4 Design of Tension Members

4.1 TYPES OF TENSION MEMBERS

Tension members do not buckle. Therefore, steel can be used most efficiently as tension members. Different types of sections used as tension members are shown in Fig. 4.1. Steel cables are constructed of a number of wire ropes or strands. The wires are cold-drawn high-strength steel alloys. This combination of cold work and special steel alloys produces very high yield strength in the range of 200 to 250 ksi. Thus, cables are particularly suitable for covering large spans and are used in long-span suspension bridges, cable roofs, cable-stayed bridges, and ski lift cables. Cables, of course, are flexible. To provide stiffness, cable structures may be stiffened by adding stiffening members.

When the magnitude of tensile force is small in a tension member, solid round or rectangular bars are used. For larger tensile forces or when more stiffness is required, round or rectangular tubes may be used. Round tubes or pipes might be preferred when the tension member is exposed to high-wind condition. Connection details for round tube, however, are cumbersome to construct.

Single angles are commonly used as tension members, for example, as bracing for carrying lateral forces due to wind or earthquake. Angle end connection is simple but eccentric to its centroidal axis. The eccentric application of tensile force produces bending stresses in members which are often ignored in design practice. The eccentricity may be minimized by suitable detailing. For example, when only one leg of the angle is connected to the joint, it should be the larger one.

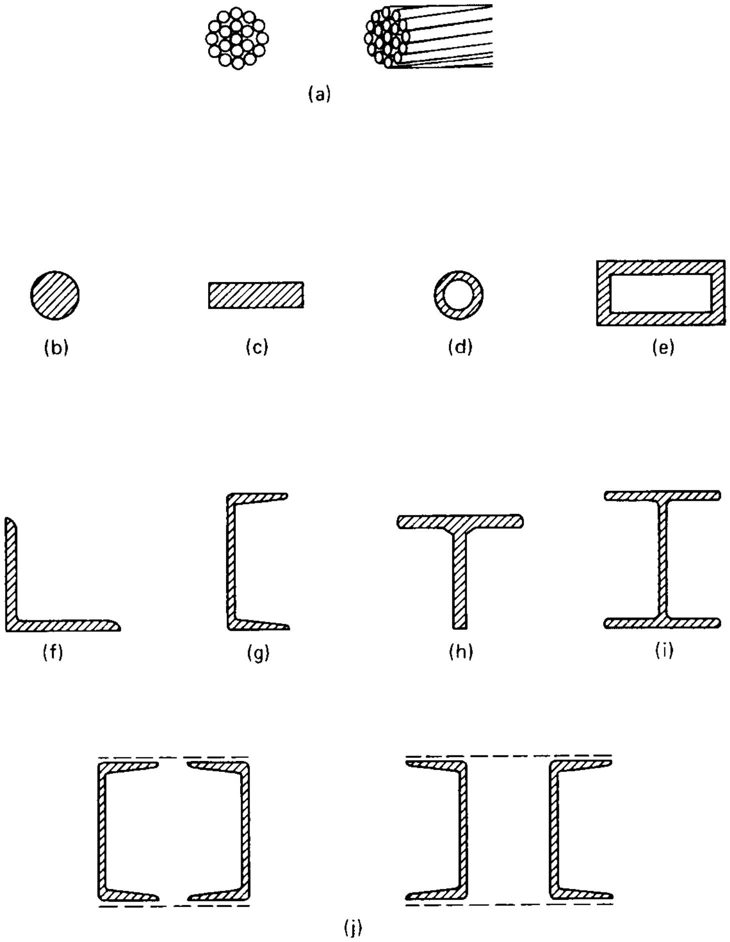


Figure 4.1 Types of tension members: (a) Cable with strands, (b) Round bar, (c) Rectangular bar, (d) Round tube, (e) Rectangular tube, (f) Angle (L), (g) Channel (C), (h) T section, (i) W section, (j) Channels with lacings

Compared with an angle, a channel connected to the joint at its web often produces less eccentricity, since the centroid of most channels is close to their web. For carrying a large tensile force, W sections are used. For a very large tensile force, built-up sections (for example, channels with lacing bars) may be used. Dashed lines in Fig. 4.1(j) indicate the lacing bars. Inclined single or double lacing bars connect the two elements of the built-up section.

4.2 NET AND EFFECTIVE NET AREA

When tension members are connected by welding, the total cross-sectional area is available for transferring the tension. When the connection is done by bolting (or riveting), holes must be made in the member. These holes evidently reduce the cross-sectional area available for transferring the tension. Thus, the net area of the section is the gross area minus deductions for the holes [Fig. 4.2(a)].

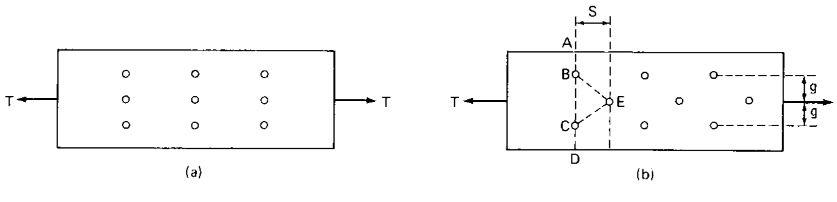


Figure 4.2 Holes in a tension member

Holes are sometimes staggered, as shown in Fig. 4.2(b). Staggering of the holes increases the net area of the section. In Fig. 4.2(b), the plate may fail along section ABCD or section ABECD. How do we calculate the net area or the net width along a zigzag line of holes such as ABECD? According to the approximate procedure provided by ASD B2 (or LRFD B2), the net width (w_n) is obtained by deducting the sum of the diameters of all the holes located on the zigzag line from the gross width (w_g) and then adding for each inclined line such as BE the quantity $s^2/4g$.

$$w_n = w_g - \sum D + \sum \frac{s^2}{4g} \tag{4.1}$$

where

s = the center-to-center spacing of the two consecutive holes in the direction of stress (pitch)

g = the transverse center-to-center spacing of the same two holes (gage)

ΣD = sum of the diameters of the holes in the line of holes.

