CE 312
Structural Analysis and Design Sessional-I
(Lab Manual)

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December, 2017

Preface

Structural Analysis and Design Sessional-I (CE 312) manual contains the analysis and design of an industrial roof truss and a plate girder. For providing a complete guideline to the students, basic design concepts of roof truss and plate girder are elaborated with examples and detailed drawings in this manual. Design of support and anchorage system is also discussed for a complete understanding of the students. This manual is prepared using AISC and BNBC standards. The main objective of this manual is to provide the students with sufficient fundamental knowledge about analysis and design of steel members and connections. It is designed to familiarize the students with practical problems and also to develop their ability to design steel structural systems. The manual will also introduce the student to design guidelines that are commonly used by practicing structural engineers.

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Part 1: Steel Fundamentals

Steel Structure: a structure which is made from organized combination of structural steel members designed to carry loads and provide adequate rigidity. Steel structures involve sub-structure or members in a building made from structural steel

Types of Steel Structure:

Truss structures (bar or truss members)
Frame structures (beams and columns)
Grids structures
Arch
Prestressed structures

Beam bridge
Truss bridge
Arch bridge
Cable-stayed bridge
Suspension bridge

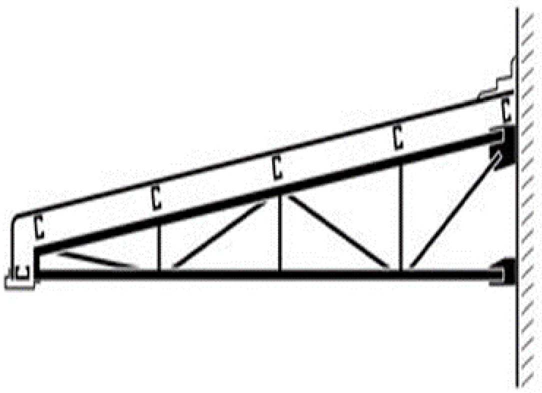


Figure 1.1(a): Truss Structure

Figure 1.1(b): Frame Structure (Structural steel frame, n.d.)

Figure 1.2(a): Truss bridge (Trenton through truss bridge, 2008)

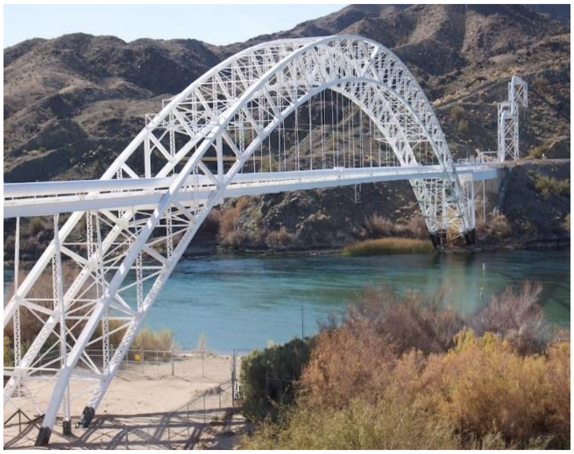


Figure 1.2(b): Arc bridge (Old trails arch bridge, n.d.)

Advantages of Steel:

a) High strength per unit weight especially when compared to concrete. This can reduce the size of the elements in the structure and increase the living space.
b) Uniformity: that reduces the effect of time on steel as compared to concrete that changes throughout its life.
c) Elasticity: steel is elastic, that is it follows Hook's Law as long as its stresses do not exceed its yielding stress. So, steel behaves closer to design assumptions as compared to other materials
d) Moment of inertia of steel is accurately calculated where as that of concrete changes as the cracks move up towards the neutral axis and past it.
e) Durability and performance: If properly maintained the properties of steel do not change appreciably with time.
f) Ductility: since steel is a ductile material, it can undergo extensive deformations after which increased stresses are required for failure to occur. This is a property that can save lives.
g) It is easier to add to a steel structure than it is to a concrete structure mainly due to connections.
h) It is faster to build a steel structure than it is a concrete structure due to its lightness compared to concrete, it requires no curing time, and the members are easily connected (bolted, welded, and riveted).
i) Reliability: Steel Structures are very reliable. The reason for this reliability is uniformity and consistency in properties, and better-quality control because of factory manufacture.
j) Possible Reuse: Steel sections can be reused after a structure has been disassembled. Steel also has very good scrap value.

Disadvantages of Steel:

a) Maintenance cost: steel requires maintenance against corrosion. However, this cost may be eliminated by using atmospheric corrosion-resistant steel such as A242 and A588.
b) Fireproofing costs: steel will not ignite. However, at 1200°F steel has very little strength. Its temperature should not exceed 800°F beyond which its strength is reduced quickly.
c) Buckling: can occur when long slender steel members are exposed to compressive loads. To avoid buckling, a larger cross-section is needed which will increase cost.
d) Fatigue: is caused by a large number of repetitive tensile stress variations. This can reduce the strength and ductility of the steel causing a sudden failure.
e) Aesthetics: A considerable amount of money has to be spent on steel structure to improve their appearance.

Steel Design Specifications:

The specifications of most interest to the structural steel designer are those published by the following organizations.

- American Institute of Steel Construction (AISC)
- American Association of State Highway and Transportation Officials (AASHTO)
- American Railway Engineering and Maintenance-of-Way Association (AREMA)
- American Iron and Steel Institute (AISI)

Design Methodology

The design of a structural member entails the selection of a cross section that will safely and economically resist the applied loads. The fundamental requirement of structural design is that the required strength not exceed the available strength; that is,

$$\text{Required strength} \leq \text{available strength}$$

Design for strength is performed according to the provisions for load and resistance factor design (LRFD) or to the provisions for allowable strength design (ASD).

Allowable Strength Design (ASD):

In this method a member is selected that has cross-sectional properties such as area and moment of inertia that are large enough to prevent the maximum applied axial force, shear, or bending moment from exceeding an allowable, or permissible, value. This allowable value is obtained by dividing the nominal or theoretical, strength by a factor of safety.

This can be expressed as,

$$Allowable \text{ strength } = \frac{Nominal\ strength}{Safety\ factor}$$

Load and resistance factor design (LRFD)

In this method load factors are applied to the service loads, and a member is selected that will have enough strength to resist the factored loads. In addition, the theoretical strength of the member is reduced by the application of a resistance factor. The criterion that must be satisfied in the selection of a member is

$$Factored\ load\ \leq factored\ strength$$

In this expression, the factored load is actually the sum of all service loads to be resisted by the member, each multiplied by its own load factor. For example, dead loads will have load factors that are different from those for live loads. The factored strength is the theoretical strength multiplied by a resistance factor.

$$\Sigma(loads\ \times load\ factors\ ) \leq resistance\ \times resistance\ factor$$

Structural Steel:

Steel Grade: Different grades of structural steel are identified by the designation assigned them by the American Society for Testing and Materials (ASTM).

Property	*A36*	*A572 Gr. 50*	*A992*
Yield point, min.	*36 ksi*	*50 ksi*	*50 ksi*
Tensile strength, min.	*58 to 80 ksi*	*65 ksi*	*65 ksi*
Yield to tensile ratio, max.	—	—	*0.85*

Classification of structural steel:

- **Compact:** Section reaches its full strength (yield) before local buckling occurs. Strength of section is governed by material strength
- **Non-compact:** Only a portion of the cross-section reaches its full strength (yield) before local buckling occurs
- **Slender:** Cross-section does not yield before local buckling occurs. Strength is governed by buckling

AISC classifies cross-sectional shapes as compact, noncompact, or slender, depending on the values of the width-to-thickness ratios. Classification are given in AISC Table B4.1

Notation:

λ = width / thickness ratio

λ_p = upper limit for compact category

λ_r = upper limit for non-compact category

If $\lambda \leq \lambda_p$ and the flange is continuously attached to the web, the shape is compact

If $\lambda_p \leq \lambda \leq \lambda_r$, the shape is non-compact

If $\lambda > \lambda_r$, the shape is slender (These values are discussed later in the manual)

The category is based on the worst width-to-thickness ratio of the cross section. For example, if the web is compact and the flange is noncompact, the shape is classified as noncompact.

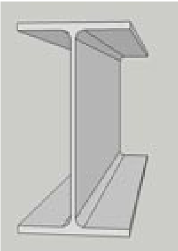
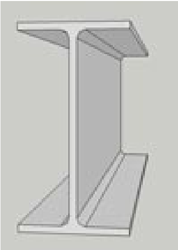
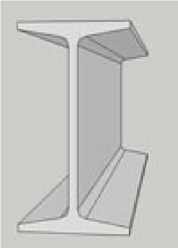
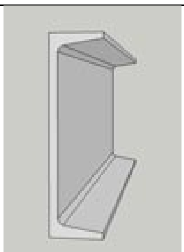
Types of Structural Steel:

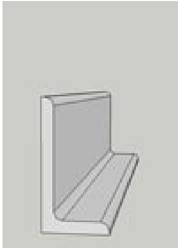
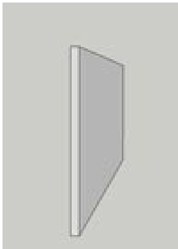
Hot Rolled Steel: Hot rolling is a mill process which involves rolling the steel at a high temperature (typically at a temperature over 1700° F), which is above the steel's recrystallization temperature. When steel is above the recrystallization temperature, it can be shaped and formed easily, and the steel can be made in much larger sizes. Hot rolled steel is typically cheaper than cold rolled steel due to the fact that it is often manufactured without any delays in the process, and therefore the reheating of the steel is not required (as it is with cold rolled). When the steel cools off it will shrink slightly thus giving less control on the size and shape of the finished product when compared to cold rolled.

