \times 10^6 \times 1.0 = 816 \times 10^6 \text{ mm}^4$

Effective Width (Clause 17.3.2.1)

a) $0.25\ L = 0.25 \times 12 = 3$ m = 3 000 mm

b) beam spacing = 3 m = 3 000 mm

c) b + 16t — assume b = 200 mm

   Therefore, $b + 16t = 200 + 16 \times (76 + 65) = 2\,460 \text{ mm} < 3\,000$ mm

Therefore b + 16t rule governs.

Beam Selection

From composite beam selection tables for 76 mm steel deck with 65 mm cover slab, page 5—60, the first suitable shape is a W460 × 82 with M_{rc} for 50 per cent shear connection = 717 kN·m; however, for the same mass the W530 × 82 can be selected.

Select W530 × 82 with 50 percent shear connection (b_1 = 2 470 mm)

M_{rc} (50%) = 808 kN·m > 698 kN·m — OK

V_r = 894 kN > 233 kN — OK

I_1 = $1\,390 \times 10^6 \text{ mm}^4 > 816 \times 10^6 \text{ mm}^4$ — OK

Q_r (100%) = 1 640; $S_{s1} = 2\,800 \times 10^3 \text{ mm}^3$; M_r = 559 kN·m; L_u = 2 860 mm

Check Steel Beam Under Dead Load

As required by Clause 17.7, the steel section alone shall be capable of supporting all factored loads applied before concrete hardens. Assume this to be the total design dead load ($\alpha_D D$).

$\alpha_D D \ = \ 1.25 \times 7 = 8.75$ kN/m

$$M_{fD} = \frac{8.75 \times 12^2}{8} = 157 \text{ kN·m} < M_r = 559 \text{ kN·m}$$

In this case the steel deck when fastened to the steel beam will provide sufficient lateral support to the compression flange of the beam during construction. Thus $M_r = 559$ kN·m may be used.

Check Unshored Beam Tension Flange (Clause 17.6)

Assume specified dead load (7 kN/m) is applied before concrete strength reaches $0.75f'_c$.

S of steel beam = $1\,810 \times 10^3$ mm^3

Stress in tension flange due to specified load acting on steel beam alone,

$$f_1 = \frac{M_b}{S} = \frac{7 \times 12\,000^2}{8 \times 1\,810 \times 10^3} = 69.6 \text{ MPa}$$

Stress in tension flange due to specified load acting on composite section,

$$f_2 = \frac{M_t}{S_{s_1}} = \frac{20 \times 12\,000^2}{8 \times 2\,800 \times 10^3} = 129 \text{ MPa}$$

$0.9\ F_y = 0.9 \times 300 = 270$ MPa

$f_1 + f_2 = 69.6 + 129 = 199$ MPa < 270 MPa — OK

Shear Connectors

Q_r (100%) = 1 640 kN

The 76 mm steel deck to be used will have a flute width of at least 152 mm (2 × 76). Therefore Table 5—3, page 5—21, may be used to determine, q_r, factored shear resistance of the studs.

Assume (a) mass density = 2 300 kg/m^3

(b) 20 mm stud will be used.

Check beam flange thickness = 20/2.5 = 8 mm < 13.3 mm — OK
(Clause 17.3.5.5; CAN3-S16.1-M84)

$q_r = 81.9$ kN

No. of studs required = 2 × Q (100%) × Percent connection/100/q_r
= 2 × 1 640 × 50/100/81.9
= 20 use 20 studs.

Since there are no concentrated loads, the studs can be spaced uniformly along the full length of the beam as permitted by the deck flutes.

Example 2

2. Given:

Design a simple span composite truss to span 12 m carrying a 3 m wide office floor with a live load of 4.8 kPa and a partition load of 1.0 kPa. Floor slab consists of 76 mm deep wide rib profile deck with 65 mm of 2 300 kg/m^3 and 20 MPa concrete cover slab. Truss overall depth = span /16 or 750 mm. Live load deflection is to be limited to span /360, and compute approximate shop camber.

Solution:

Compute Factored Loads

Dead load (concrete + deck + steel) = 2.60 kPa
Dead load (partitions) = 1.00 kPa

Total specified dead load = 3.60 kPa

Tributary floor area = $12 \times 3 = 36\ m^2$

Live load reduction factor (office) = $0.3 + \sqrt{9.8/A} = 0.822$

Reduced live load = $0.822 \times 4.8 = 3.95$ kPa

Total factored load = $1.25 \times D + 1.5 \times L$
$= 1.25 \times 3.6 + 1.5 \times 3.95 = 10.4$ kPa