The critical net area A_n of the tension member is found from the line of holes with the minimum net width.

According to ASD B2 (or LRFD B2), in calculating the net area the width of a hole should be assumed 1/16 in. greater than the nominal size of the hole.

When a tension member consists of more than one plate segment, the axial tension may not be transmitted through all the segments. For example, in the case of an angle, only one leg may be connected to the joint. In this case, when connection is through bolts or rivets, an effective net area should be calculated by using the following formula (ASD B3 or LRFD B3):

$$A_e = UA_n \tag{4.2}$$

TABLE 4.1. EFFECTIVE NET AREA COEFFICIENT FOR FASTENERS *U* (ASD B3)

Types of members	Minimum number of fasteners per line in the direction of tension	U
(a) All segments are connected to transmit the tension	1	1
(b) W, M, or S sections $\frac{b_f}{d} \geq \frac{2}{3}$ Connection to flange(s)	3	0.90
(c) Tees $\frac{b_f}{d} \geq \frac{4}{3}$ Connection to flange	3	0.90
(d) W, M or S sections not meeting the conditions of (b), Tees not meeting the conditions of (c), and all other shapes, including built-up sections	3	0.85
(e) All sections	2	0.75

where *U* is a reduction factor. The values of U for different types of sections according to ASD are given in Table 4.1. This reduction is necessary to take into account the non-uniformly distributed transfer of stresses.

4.3 DESIGN OF TENSION MEMBERS ACCORDING TO ASD

1. Allowable tensile stress at the holes for pin-connected members and eye-bars (ASD D3.1):

$$F_t = 0.45\ F_y \qquad \text{on the actual net area, } A_n \tag{4.3}$$

2. For threaded rods (ASD Table J3.2):

$$F_t = 0.33\ F_u \qquad \text{on the gross area, } A_g \tag{4.4}$$

where F_u is the specified minimum ultimate tensile stress.

3. For other members, the allowable tensile stress is the smaller value obtained from the following two formulas (ASD D1):

$$F_t = 0.60\ F_y \qquad \text{on the gross area, } A_g \tag{4.5}$$

$$F_t = 0.50\ F_u \qquad \text{on the effective net area, } A_e \tag{4.6}$$

Equation (4.5) is based on the yield criterion, while Eq. (4.6) is based on the fracture criterion.

Since tension members do not buckle, there is no slenderness ratio limitation. In order to prevent the undesirable lateral movement and vibration, however, it is recommended that the slenderness ratio L/r of tension members, except rods, should preferably not exceed 300 (ASD B7).

4.4 EXAMPLES OF DESIGN OF TENSION MEMBERS BASED ON ASD

Example 1

Find the maximum allowable tensile load T for a channel section connected to a gusset plate as shown in Fig. 4.3, assuming that the gusset plate and the 1-in.-diameter bolts do not control the design. The yield stress and the ultimate strength of the steel are 50 ksi and 65 ksi, respectively.

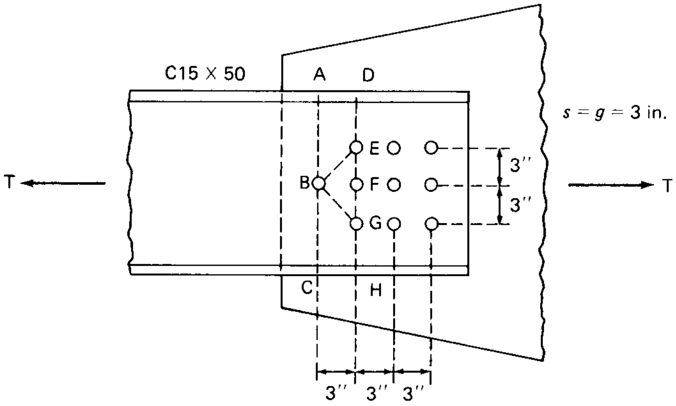


Figure 4.3

Solution Properties of C15 X 50

A_g = 14.7 in.^{2} t_w = 0.716 in. (web thickness)

For standard-size holes, the size of hole should be 1/16 in. greater than the diameter of the fastener (ASD J3.2).

$$\text{Size of hole} = 1 + \tfrac{1}{16} = \tfrac{17}{16} \text{ in.}$$

$$\text{Deduction for one hole} = \tfrac{17}{16} + \tfrac{1}{16} = \tfrac{9}{8} \text{ in.}$$

Reduction factor: $U = 0.85$ (Table 4.1)

At section ABC: Tension = T

$$A_n = A_g - t_w(\tfrac{9}{8}) = 14.7 - 0.716(\tfrac{9}{8}) = 13.89 \text{ in.}^2$$

At section DEBGH: Tension = T

$$A_n = A_g - 3t_w(\tfrac{9}{8}) + 2\left(\frac{s^2}{4g}\right)t_w$$

$$= 14.7 - 3(0.716)(\tfrac{9}{8}) + 2(\tfrac{9}{12})(0.716) = 13.36 \text{ in.}^2$$

At section DEFGH: Tension = 0.9T

$$A_n = A_g - 3t_w(\tfrac{9}{8}) = 14.7 - 3(0.716)(\tfrac{9}{8}) = 12.28\,\text{in.}^2$$

$$\frac{A_n}{0.9} = \frac{12.28}{0.90} = 13.65\,\text{in.}^2$$

Therefore, the zigzag section DEBGH is the critical section and the governing net area is A_n = 13.36 in.^{2}.