Cold Rolled Steel: Cold rolled steel is essentially hot rolled steel that has had further processing. The steel is processed further in cold reduction mills, where the material is cooled (at room temperature) followed by annealing and/or tempers rolling. This process will produce steel with closer dimensional tolerances and a wider range of surface finishes. The term Cold Rolled is mistakenly used on all products, when actually the product name refers to the rolling of flat rolled sheet and coil products.

Built-up Section: Built -up members are obtained by connecting two or more plates or shapes which then act as a single member. Such members may be made necessary by requirement of the area, which can't be provided by a single rolled shape, or by the requirement of rigidity because for the same area, much greater moment of inertia can be obtained with built-up sections compared to single rolled shapes, or by the requirement of suitable connection, where the width or depth of member necessary for proper connection can't be obtained in a standard rolled section.

Standard rolled Shapes (Structural steel shapes, n.d.)

Symbol	Type of shape	Description	Figure
W	Wide Flange	Flange surfaces are parallel; flange thickness is not necessarily equal to the web thickness.	
HP	Bearing Pile	Flange surfaces are parallel; flange and web have equal thicknesses.	
S	American Standard Beam	The inner flange surface is sloped.	
C	Channel	Standard AISC flanges have sloped inner flange surfaces.	

Symbol	Type of shape	Description	Figure
WT ST MT	Tee	WT shapes are cut from a wide flange. ST shapes are cut from American Standard Beams. MT shapes are cut from non-standard I-shapes.	
HSS TS	Hollow Steel Section Steel Tube	Either nomenclature is acceptable; however, HSS is more common.	
L	Angle	Angles come in equal leg or unequal leg sizes. The diagram at left shows an unequal leg.	
Pipe	Pipe	--	
PL	Plate	Very small plates can also be called bars.	

Loads encountered in structural steel design

To be able to design a safe, efficient and economical structure, we have to have an accurate idea of the types of loads the structure will be exposed to during its life time, and what combinations of these loads can occur at the same time.

Types of Loads:

- Dead Loads: have a constant magnitude and a fixed position. That includes the structures own weight and anything fixed to it. However, to estimate the structures weight we have to know that members are being used. Therefore, we assume the members then check our results. The more experience the designer has, the lower the number of member estimates he has to do.

- Live loads: change in magnitude and position. If it is not a dead load then it is a live load. Live loads are of 2 types: Moving loads that move by their own power (cars and trucks). Movable loads (furniture). Few examples of live loads are:
    1. floor loads
    2. Snow and ice
    3. Rain especially on flat roofs because ponding develops causing deflections.
    4. Traffic loads for bridges.
    5. Impact loads: such as falling objects or sudden car braking.
    6. Lateral loads: such as wind, which changes with height, geographic location, surrounding structures
    7. Earthquakes are another example of impact loads.
    8. longitudinal loads: such as sudden stopping of trains or trucks on bridges.
    9. Other live loads: soil pressure on walls or foundations, water on dams, explosions, thermal forces due to temperature changes…...etc.

Part 2: Design of an Industrial Steel Roof Truss

2.1 Introduction

A truss is a structure composed of slender members joined together at their end points. Planar trusses lie in a single plane. Typically, the joint connections are formed by bolting or welding the end members together to a common plate, called a gusset plate. The basic building block of a truss is a triangle. Large trusses are constructed by attaching several triangles together. A new triangle can be added truss by adding two members and a joint. A truss constructed in this fashion is known as a simple truss.

2.2 Assumptions:

The main assumptions made in the analysis of truss are:

- Truss members are connected together at their ends only.
- Trusses are connected together by frictionless pins.
- The truss structure is loaded only at the joints.

2.3 Advantages of Truss:

Quick Installation

The primary advantage of a truss is that it can be installed quickly and cost-effectively, even without heavy equipment to lift it into place. Most trusses are factory-built, and delivered to the job site as a complete set for the structure to be built. A truss is traditionally leveraged to the top of the wall, and then slid into position and pivoted upright before being fastened in place.

Increased Span

The unique properties of a triangular object allow trusses to span across longer distances. Where a square-sided roof would tend to shift or twist, a triangular one maintains its shape, preventing shift and sag. As a further advantage, the entire set of trusses combined becomes stable and able to support many times the weight of a non-reinforced straight roof.

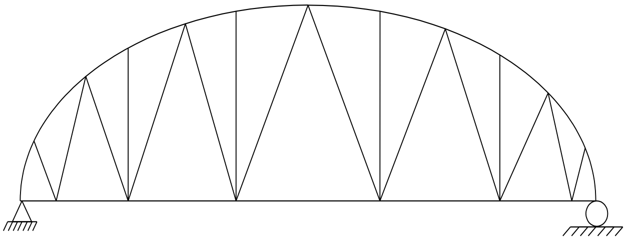
Load Distribution

The shape of a triangle allows all of the weight applied to the sides (or legs) to be redistributed down and away from the center. In trusses, this transfers the entire weight of the roof to the outer walls, and has the advantage of allowing the interior walls to be built arbitrarily, or even moved or omitted.

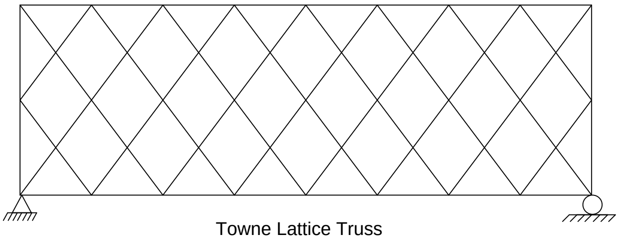
Accessibility

Since the bottom rail of a truss is typically the ceiling of the rooms below, the triangular spaces of the trusses themselves form accessible paths for the installation of electric and other utility applications. The central void of a truss system is generally the attic of a home, with the slope of the roof forming the legs of the triangle.

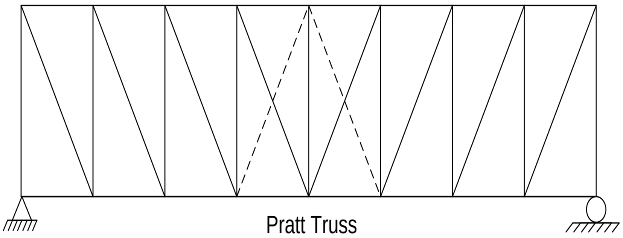
2.4 Types of Truss:



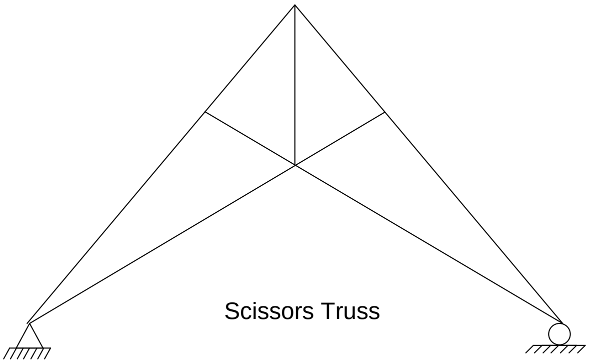
Bowstring Truss



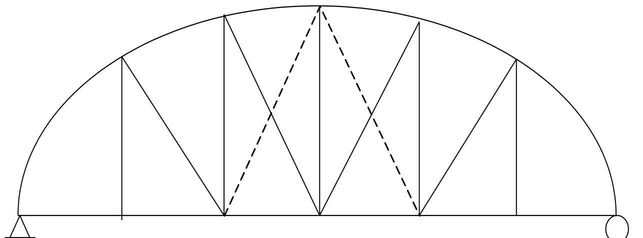
Towne Lattice Truss



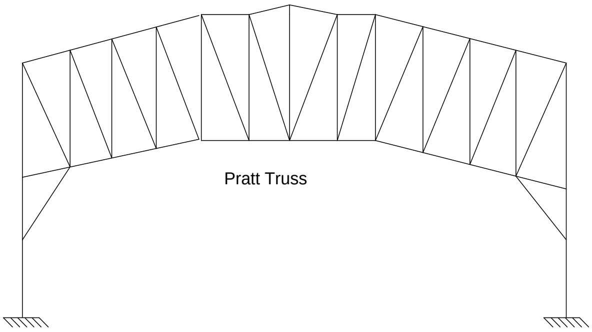
Pratt Truss



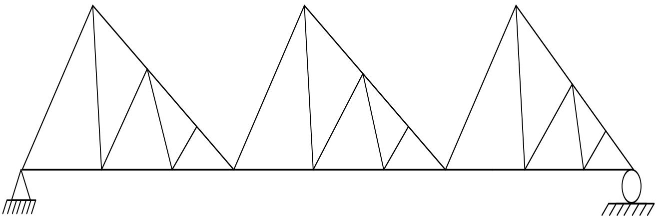
Scissors Truss



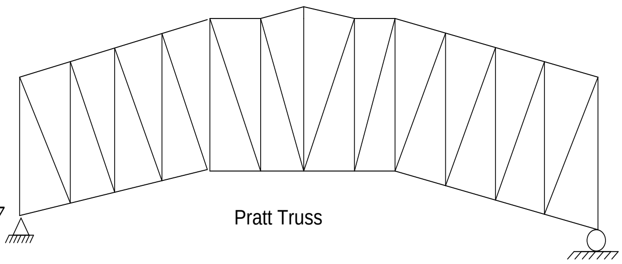
Camel Back Pratt



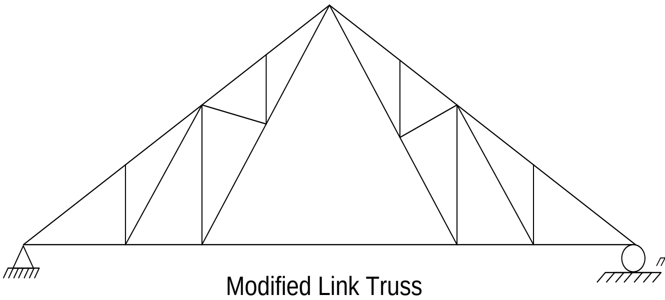
Pratt Truss



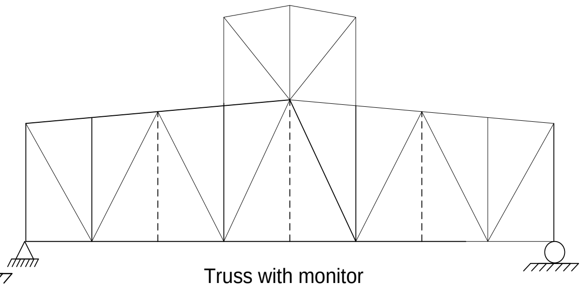
Saw-Tooth Truss



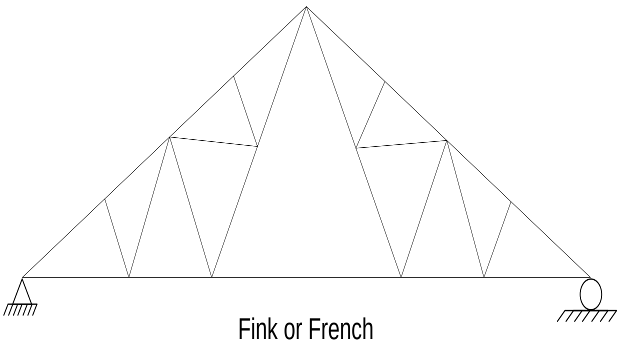
Pratt Truss



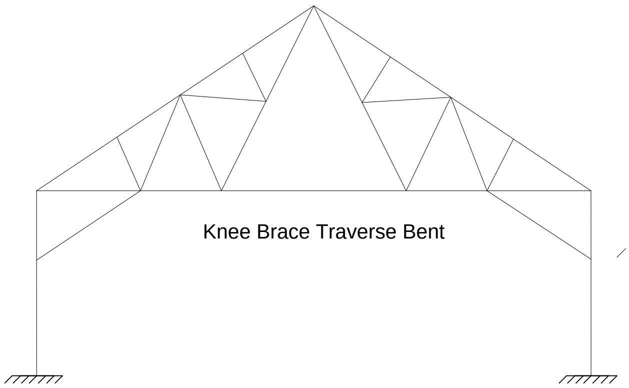
Modified Link Truss



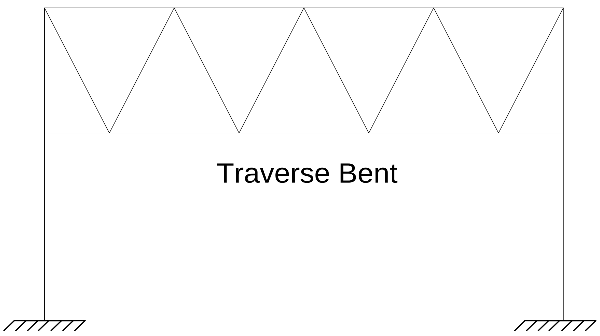
Truss with monitor



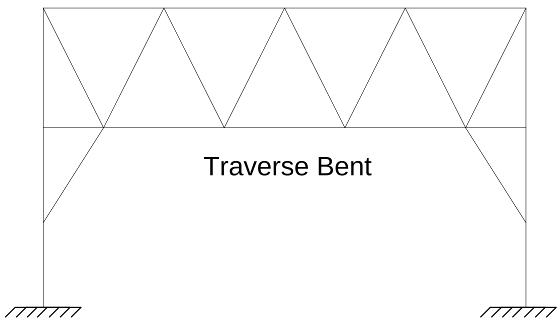
Fink or French



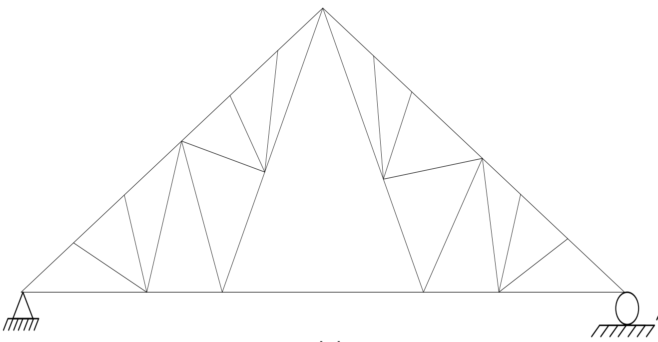
Knee Brace Traverse Bent



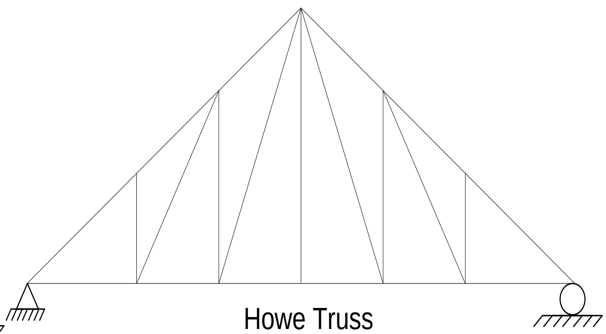
Traverse Bent



Traverse Bent



Fan Fink



Howe Truss

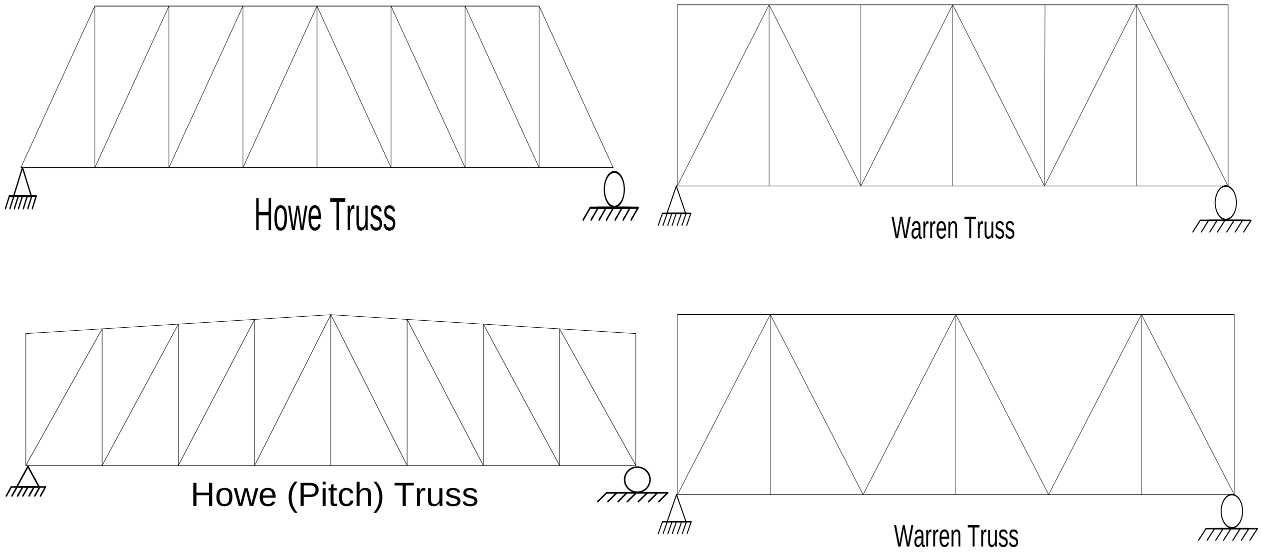


Figure 2.1: Different types of truss

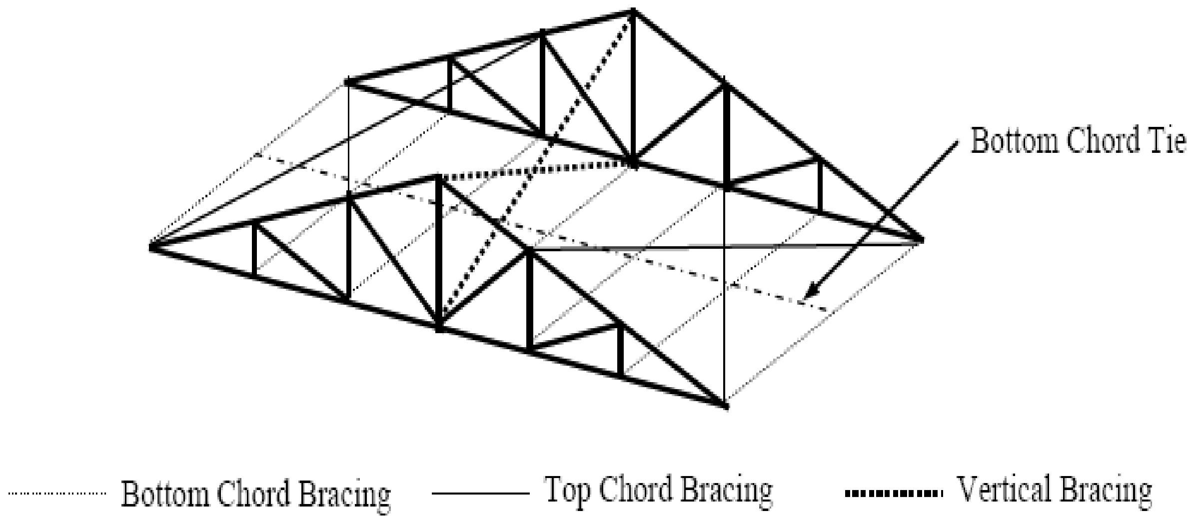


Figure 2.2: Bracing System of truss

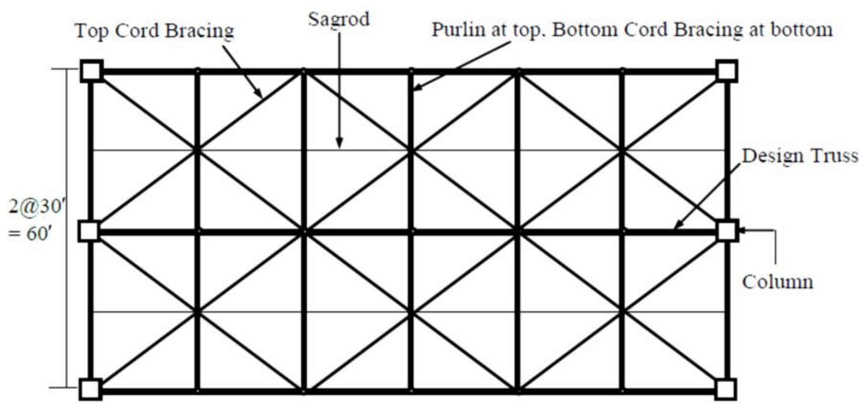


Figure 2.3: Building Plan

2.5 Roof Truss Design

Design a Pratt type roof truss from the following data:

Design Data:

Span = 40 feet

Span-to-rise ratio (pitch) = 4:1

Rise = 10 feet

Slope (θ) = $\tan^{-1}(10/20) = 26.5651^0$ (degree)

Bay distance (truss-to-truss distance) = 25 feet.

Location: Dhaka, Basic wind speed = 210 Km/h.

Exposure category: Exposure A

Truss is supported on brick wall of height = 12 feet.

Design Loads:

(1) Dead load:

Self-weight of truss = 60 lb per ft. horizontal span of truss.

Sag rod + bracing = 1 psf. (approximately known)

C.G.I. sheet roofing = 2 psf. (known)

Purlin (self-weight) = 1.5 psf. (assumed)

(2) Wind load = according to BNBC 1993 (Bangladesh National Building Code 1993).

(3) Snow load = not applicable for our country.

Design Method:

Design method followed here is AISC/ASD

Steel to be used: A36 (Yield stress (F_y = 36 ksi)

Electrode to be used: E60XX (electrode material tensile strength (F_{EXX}) = 60 ksi)