Factored mid span moment = $\dfrac{10.4 \times 3 \times 12^2}{8} = 562$ kN·m

Compute Trial Bottom Chord Size

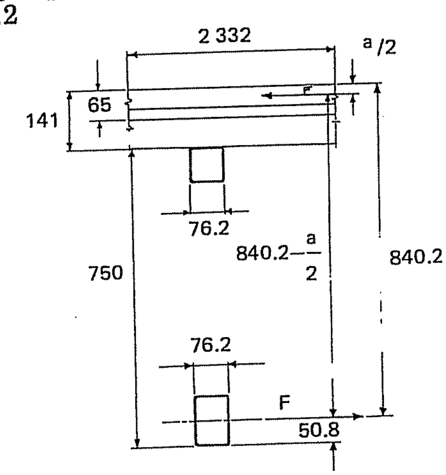
Step 1

Determine effective slab width:
least value of — span/4 = 3 000 mm
— flange width + 16 thickness (slab & deck) = 2 332 mm
— centres of truss = 3 000 mm

Therefore effective slab width = 2 332 mm (Clause 17.3.2.1 — CAN3-S16.1-M84)

Step 2

Determine neutral axis position, a, neglecting top chord — (See Clause 17.4.2 of CAN3-S16.1-M84) — Assume HSS 101.6 × 76.2



— Equating external and internal moments (Clause 17.4.3(a) — CAN3-S16.1-M84) $(804.2 - a/2) \times F = 562 \times 1\,000$ .... (1)
— Compute slab force F (kN)
$F = 0.60 \times 0.85 \times a \times 2\,332 \times 20/1\,000$
$F = 23.8a$ .... (2)
— Equating (1) and (2) and solving for 'a'
$(840.2 - a/2)\ 26.56a = 562\,000$
$a^2 - 1\,680.4a + 42\,319 = 0$

Therefore a = 28.6 mm

Step 3

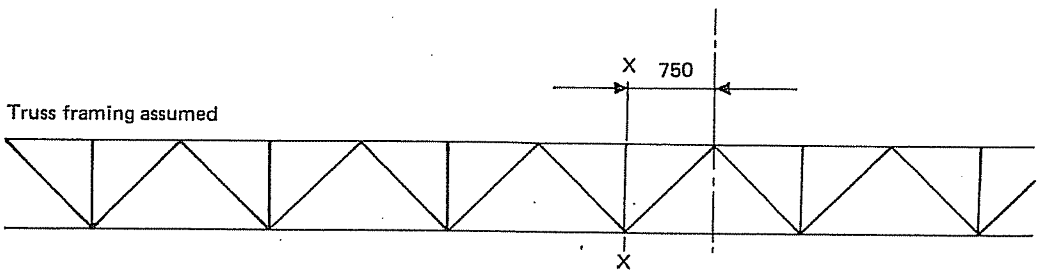
Find bottom chord force, F = 23.8a = 680 kN

A req'd (a) for W shape, $F_y = 300$ MPa, $A = \dfrac{680 \times 10^3}{0.9 \times 300} = 2\,519\ mm^2$

(b) for HSS, $F_y = 350$ MPa, $A = \dfrac{680 \times 10^3}{0.9 \times 350} = 2\,159\ mm^2$

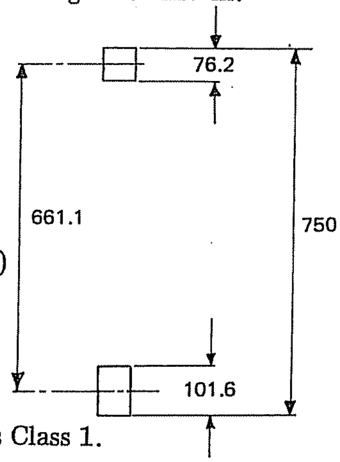
Try HSS 101.6 × 76.2 × 7.95, $A = 2\,410\ mm^2$

Compute Trial Top Chord Size



Assume factored bending moment on steel truss under slab pouring @ XX = 260 kN·m; and assume factored local bending at top chord under slab pouring = 1.0 kN·m.

Therefore factored axial force $= \dfrac{260 \times 1\,000}{661.1} = 393$ kN



Try HSS 76.2 × 76.2 × 6.35

$\dfrac{KL}{r} = \dfrac{0.9 \times 750}{28} = 24$; $A_s = 1\,670$ mm²

C_r/A (for Class C HSS $F_y = 350$) = 299 MPa (See Table 4—3)

$C_r = 499$ kN; t = 6.35

$\dfrac{b}{t} = \dfrac{h}{w} \simeq \dfrac{76.2 - 2 \times (2t)}{t} = 8.0$

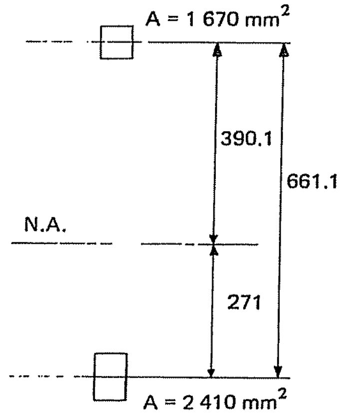
Since b/t < 22.4 and h/w < 35.8, HSS 76.2 × 76.2 × 6.35 is Class 1.