Maximum allowable tensile load based on the yield criterion:

$T = 0.60F_y A_g = 0.60(50)(14.7) = 441$ K

Maximum allowable tensile load based on the fracture criterion:

$T = 0.50F_u A_e = 0.50(65)(0.85)(13.36) = 369.1$ K

Therefore, the maximum allowable tensile load is

$T = 369.1$ K

Example 2

Consider the *preliminary* design of a 12-story office building shown in Fig. 4.4. The building has a circular plan with a diameter of 50 ft and a constant story height of 10 ft. Floors 1 to 10 are supported at the center by a hollow circular reinforced concrete shear wall with a diameter of 16 ft and at the perimeter by 12 identical hangers suspended from 12 story-high truss systems located at the twelfth story.

Design the hangers for a dead load of 60 psf and a live load of 80 psf, choosing the lightest WT section and assuming uniform section throughout the height of each hanger. Use A36 steel with Fy = 36 ksi.

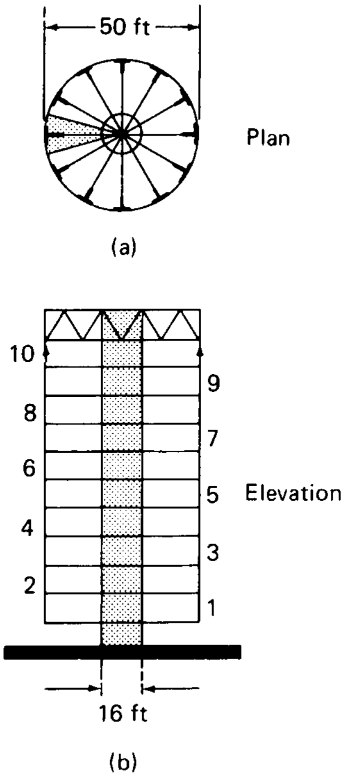


Figure 4.4 (a) Plan, (b) Elevation

Solution In order to take into account the effect of impact for hangers supporting floors, the assumed live load should be increased by 33 percent (ASD A4.2).

Design load intensity:

$$w = 60 + 1.33(80) = 166.4 \text{ psf}$$

The load-carrying tributary area for each beam is shaded in the plan of Fig 4.4.

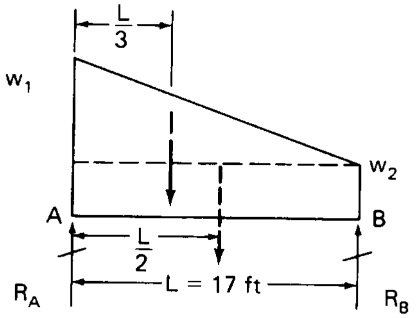


Figure 4.5 Load distribution on each radial beam

The load distribution on each radial beam is approximately trapezoidal, as shown in Fig. 4.5, with inner intensity w_2 and outer intensity w_1.

$$w_1 = \frac{\pi(50)}{12}(166.4) = 2178\,\text{lb/ft} = 2.178\,\text{K/ft}$$

$$w_2 = \frac{\pi(16)}{12}(166.4) = 697\,\text{lb/ft} = 0.697\,\text{K/ft}$$

The reaction at the outer end of any radial beam is (the supports condition of the radial beams is assumed simple):

$$R_A = \frac{w_2 L}{2} + \frac{2}{3}\left(\frac{1}{2}\right)(w_1 - w_2)L = \frac{(2w_1 + w_2)L}{6}$$

$$= \frac{(2\times 2.178 + 0.697)(17)}{6} = 14.317\,\text{K}$$

Maximum tension in a hanger (neglecting its weight):

$T_{max} = 10R_A = 143.17$ K

Allowable tensile stress:

$F_t = 0.60Fy = 22$ ksi

Required area of the cross section:

$$\text{Area} = \frac{T_{\max}}{F_t} = \frac{143.17}{22} = 6.51\,\text{in.}^2$$

Try WT8 X 22.5

A = 6.63 in.^{2} r_x = 2.39 in. R_y = 1.57 in.

Check for the maximum tensile stress, including the weight of the hanger.

Weight of the hanger = 10(10)(0.0225) = 2.25 K

T_{max} = 143.17 + 2.25 = 145.42 K

Maximum tensile stress:

$$f_t = \frac{T_{max}}{A} = \frac{145.42}{6.63} = 21.93\,\text{ksi} < 22\,\text{ksi}$$ **<u>O.K.</u>**

Check the slenderness ratio (not mandatory), ASD B7.

$$\frac{L}{r_y} = \frac{(10)(12)}{1.57} = 76.4 < 300$$ **<u>O.K.</u>**

4.5 DESIGN OF CIRCULAR SUSPENSION CABLE ROOFS ACCORDING TO ASD

Figure 4.6 shows an example of this type of structure. The roof carries its own weight and a small live load to allow for repairs. This type of structure is not economical for regions with heavy snowfalls, since the snow would accumulate in the dished roof surface and impose heavy loads (Cowan and Wilson, 1981). The cables are supported by an outer reinforced concrete (R.C.) compression ring and a small steel tension ring at the center.