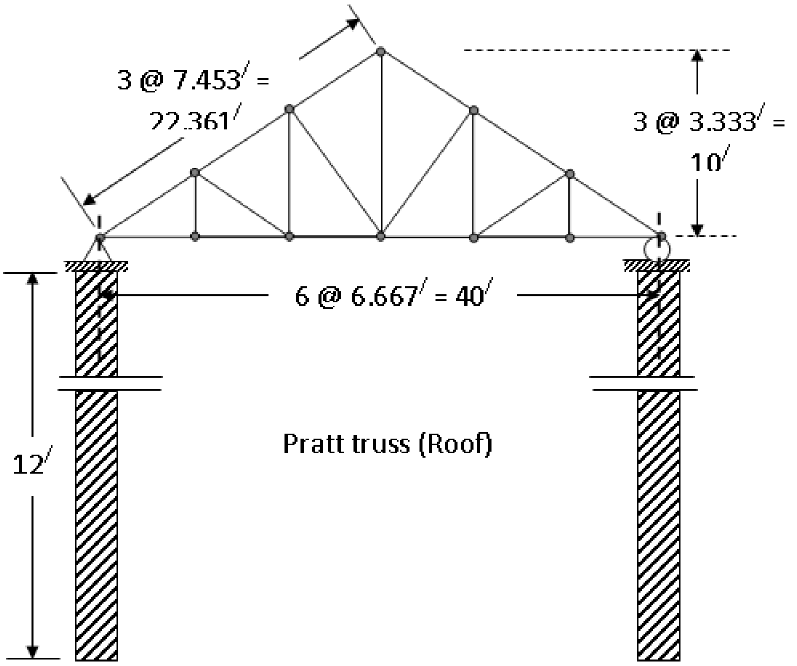


Figure 2.4: Pratt type Roof Truss

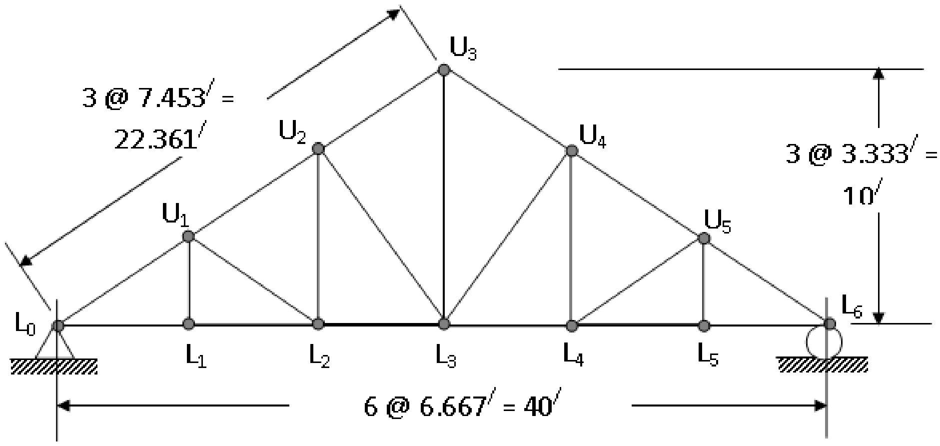


Figure 2.5: Truss notations (member numbering): Pratt truss (Roof)

2.6 Analysis and Design of Purlins:

- Analysis and design of purlin for dead load
- Analysis and design of purlin for dead load plus wind load

2.6.1 Analysis and Design of Purlin for Dead Load:

Purlins are nothing but beams. They span between the adjacent trusses, i.e. the spacing of the trusses is the span of purlins. Purlins are placed at top chord joint. Since the principal axes of the purlin are inclined, the dead load causes bi-axial bending in the purlins. A component of dead load acts in the negative Y direction and the other component acts in the X direction. For the loads acting along Y-axis, the purlin acts as a simply supported beam (see figure 7) of span 25 feet (bay distance). Due to the presence of sagrods, the midspan deflection is restrained in the X direction. As a result the purlin act as a continuous beam (see figure 8) for bending in the plane of the roof surface (X direction).

The dead load coming on the purlin is from the roofing material and the self-weight of the purlin itself. Weights of the sagrods are so small compared to the other loads that we can safely neglect it.

⇒ Calculation of total dead load on purlin:

C.G.I sheet roofing = 2 psf (known)

Self-weight of purlins = 1.5 psf (assumed but will be checked later)

Sagrod weight = negligable

Total dead load = 3.50 psf

⇒ Uniformly distributed load (UDL) on purlin, W_{DL}= 3.50 psf × purlin spacing

= 3.50 psf × 7.453 ft.

= 26.0855 lb. per feet

Component of W_{DL} in X direction, $W_{DLx} = W_{DL} \times \sin\theta$

$= 26.0855 \times \sin 26.565^0$

= 11.666 lb. per feet

Component of W_{DL} in Y direction, $W_{DLy} = W_{DL} \times \cos\theta$

$= 26.0855 \times \cos 26.565^0$

= 23.332 lb. per feet

⇒ Purlin span = 25 feet for loading in Y direction (loading perpendicular to the plane of roof surface).