$M_r = 13.5$ kN·m (see page 4—61)

Therefore interaction value $= \dfrac{C_f}{C_r} + \dfrac{M_f}{M_r} = \dfrac{393}{499} + \dfrac{1}{13.5} = 0.86 < 1$

Note: top chord thickness is also governed by the requirement of shear stud welding. (Clause 17.3.5.5, CAN3-S16.1-M84.)

Estimate effective moment of inertia of steel truss for deflection calculation



Neutral axis of chords from C.G. of bottom chord
$1\,670 \times 661.1/(1\,670 + 2\,410) = 271$ mm

Moment of inertia of chords $(I_t + I_b) =$
$(1.31 + 3.10) \times 10^6$ mm⁴

Therefore moment of inertia of truss

$\simeq 0.85 \times (2\,410 \times 271^2 + 1\,670 \times 390^2) + (I_t + I_b)$

(Note: Estimate 15% reduction to account for deflections contributed by web strain)

$\simeq (366 + 4.4) \times 10^6$ mm⁴

$\simeq 370 \times 10^6$ mm⁴

ompute camber (2.6 kPa)

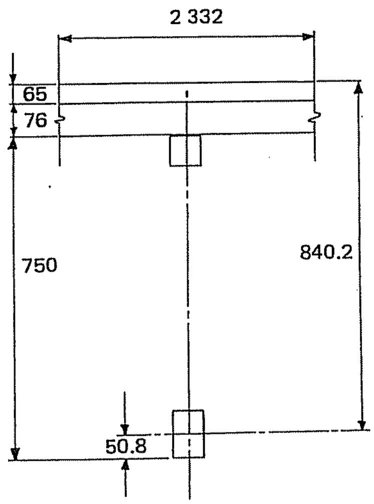
Camber ≃ Δ under slab pour

$$= \frac{5}{384}\frac{WL^3}{EI} = \frac{5}{384} \times \frac{93.6 \times 12\,000^3 \times 10^3}{200\,000 \times 370 \times 10^6}$$

$= 28.5$ mm

Therefore assume shop camber of 28 mm

Compute composite truss moment of inertia



$E_{concrete} \simeq w_c^{1.5}\ 0.043\sqrt{f_c'}$

$\simeq 2\,300^{1.5} \times 0.043 \times \sqrt{20}$

$\simeq 21\,200$ MPa

$E_{steel} = 200\,000$ MPa

$n = E_{steel}/E_{concrete} = 9.43$

Transformed slab width $= \frac{2\,332}{9.43} = 247$ mm

Material	Area (a) (mm²)	distance (d) to top (mm)	ad	ad²
Concrete	16 055	32.5	521 788	17 x 10⁶
Steel	2 410	840.2	2 024 882	1 701 x 10⁶
Total	18 465		2 546 670	1 718 x 10⁶

Neutral axis $= \frac{2\,546\,670}{18\,465} = 138$ mm

I about NA $= 1\,718 \times 10^6 - 18\,465 \times 138^2$

$= 1\,370 \times 10^6\ mm^4$

Assume I lost for web deflection due to strain

$\simeq$ 15% of I of truss chords ($431 \times 10^6\ mm^4$)

$= 65 \times 10^6\ mm^4$

Therefore moment of inertia of composite truss

$\simeq (1\,370 - 65) \times 10^6 \simeq 1\,300 \times 10^6\ mm^4$

Compute LL Deflection (Live load = 3.95 X 3 X 12 = 142 kN)

$$\Delta_{LL} = \frac{5}{384} \times \frac{WL^3}{EI} \times (1.15 \text{ for the deck} + 0.15 \text{ for creep}) \text{ (Clause 17.3.1.1)}$$

$$= \frac{5}{384} \times \frac{142 \times 12\,000^3 \times 10^3}{200\,000 \times 1\,300 \times 10^6} \times 1.30$$

$= 16$ mm < 33 mm (L/360)

Compute number of shear studs using 16mm diameter studs

Factored Q force to be developed by shear connectors = F = 680 kN

Using 16 mm studs in wide rib profile steel deck (rib width/deck height > 2) with concrete of density 2 300 kg/m³ and strength 20 MPa, factored shear resistance per stud = 52.4 kN (see table on page 5—21).

Therefore number of shear studs per half span $= \frac{680}{52.4} = 13$

Therefore use 26 — 16 mm studs per truss.

Web Members

Web members consisting of angles welded to outside face of truss chords, HSS or other shapes, can be sized by traditional means.

Preliminary Truss

Therefore use an HSS 76.2 × 76.2 × 6.35 for top chord and an HSS 101.6 × 76.2 × 7.95 for the bottom chord for the preliminary truss design.