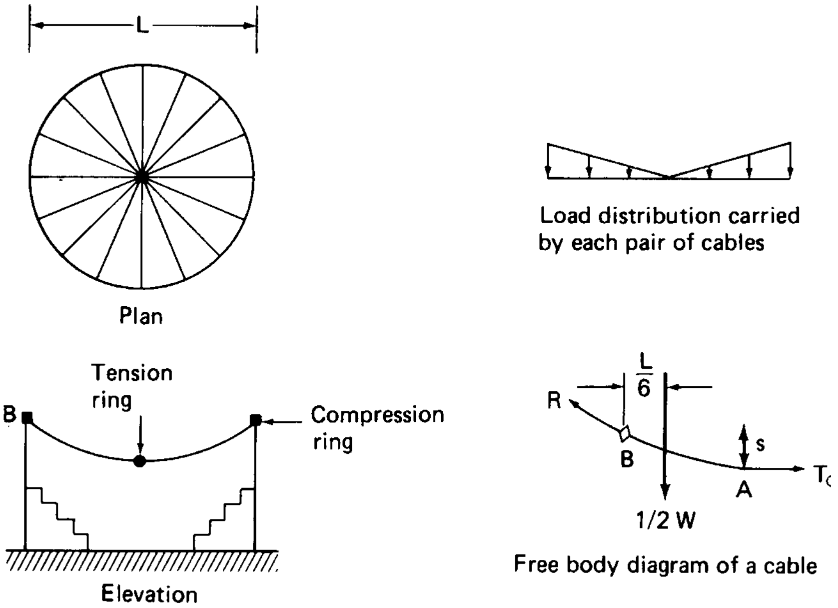


Figure 4.6

4.5.1 Design of Cables

Each cable carries a triangular slice of load. The maximum tension in the cable is equal to the resultant reaction at the supports. This reaction can be calculated by considering the equilibrium of one cable, such as cable *AB* in Fig. 4.6. Considering the moment equilibrium of *AB* about the point *B* (compression ring), we find the value of tension T_o at the lowest point *A* (tension ring).

$$T_o s = \tfrac{1}{2} W \left(\tfrac{1}{6} L\right)$$
$$T_o = \frac{WL}{12s} \tag{4.7}$$

where *L* is the span length, *s* is the sag of cable, and *W* is the total load carried by a pair of cables. Quantity T_o is the minimum tension on the cable. The horizontal component of tension at any point on the cable is equal to T_o. Noting that the vertical component of reaction at *B* is *W*/2, the magnitude of reaction at *B*, equal to the maximum tension in the cable, becomes

$$T_{\max} = R = \sqrt{T_o^2 + \left(\frac{W}{2}\right)^2} = \sqrt{\left(\frac{WL}{12s}\right)^2 + \left(\frac{W}{2}\right)^2} \tag{4.8}$$

It should be noted that in this approximate analysis the weight of cables has been neglected. Thus, the required cross-sectional area of cable is

$$A_w = T_{max}/F_t \tag{4.9}$$

where F_t is the allowable tensile stress in steel cables.

Cables used in suspension roofs are made by twisting thin wires, and their cross-sectional area is approximately equal to $2/3(3.14/4)d^2$ (Cowan and Wilson, 1981), where *d* is the diameter of the cable. The load *W* carried by a pair of cables depends on the spacing of cables around the circumference. Thus, for a given sag, the area of steel required depends in the spacing of cables around the circumference.

4.5.2 Design of R.C. Ring

If it is assumed that the compression ring is supported by vertical walls or columns, the vertical reaction will be carried by the vertical elements, and the compression ring needs to be designed for the horizontal reaction of T_o over a distance of *s*', the spacing of cables around the circumference. In other words, the compression ring is acted on by inward forces as in a pipe under suction. These inward forces produce roughly a uniform axial compressive force in the circular ring. We can find the magnitude of this compressive force *C* by considering the equilibrium of one-half of the compression ring as shown if Fig. 4.7.

$$2C = \frac{T_o}{s'}(L) \qquad (4.10)$$

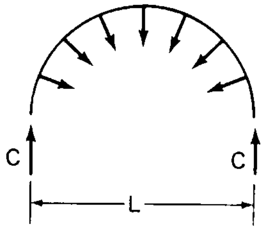


Figure 4.7

Substituting *T*o for from Eq. (4.7) in Eq. (4.10), we obtain

$$C = \frac{WL^2}{24ss'} \qquad (4.11)$$

The R.C. ring beam is designed as per American Concrete Institute (ACI) specification (ACI, 1996). In the web-based interactive program the user has the following options for designing the reinforced concrete ring:

1. The cross-sectional area is found-by neglecting the area of reinforcement and providing a nominal reinforcement of about 1 percent of the concrete cross-sectional area. Thus in this case, the area of compression ring is given by

   $$A = C / 0.476 f_c^{'} \tag{4.12}$$

   where $f_c^{'}$ is the compressive strength of concrete.

2. In this alternative, the program computes the cross-sectional area of the compression ring and the amount of reinforcement treating the ring as a reinforced concrete section. For a given reinforcement, the area of concrete can be calculated by using the design strength equation for a tied column (ACI section 10.3.5.2) (ACI, 1996):

   $$\phi P_{n(\max)} = \phi\{0.80[0.85 f_c^{'} (A_g - A_{st}) + f_y A_{st}]\} \tag{4.13}$$