⇒ Purlin span = 12.5 feet + 12.5 feet for loading X direction (loading in the plane of roof surface)

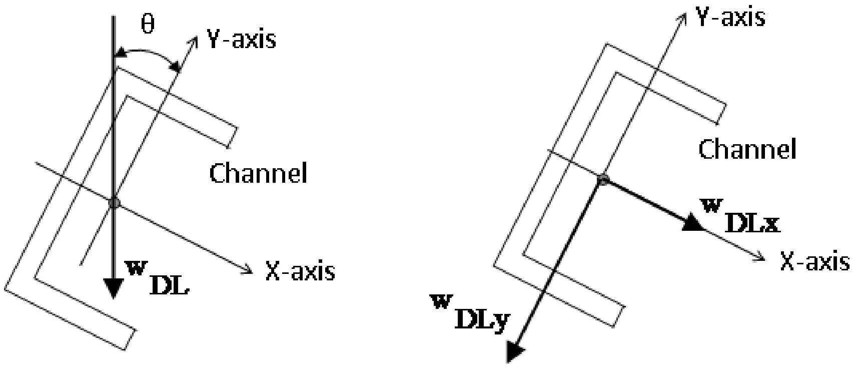


Figure 2.6: Bi-axial loading on the purlin

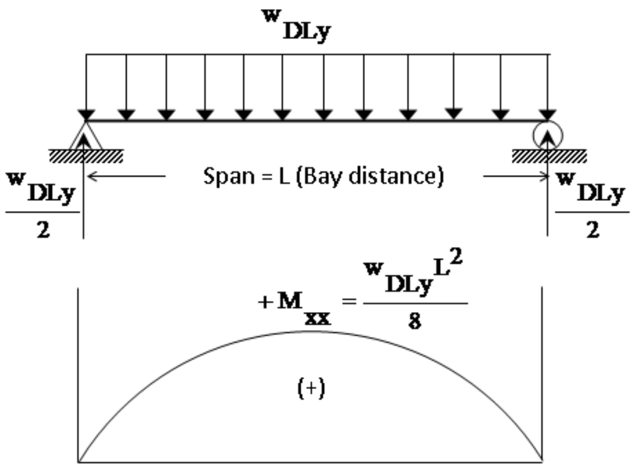


Figure 2.7: Bending moment diagram for loading in Y direction (loading perpendicular to the roof surface, bending parallel to the roof)

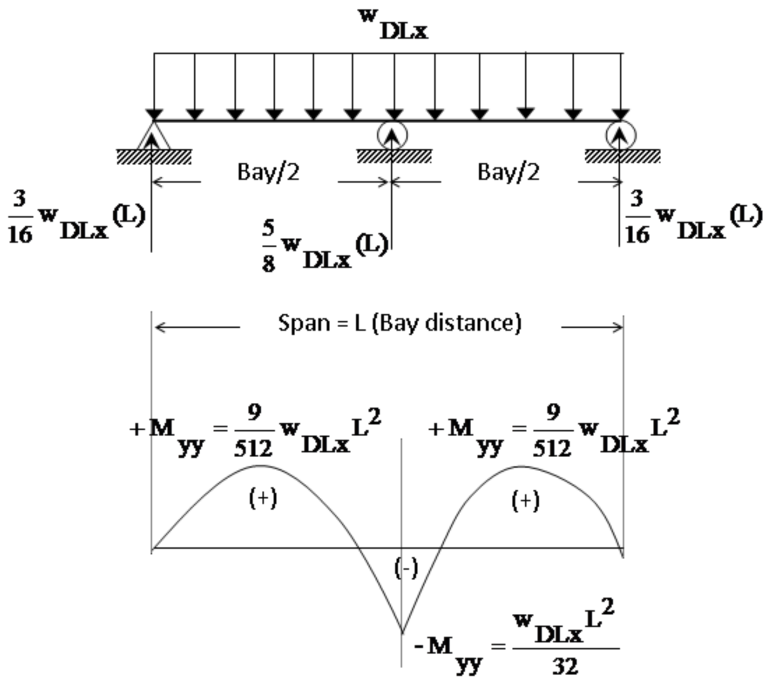


Figure 2.8: Bending moment diagram for loading in X direction (loading parallel to the roof surface, bending perpendicular to the roof)

$\Rightarrow$ Calculation of bending moment:

$$M_{xx} = \frac{W_{DLy} L^2}{8} = \frac{23.332 \times 25^2}{8} = 1822.8125\,\text{ft} - \text{lb} = 1.8228\,\text{kip - ft}$$

$$M_{yy} = \frac{w_{DLx} L^2}{32} = \frac{11.666 \times 25^2}{32} = 227.852\,\text{ft} - \text{lb} = 0.22785\,\text{kip - ft}$$

M_{xx} = moment about X axis (moment in plane of roof surface)

M_{yy} = moment about Y axis (moment perpendicular to the plane of roof surface)

In the design of purlin, we assume that the purlin has adequate lateral bracing due to the presence of roofing and sag-rod so that pure bending will govern the design. As our first trial, we select the smallest available American Standard Channel C 3×4.1. From AISC manual

Channel	S_{xx} (inch^3)	S_{yy} (inch^3)
C 3×4.1	1.10	0.202

S_{xx} & S_{yy} = Section modulus about X axis & Y axis respectively.

⇒ Allowable bending stress, $F_b = 0.66F_y$

For A36 steel, $F_b = 0.66F_y$ = 0.66 × 36 ksi = 23.76 ksi

⇒ Calculation of actual bending stress:

Bending stress developed on purlin section, $f = \pm\frac{M_{xx}(c_y)}{I_{xx}} \pm \frac{M_{yy}(c_x)}{I_{yy}}$

Maximum bending stress developed on purlin section, $f = \frac{M_{xx}(c_y)}{I_{xx}} + \frac{M_{yy}(c_x)}{I_{yy}}$

$$f = \frac{M_{xx}}{(I_{xx}/c_y)} + \frac{M_{yy}}{(I_{yy}/c_x)}$$

$$f = \frac{M_{xx}}{S_{xx}} + \frac{M_{yy}}{S_{yy}}$$

$$f = \frac{1.8228\times12}{1.10} + \frac{0.22785\times12}{0.202} = 33.421\,\text{ksi}$$

Check of bending stress:

Actual bending stress (f = 33.421 ksi) > allowable bending stress (F_b = 23.76 ksi)

Thus, section is not OK. Select a higher section.

Table 2.1: Criteria for adequacy of the section

Criteria	Comments
If, $f < F_b$	Section is OK
If, $f > F_b$	Section is not OK; select a higher section
If, $f << F_b$	Section is OK but not economical; select a lower section

Table 2.2: Purlin section selection for dead load

Section	S_{xx} (inch^3)	S_{yy} (inch^3)	Actual bending stress (f) in ksi	Allowable bending stress (F_b) in ksi	Comments
C 3×4.1	1.10	0.202	33.421	23.76	not OK
C 3×5	1.24	0.233	29.374	23.76	not OK
C 3×6	1.38	0.268	26.053	23.76	not OK
C 4×5.4	1.93	0.283	20.995	23.76	OK
C 4×7.25	2.29	0.343	17.523	23.76	OK but not economical
C 5×6.7	3.00	0.378	14.525	23.76	OK but not economical
C 5×9	3.56	0.450	12.220	23.76	OK but not economical

$\Rightarrow$ Check self-weight of purlin:

For C 4×5.4 channel, self-weight is 5.4 lb/ft which is equivalent to $\frac{5.4\,\text{lb/ft}}{7.4535\,\text{ft}}$ = 0.7245 psf (distributed load over the roof surface) which is smaller than previously/initially assumed purlin self-weight 1.50 psf. So, the purlin C 4×5.4 is adequate for resisting bending moment (i.e. bending stress) & its self-weight is well-below the previously/initially assumed value.

Select a C 4×5.4 section for purlin (mind it, this selection is done only for dead load).