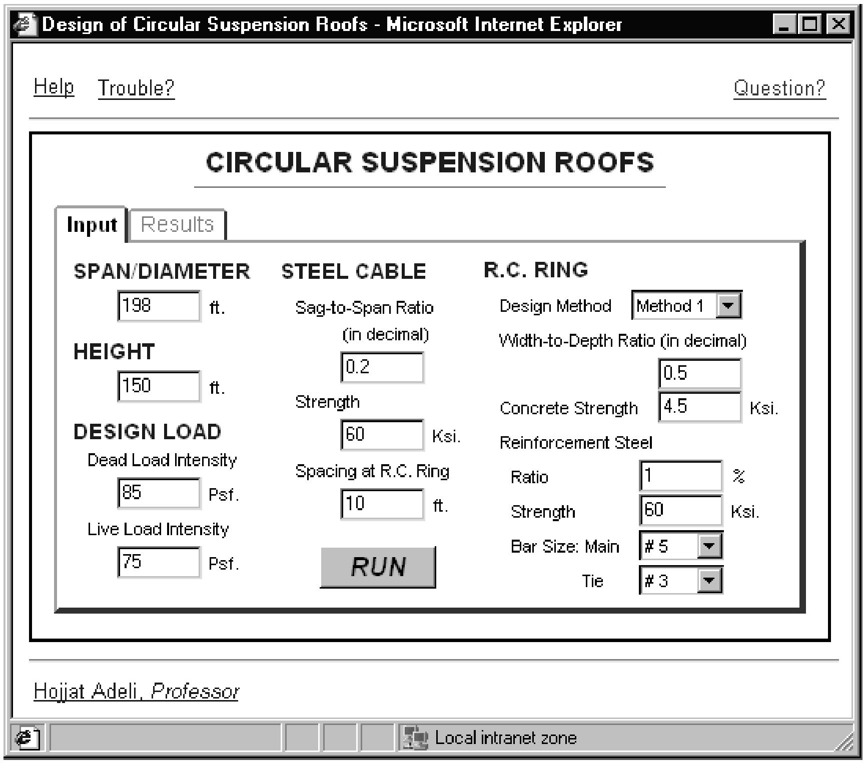
   where

   ϕ = strength reduction factor = 0.7
   $P_{n(max)}$ = maximum nominal load
   A_g = gross cross-sectional area of concrete
   A_{st} = total area of longitudinal reinforcement
   f'_c = compressive strength of concrete
   f_y = yield stress of steel reinforcement

The program checks for minimum and maximum amount of reinforcement on the ring as per ACI specification Sec. 10.9.1. The lateral ties are designed as per ACI specification Sec. 7.10.5 (ACI, 1996).

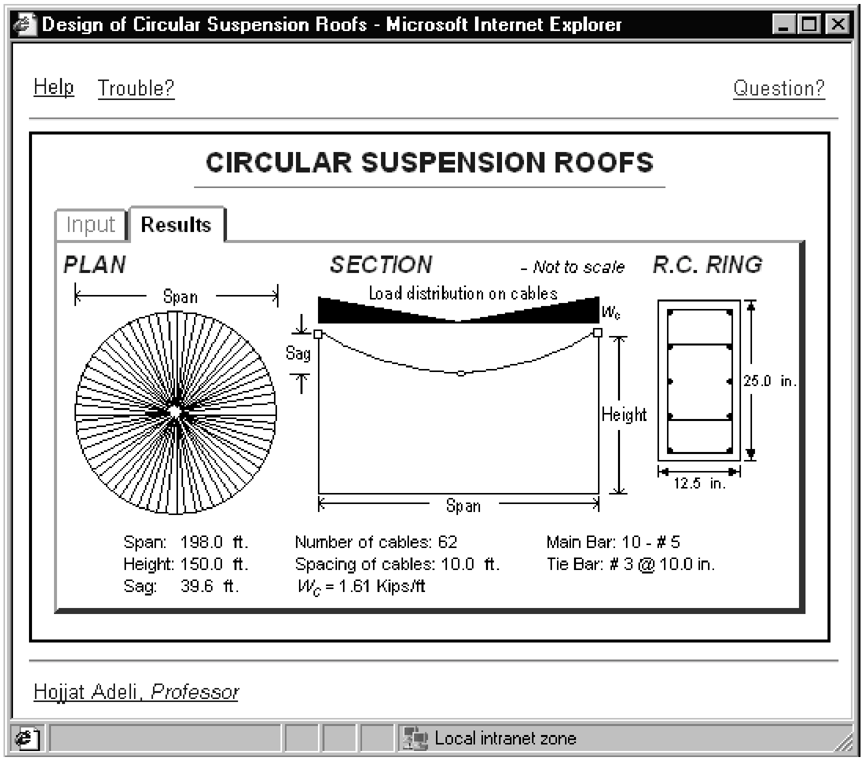
4.6 WEB-BASED INTERACTIVE DESIGN OF CIRCULAR SUSPENSION ROOFS

The applet for interactive design of circular suspension roofs according to ASD consists of *Input* and *Results panels* shown in Figures



SPAN/DIAMETER 198 ft.
HEIGHT 150 ft.
Dead Load Intensity 85 Psf.
Live Load Intensity 75 Psf.
Sag-to-Span Ratio (in decimal) 0.2
Strength 60 Ksi.
Spacing at R.C. Ring 10 ft.
Design Method Method 1
Width-to-Depth Ratio (in decimal) 0.5
Concrete Strength 4.5 Ksi.
Reinforcement Steel
Ratio 1 %
Strength 60 Ksi.
Bar Size: Main # 5
Tie # 3

Figure 4.8 Input panel for the applet for design of circular suspension roofs



Span: 198.0 ft.
Height: 150.0 ft.
Sag: 39.6 ft.

Number of cables: 62
Spacing of cables: 10.0 ft.
W_c = 1.61 Kips/ft

Main Bar: 10 - # 5
Tie Bar: # 3 @ 10.0 in.

Figure 4.9 Results panel for the applet for design of circular suspension roofs

4.8 and 4.9, respectively. The user can toggle back and forth to these panels by clicking on the corresponding tab attached to them.

The user can enter the input for design of suspension roofs in the input panel (Figure 4.8). This input includes basic input data, input for cables, and input for R.C. ring.

1. **Basic Input Data.** The basic input data for a design problem are those usually fixed for a particular design. In the design of a cable roof, the user has to input the following basic data:
   a. Span/diameter of the roof (L)
   b. Height of the structure (H), and
   c. Vertical dead load intensity (W_{DL}) and vertical live load intensity (W_{LL}).

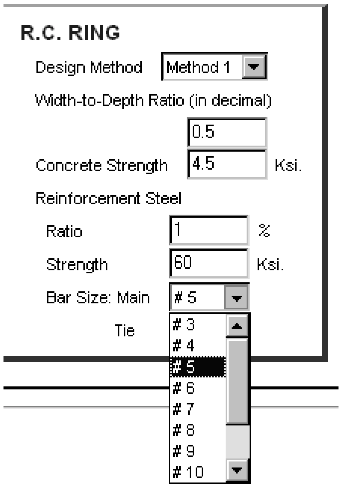
2. **Cable Input.** For the cable, the user is asked to input
   a. Spacing of cables around the circumference (at the compression R.C. ring)
   b. The sag-to-span ratio of the roof, and
   c. Allowable tensile stress of steel cable (F_t).

3. **R.C. Ring Input.** For R.C. ring design, the user has to input
   a. Width-to-depth ratio of R.C. ring cross section
   b. Compressive strength of concrete (f'_c)
   c. Yield stress of compression ring reinforcement (f_y)
   d. Size (bar number) of the reinforcements bars
   e. Reinforcement ratio as a percentage of the gross cross-sectional area of the R.C. ring.

The user can select the size of main and tie bars by scrolling down the choice list (Figure 4.10). Further, the user can also choose between two options: design method 1 and design method 2 for the R.C. ring design. Method 1 designs the concrete section by neglecting contribution of reinforcement and providing a nominal reinforcement (Eq. 4.12). Method 2 designs the ring as a reinforced concrete section as per ACI specification (Eq. 4.13). The applet shows the description of design methods by a pop-up message (Figure 4.11).

The applet accepts legal and practical values only. For improper or impractical input values a warning or pop-up message will be displayed and the user will be asked to re-enter/re-select the input. Other bits of information and knowledge are also displayed to guide the user in the design process. As an example, when the user enters the reinforcement ratio less than 1 % the pop-up message shown in Figure 4.12 will be displayed. The following inputs given by the user are considered to be unrealistic:

a. Sag of the cable computed, using the sag-to-span ratio given by the user, is greater than 0.8 times the height of the structure.
b. Spacing of cables at the circumference is greater than 25 ft.
c. Bar size (number) of reinforcement is not a practical one.
d. Allowable/yield stress in steel is abnormally high or low with respect to its practical value.
e. Compressive strength of concrete is abnormally high or low with


R.C. RING

Design Method: Method 1

Width-to-Depth Ratio (in decimal): 0.5

Concrete Strength: 4.5 Ksi.

Reinforcement Steel

Ratio: 1 %

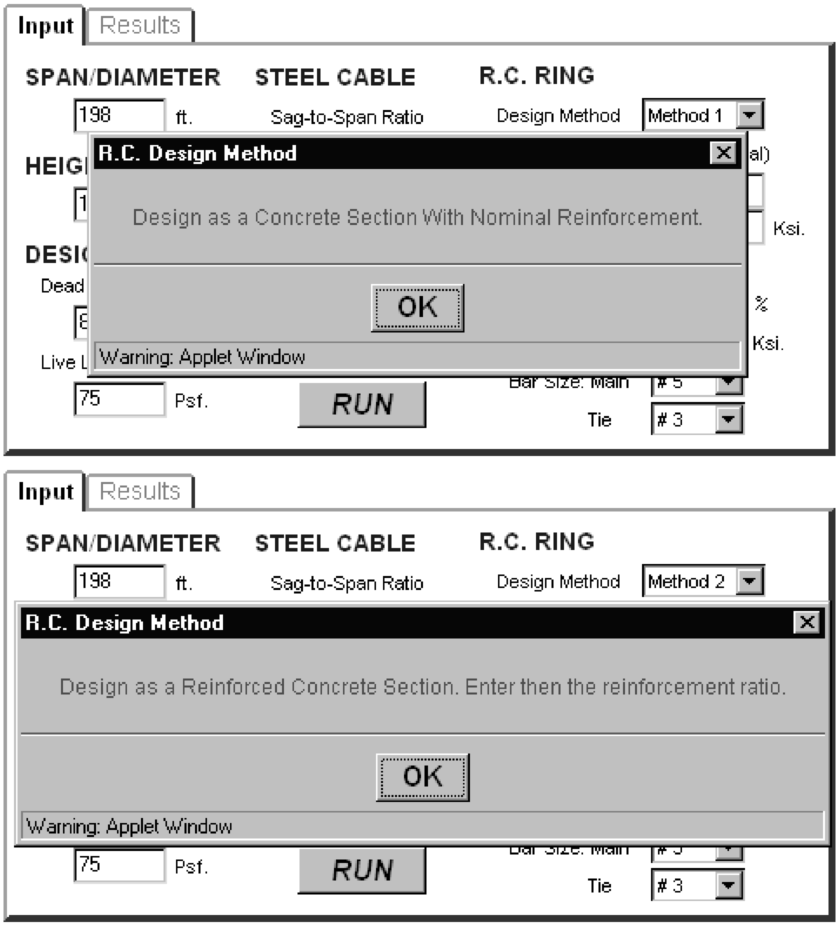
Strength: 60 Ksi.