2.6.2 Analysis and Design of Purlin for Dead Load plus Wind Load:

Wind Load Calculation (according to BNBC 1993):

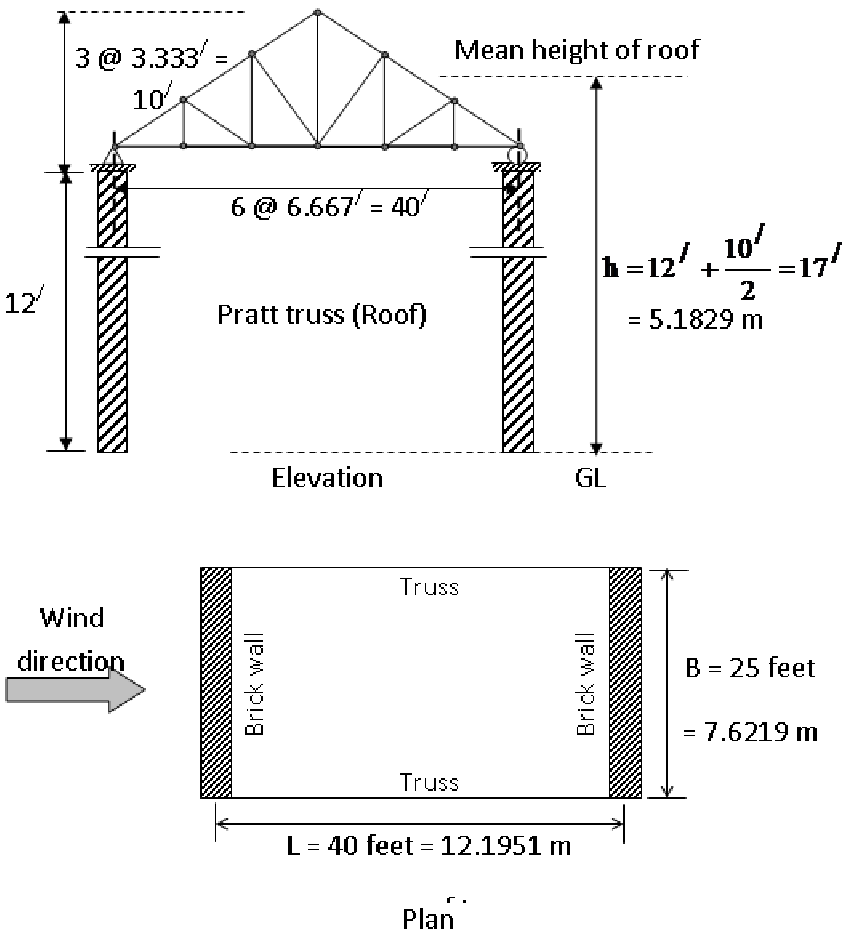


Figure 2.9: Plan and elevation of roof truss

⇒ Different parameters for wind load calculation:

Truss location: Dhaka

V_b= Basic wind speed in km/h = 210 km/h

B = Horizontal dimension of the building, in meters measured normal to wind direction = bay distance (truss-to-truss spacing) = 25 feet = 7.6219 meter.

L = Horizontal dimension of the building, in meters measured parallel to wind direction = span of truss = 40 feet = 12.1951 meter.

H = average/mean height of the roof in meters = 17 feet = 5.1829 meter.

z = Height above the ground in meters

θ = Angle of the plane of roof from horizontal, degrees = 26.5651^0 (degree)

C_c= Velocity-to-pressure conversion co-efficient = 47.2×10^{-6}

C_I = Structure importance co-efficient (a factor that accounts for the degree of hazard to hazard to human life and damage to property) = 1.00 for standard occupancy structures

C_Z = Combined height and exposure co-efficient = 0.3897 for exposure A

*For exposure category A, you can use the table 4 or alternatively you can use the following equation of combined height and exposure co-efficient C_Z.

⇒ Calculation of wind pressure:

$C_z = 0.1879(z)^{0.4435} \geq 0.368$; z = height above ground in meters

Here, z = h = 5.1829 meter (17 feet)

$C_z = 0.1879(5.1829)^{0.4435} = 0.3897\,(> 0.368)$

C_G = Gust response co-efficient = 1.6257 exposure A

C_{pe} = External pressure co-efficient = -0.14345 for windward side, wind direction normal to ridge

C_{pe} = External pressure co-efficient = -0.70 for leeward side, wind direction normal to ridge

q_z = Sustained wind pressure in kN/m^2

p_z = Design wind pressure in kN/m^2

$$q_z = C_c C_I C_z V_b^2$$

$$q_z = (47.2 \times 10^{-6}) \times (1) \times (0.3897) \times (210)^2$$

$$q_z = 0.81137 \text{ kN/m}^2$$

$$p_z = C_G C_{pe} q_z$$

Design wind pressure for windward side: p_z = (1.6257) × (-0.14345) × (0.81137)

$$p_z = -0.189216891 \text{ kN/m}^2 = -3.947747 \text{ psf (suction)}$$

Design wind pressure for leeward side: p_z = (1.6257) × (-0.70) × (0.81137)

$$p_z = -0.923330946 \text{ kN/m}^2 = -19.26419306 \text{ psf (suction)}$$

Positive value of design wind pressure indicates thrust (compression) & negative value of design wind pressure indicates suction (tension). Design wind pressure always acts normal/perpendicular to the roof surface

[1 ksf = 47.89 kN/m^2; 1 kN/m^2 = 20.88 psf; 1 MPa = 1 MN/m^2 = 1 N/mm^2 = 145 psi]

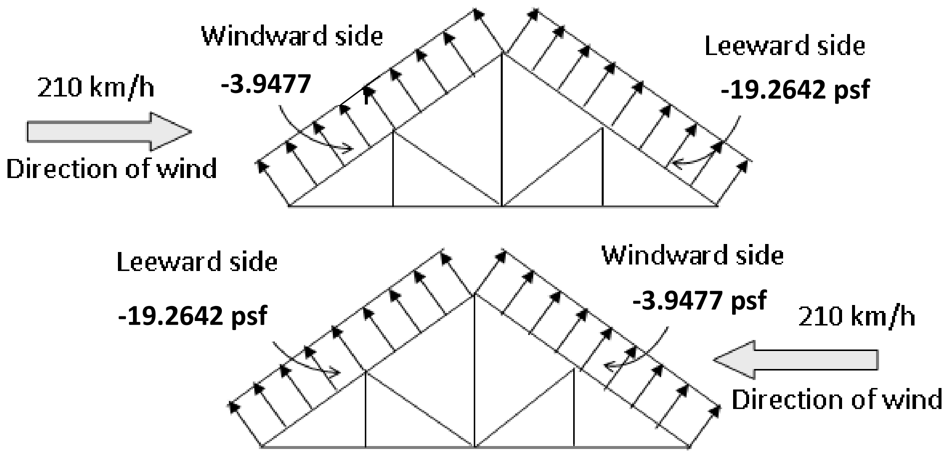


Figure 2.10: Wind direction and pressure distribution on windward side &leeward side

⇒ Calculation of UDL on purlin:

UDL on purlin on windward side = (design wind pressure on the windward side × purlin spacing) = -3.947747 psf × 7.4535 feet = - 29.4247 lb/ft

UDL on purlin on leeward side = (design wind pressure on the leeward side × purlin spacing)

= -19.26419 psf × 7.4535 feet = - 143.5857 lb/ft

Since wind load acts perpendicular to the roof surface, these loads will be combined with the Y component of the dead load (W_{DLy}) to get the resultant load. It is clear from the above that the leeward side will govern since its magnitude is higher.

Resultant load in Y direction, $w_y = w_{DLy} + p_z$

(i.e. resultant load perpendicular to roof surface) = +23.332 lb/ft – 143.5856 lb/ft

$$w_y = -120.2536 \text{ lb/ft}$$

$$M_{xx} = \frac{w_y (L)^2}{8} = \frac{-120.2536 \times 25^2}{8} \text{ ft} - \text{lb} = -9394.8125 \text{ ft} - \text{lb}$$

M_{xx} = -9.3948 kip-ft

Load (w_{DLx}) in X direction (in plane of roof) remains the same, so moment about Y axis remains the same

M_{yy} = 0.22785 kip-ft

⇒ Allowable bending stress, $F_b = 0.66F_y$

For A36 steel, $F_b = 0.66F_y$ = 0.66 × 36 ksi = 23.76 ksi

⇒ Bending stress developed on purlin section, $f = \pm\dfrac{M_{xx}(c_y)}{I_{xx}} \pm \dfrac{M_{yy}(c_x)}{I_{yy}}$

Maximum bending stress developed on purlin section, $f = \dfrac{M_{xx}(c_y)}{I_{xx}} + \dfrac{M_{yy}(c_x)}{I_{yy}}$

$$f = \frac{M_{xx}}{(I_{xx}/c_y)} + \frac{M_{yy}}{(I_{yy}/c_x)}$$

$$f = \frac{M_{xx}}{S_{xx}} + \frac{M_{yy}}{S_{yy}}$$

For previously selected channel section (for dead load) C 4×5.4 (S_{xx} = 1.10 inch^3 & S_{yy} = 0.202 inch^3)

$f = \dfrac{9.3948 \times 12}{1.10} + \dfrac{0.22785 \times 12}{0.202} = 68.075\,\text{ksi}$ > 23.76 ksi (not OK)

Select channel C 6×13 (S_{xx} = 5.80 inch^3 & S_{yy} = 0.642 inch^3)

$f = \dfrac{9.3948 \times 12}{5.80} + \dfrac{0.22785 \times 12}{0.642} = 23.696\,\text{ksi}$ < 23.76 ksi (OK)

⇒ Check self-weight of purlin:

For C 6×13 channel, self-weight is 13 lb/ft which is equivalent to $\dfrac{13\,\text{lb/ft}}{7.4535\,\text{ft}}$ = 1.744 psf (distributed load over the roof surface) which is greater than previously/initially assumed purlin self-weight 1.50 psf. Although the purlin C 6×13 is adequate for resisting bending moment (i.e. bending stress) but its self-weight is high. So, not OK & select another section.