Bar Size: Main: # 5

Tie

Figure 4.10 Selection of lateral reinforcement bars from the input panel respect to its practical value.

f. Reinforcement ratio less than the minimum requirement or greater than the maximum value according to the ACI specification.



SPAN/DIAMETER 198 ft.

Design Method: Method 1

R.C. Design Method

Design as a Concrete Section With Nominal Reinforcement.

Warning: Applet Window

Live Load: 75 Psf.

Tie: # 3

SPAN/DIAMETER 198 ft.

Design Method: Method 2

R.C. Design Method

Design as a Reinforced Concrete Section. Enter then the reinforcement ratio.

Warning: Applet Window

75 Psf.

Tie: # 3

Figure 4.11 The applet shows the description of design methods by a pop-up message

Error

The acceptable ratio of reinforcement is in the range of 1.0-8.0% of concrete area.

Figure 4.12 Applet warns the user when the entered reinforcement ratio is not a practical one

By clicking the *RUN* button, the user can start the applet to design. After the design is completed the applet shows the completion message (Figure 4.13). Then the user can check the summary of suspension roof design in the results panel (Figure 4.9). Results panel displays the plan of roof, vertical cross section of the structure, and the cross-section of R.C. ring. In addition to these graphical displays, the applet shows all the design details such as the span of roof, height of structure, sag of cables, number of cables, spacing of cables, load distribution on cables, and the size of the main bars and tie bars.

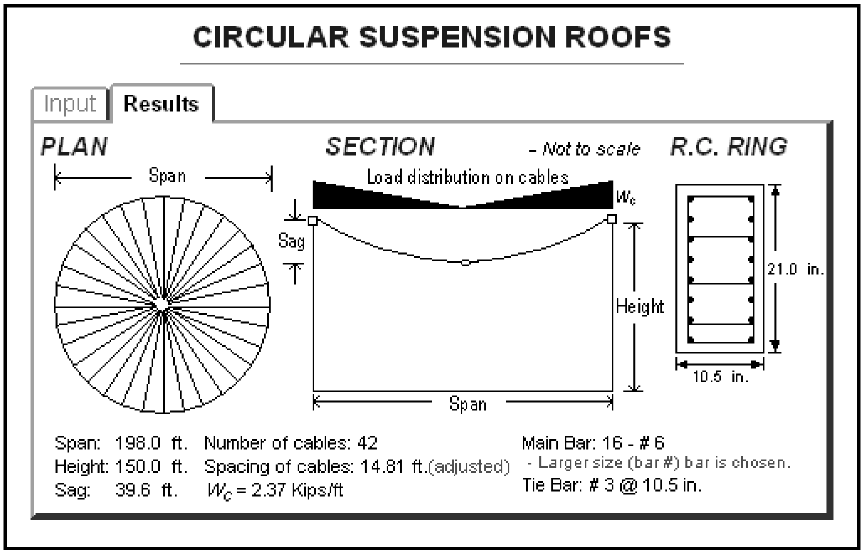
OK

Design completed.

Figure 4.13 Design completion message

If the size of the main bar given by the user cannot satisfy the design requirements, the applet tries larger size steel bars and informs the user accordingly (Figure 4.14). In Figure 4.14, the applet shows that the spacing of cables is adjusted. This is done when the spacing given by the user does not results in an even number.

This applet as well as all other applets created for the course are intended to let the student perform the design with minimum amount of data entry. Building upon the idea of the *Redesign Menus* presented in Adeli (1988), the user can perform redesigns repeatedly by simply changing only one or several of the input values. Based on the information obtained from the initial design, the user can modify or limit any design parameter and request a new design without the need to start all over again. This is an effective tutorial feature considering the open-ended nature of the design problems. The student can find the answers to



```
Span:   198.0  ft.   Number of cables: 42                          Main Bar: 16 - # 6
Height: 150.0  ft.   Spacing of cables: 14.81  ft.(adjusted)        - Larger size (bar #) bar is chosen.
Sag:     39.6  ft.    Wc = 2.37 Kips/ft                            Tie Bar: # 3 @ 10.5 in.
```

Figure 4.14

a lot of what-if scenarios in a very short period of time. Not only this boosts the learning experience tremendously but also makes it more interesting.

4.7 PROBLEMS

4.1 An angle L9 x 4 x ½ made of A441 steel (F_y = 42 ksi) is used as a tension member. Connection of the angle is through 7/8-in.-diameter bolts. The pattern of holes in the angle is as shown in Figure 4.15 (with two lines of holes in the long leg and a single line of holes in the short leg). Find the allowable tensile force for this member.

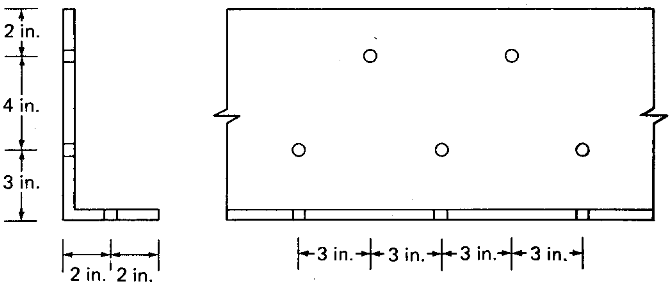
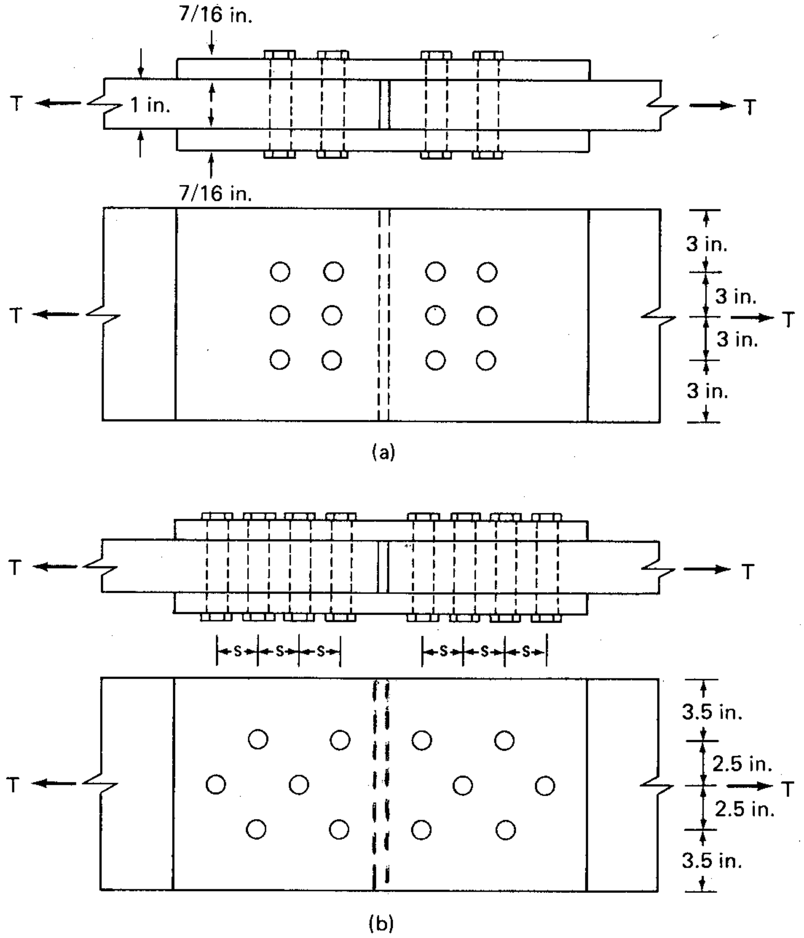


Figure 4.15

4.2 Two main plates 12 in. x 1 in. subjected to tension T have been connected to each other by two splice plates 12 in. x 7/16 in. and one-inch-diameter bolts as shown in Figure 4.16(a). The plates are made of A441 steel with yield stress of 40 ksi and ultimate stress of 60 ksi. Assuming that the bolts do not control the design of connections, determine the maximum tension capacity of the connection. Next, in order to increase the efficiency of the connection, holes are staggered as shown in Figure 4.16(b). For what spacing *s* of the holes and bolts will the tension capacity of the connection be the largest? For this spacing find the percentage increase in the capacity of the connection due to staggering of the bolts. (*Note*. You should refer to ASD B3 and J3.8).



(a)

(b)

Figure 4.16

4.3 The fatigue-critical member of a bridge truss consists of 2L8 x 6 x 1 connected to a gusset plate by fillet welds, as shown in Figure 4.17. Angles are made of A36 steel (F_y = 36 ksi). The member is subjected to a dead load tensile force of 50 K and a live load varying from 30 K in compression to 178 K in tension. Find the design lifetime of the bridge. The average daily number of standard vehicles passing over the bridge and producing the above-mentioned live load variation is 182. (*Note*. Refer to Appendix K of ASD.)

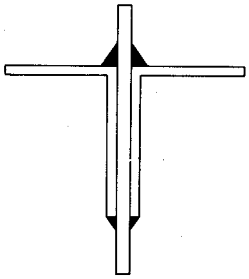


Figure 4.17

4.4 Design the members of the bottom chord of the truss of Figure 4.18. Each member shall consist of two angles with a spacing of ½ in. to be filled by gusset plates at joints. The two angles shall be connected to the gusset plate with two 1-in. bolts in the vertical leg of the angles. Use A36 steel (F_y = 36 ksi).

4.5 Find the size of the main cables and the cross-sectional dimensions of the rectangular concrete compression ring for a circular suspension roof covering an area of diameter 305.6 ft. supporting a combined dead and live load of 60 psf. Allowable tensile stress of cable steel is 65 ksi. Compressive strength of concrete is 4 ksi. Use a sag-to-span

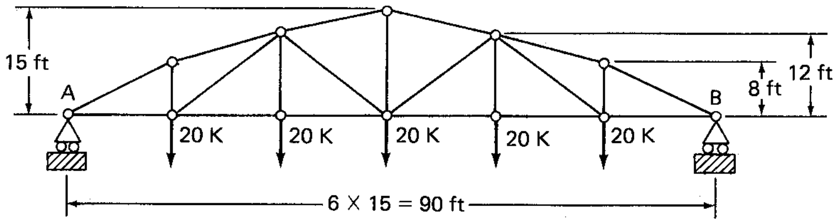


Figure 4.18

ratio of 0.2 for the cables and a width-to-depth ratio of ½ for the cross section of the compression ring. Solve the problem for three different cable spacing at the compression ring of about 8 ft., 12 ft., and 15ft.

4.6 Solve Problem 4.5 with a sag-to-span ratio of 0.25 for the cables.

4.7 Solve Example 2 of this chapter, assuming that the diameter of the structure is 60 ft., the dead load is 70 psf, and the live load 75 psf. Select the lightest channel (C section) available in the ASD for the hangers.