Select channel C 7×9.8 (S_{xx} = 6.08 inch^3 & S_{yy} = 0.625 inch^3)

$f = \dfrac{9.3948 \times 12}{6.08} + \dfrac{0.22785 \times 12}{0.625} = 22.917\,\text{ksi}$ < 23.76 ksi (OK)

⇒ Check self-weight of purlin:

For C 7×9.8 channel, self-weight is 9.8 lb/ft which is equivalent to $\frac{9\,\text{lb/ft}}{7.4535\,\text{ft}}$ = 1.3148 psf (distributed load over the roof surface) which is smaller than previously/initially assumed purlin self-weight 1.50 psf. So, the purlin C 7×9.8 is adequate for resisting bending moment (i.e. bending stress) & its self-weight is well-below the previously/initially assumed value.

Loading	Selected channel section	Check bending stress	Check self-weight
Dead load only	C 4×5.4	OK	OK
Dead load + wind load	C 7×9.8	OK	OK

Finally selected channel for purlins: **C 7×9.8**

2.7 Analysis and Design of Sagrods:

Sagrods prevent the purlin to deflect in the plane of the roof surface at midspan. Thus according to figure 8 the tensile force in the sagrods is equivalent to the midspan reaction.

Sagrod force, $F = \frac{5}{8} w_{DLx} L = \frac{5}{8} \times 11.667\,\text{plf} \times 25\,\text{feet}$

F = 182.28125 lb. = 0.18228125 kip. (tensile)

A round bar of 3/8 inch diameter will be adequate (this is the minimum size). Assuming that the bolts threads will reduce the effective diameter by 1/16 inch, the net cross-sectional area will be $(\pi/4) \times (3/8 - 1/16)^2 = 0.076699039\,\text{inch}^2$. If allowable stress in tension is $F_t = 0.6F_y$. For A 36 steel, $F_t = 0.6F_y = 0.6 \times 36\,\text{ksi} = 21.60\,\text{ksi}$, then this rod will be able to carry a load of 21.60 ksi × 0.076699039 inch^2 = 1.656699 kip, which is well above the actual load (=0.18228125 kip).

2.8 Analysis of the Truss: (Dead load calculation & wind load calculation)

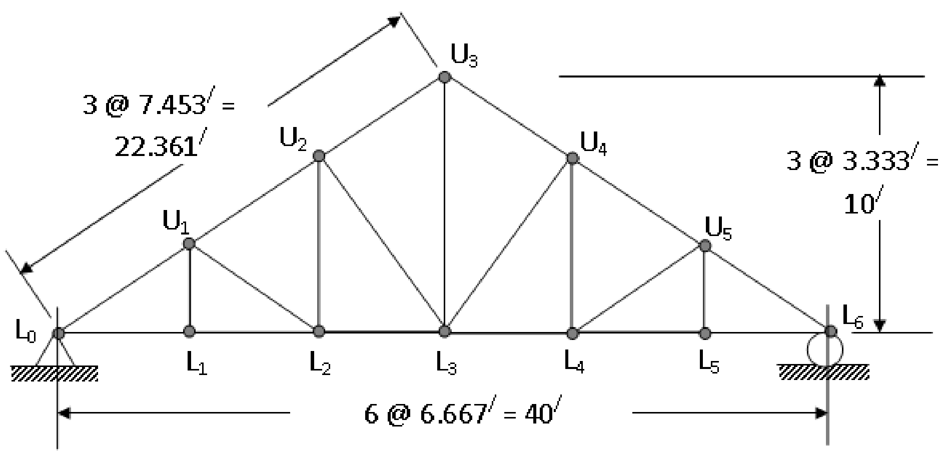


Figure 2.11: Truss notations (member numbering): Pratt truss (Roof)

⇒ **Dead Load Calculation:**

C.G.I sheet roofing = 2 psf (known)

Self-weight of purlins = 1.5 psf (assumed but will be checked later)

Sagrod + bracing = 1psf

Total dead load = 4.50 psf

(Total 4.50 psf dead load) = (4.50 psf) × (purlin spacing) × (bay)

= 4.50 psf × 7.4535 feet × 25 feet

= 838.5255 lb.

Self-weight of the truss (assumed) = 60 lb/ft horizontal span of truss

The self-weight of the truss will be equally divided among the top chord and bottom chord.

Total 60 lb/ft = 30 lb/ft in top chord & 30 lb/ft in bottom chord

⇒ Point loads at the top chord joint due to self-weight = (self-weight distributed in top chord) × (panel spacing along top chord)

= 30 lb/ft × 6.667 ft

= 200.01 lb.

Load in top chord joint (at ridge & internal top chord joint) = 838.5255 lb + 200.01 lb = 1038.5355 lb = 1.0385355 kip.

Load in top chord joint (at support top chord joint) = (838.5255 lb)/2 + (200.01 lb)/2 = 519.26775 lb = 0.51926775 kip.

⇒ Point loads at the bottom chord joint due to self-weight = (self-weight distributed in bottom chord) × (panel spacing along bottom chord)

= 30 lb/ft × 6.667 ft

= 200.01 lb. = 0.20001 kip.

⇒ Load in bottom chord joint (at internal bottom chord joint) = 0.20001 kip

Load in bottom chord joint (at support bottom chord joint) = (0.20001 kip)/2 = 0.100005 kip

Load at support joint = 0.51926775 kip + 0.100005 kip = 0.61927275 kip.

See figure 12 for dead load on the truss.

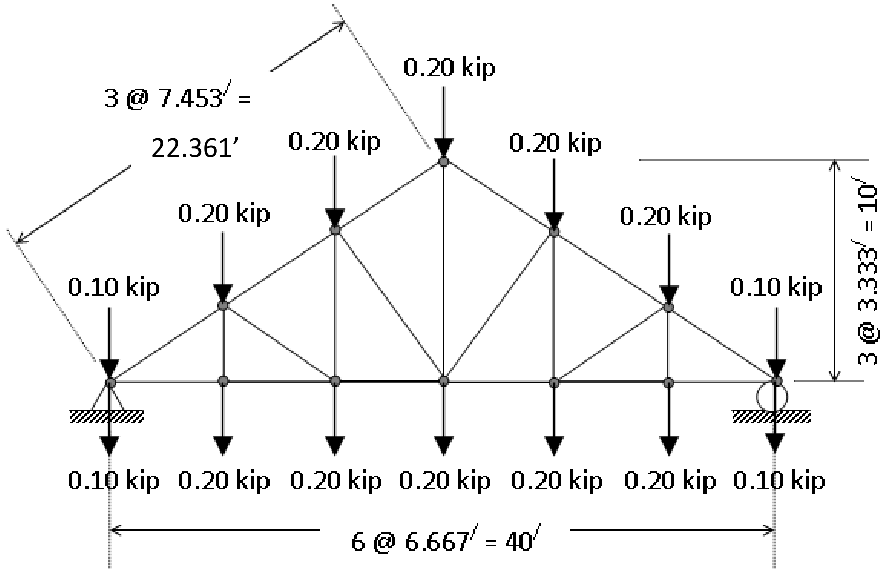


Figure 2.12(a): Dead loads on the truss (self-weight distribution)

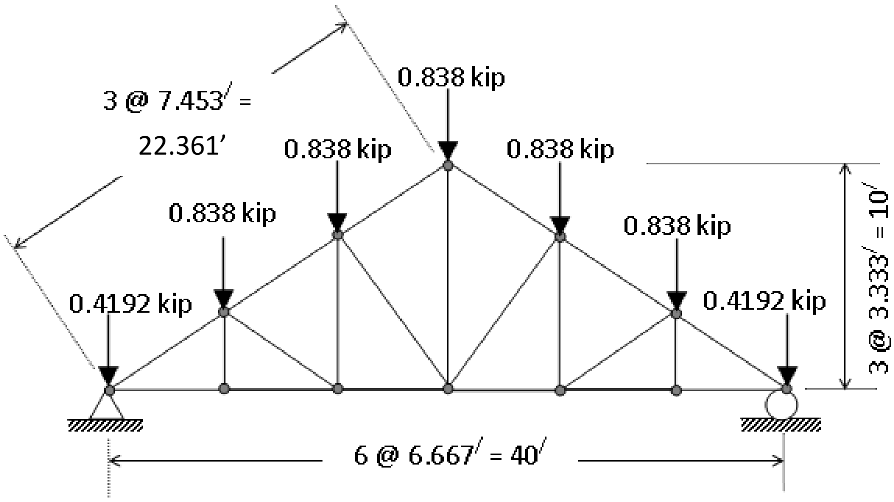


Figure 2.12(b): Dead loads on the truss (sagrod, bracing, purlin self-weight, roof weight distribution)

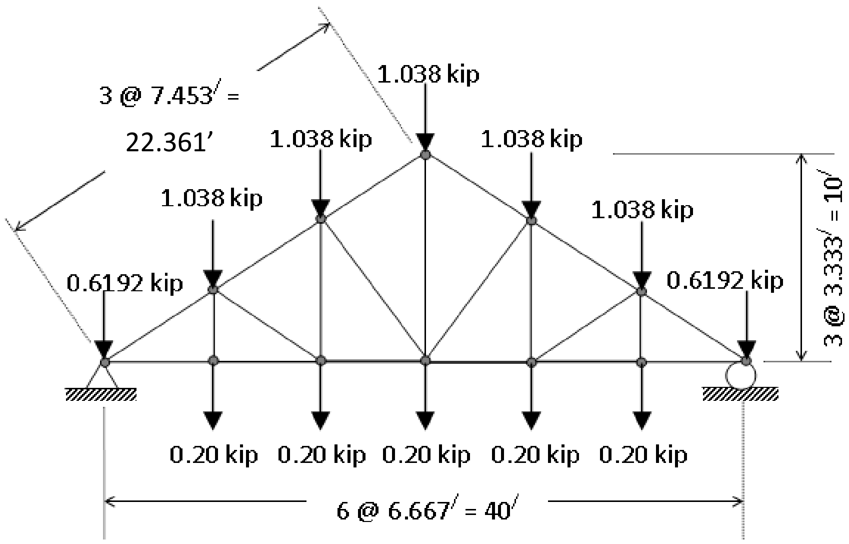


Figure 2.13: Total dead loads on the truss

⇒ **Wind Load Calculation (wind blows from left to right):**

Design wind pressure on windward side = -3.947747 psf

Wind load on interior top chord joint windward side = (design wind pressure on windward side) × (purlin spacing) × (bay)

= -3.947747 psf × 7.4535 feet × 25 feet

= - 735.613 lb = - 0.73561 kip

Wind load on exterior & ridge top chord joint windward side = (design wind pressure on windward side) × (purlin spacing/2) × (bay)

= (-3.947747 psf) × (7.4535/2 feet) × (25 feet)

= - 367.8065 lb = - 0.36781 kip

Design wind pressure on leeward side = -19.26419306 psf

Wind load on interior top chord joint leeward side = (design wind pressure on leeward side) × (purlin spacing) × (bay)

= -19.26419306 psf × 7.4535 feet × 25 feet

= - 3589.641574 lb = - 3.58964 kip

Wind load on exterior & ridge top chord joint leeward side = (design wind pressure on leeward side) × (purlin spacing/2) × (bay)

= (-19.26419306 psf) × (7.4535/2 feet) × (25 feet)

= - 1794.820787 lb = - 1.79482 kip

See figure 14 for wind loading on the truss (for wind blowing from left to right)

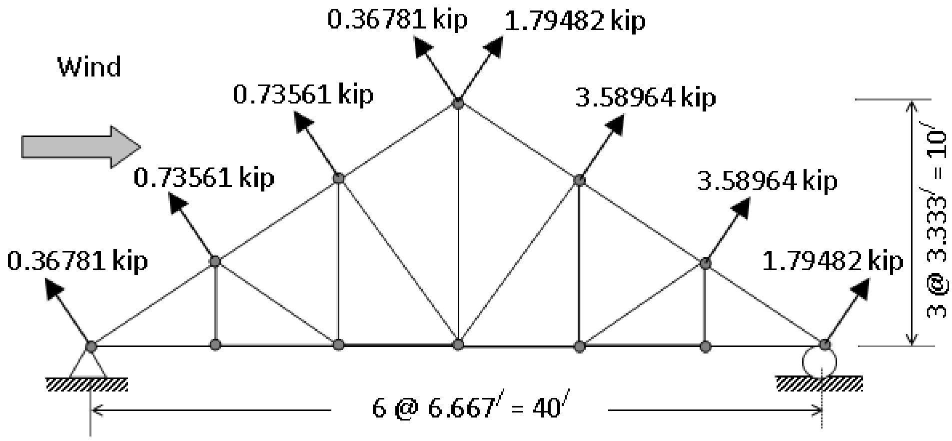


Figure 2.14: Wind loads on the truss for wind blowing from left to right

⇒ Wind Load Calculation (wind blows from left to right):

Mirror image of figure 14 because wind direction is change (previously left-to-right, now right-to-left) → (previous windward side is now leeward side & previous leeward side is now windward side).

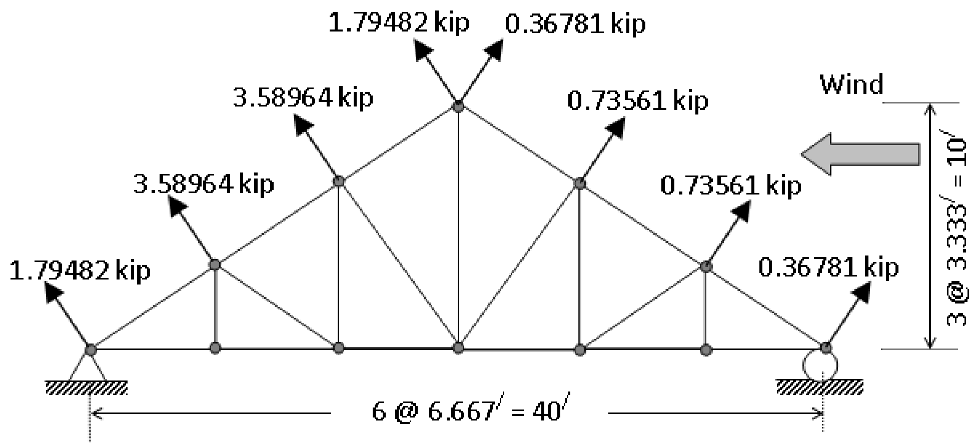


Figure 2.15: Wind loads on the truss for wind blowing from right to left

Note:

Truss loading for	Load direction	
Dead load	Vertically downward	See figure 2.13
Wind load (left-to-right & right-to-left)	Perpendicular to roof surface	See figure 2.14 & 2.15

Truss Analysis:

Truss analysis means determination axial force (which may be either compression or tension) of every of its member/bar. Now, the truss is analyzed for dead load, wind load (left-to-right) & wind load (right-to-left) (this can be done manually or by using computer software)

- Manual analysis for truss: by using method of joint and/or method of section.
- Computer software for truss analysis: GRASP. ETABS, SAP, STAAD etc.

Both manual & computer analysis will have to done & check the bar force value obtained by manual analysis with computer.

Chart: Sample Design Force for truss members

	Member	Length (ft)	Member Force (Kip)			Dead Load only (Kip)	Dead Load + Wind (Left-to-Right) (Kip)	Dead Load + Wind (Right-to-Left) (Kip)	Design member forces (Kip)	
			Dead Load (Kip)	Wind (Left-to-Right) (Kip)+	Wind (Right-to-Left) (Kip)				Tension (Kip)	Compression (Kip)
Top Chord	L0U1	7.4535	-15.73	15.995	33.829	-15.73	0.265	18.099	18.099	-15.73
	U1U2	7.4535	-12.587	17.635	24.742	-12.587	5.048	12.155	12.155	-12.587
	U2U3	7.4535	-9.438	19.255	15.631	-9.438	9.817	6.193	9.817	-9.438
	U3U4	7.4535	-9.438	15.674	19.217	-9.438	6.236	9.779	9.779	-9.438
	U4U5	7.4535	-12.587	24.704	17.597	-12.587	12.117	5.01	12.117	-12.587
	U5L6	7.4535	-15.73	33.791	15.957	-15.73	18.061	0.227	18.061	-15.73
Bottom Chord	L0L1	6.6667	14.07	4.4	-46.737	14.07	18.47	-32.667	18.47	-32.667
	L1L2	6.6667	14.07	4.4	-46.737	14.07	18.47	-32.667	18.47	-32.667
	L2L3	6.6667	11.255	1.979	-33.185	11.255	13.234	-21.93	13.234	-21.93
	L3L4	6.6667	11.255	-13.958	-17.189	11.255	-2.703	-5.934	11.255	-5.934
	L4L5	6.6667	14.07	-27.511	-14.758	14.07	-13.441	-0.688	14.07	-13.441
	L5L6	6.6667	14.07	-27.511	-14.758	14.07	-13.441	-0.688	14.07	-13.441
Verticals	U1L1	3.3333	0.275	0	0	0.275	0.275	0.275	0.275	0
	U2L2	6.6667	1.683	1.21	-6.776	1.683	2.893	-5.093	2.893	-5.093
	U3L3	10	5.902	-11.072	-11.132	5.902	-5.17	-5.23	5.902	-5.23
	U4L4	6.6667	1.683	-6.777	1.216	1.683	-5.094	2.899	2.899	-5.094
	U5L5	3.3333	0.275	0	0	0.275	0.275	0.275	0.275	0
Diagonals/Web members	U1L2	7.4537	-3.149	-2.707	15.152	-3.149	-5.856	12.003	12.003	-5.858
	U2L3	9.4286	-3.979	-3.436	19.179	-3.979	-7.415	15.2	15.2	-7.415
	U4L3	9.4286	-3.979	19.094	-3.436	-3.979	15.115	-7.415	15.115	-7.415
	U5L4	7.4535	-3.149	15.153	-2.718	-3.149	12.004	-5.867	12.004	-5.867

2.9 Design of Truss Members

2.9.1 Design of Top Chord:

From the design chart for truss member

For top chord (length, L = 7.4535 feet), maximum compressive force = − 15.73 kip & maximum tensile force = + 18.099 kip.

Select an angle section L $3\times 2\frac{1}{2}\times\frac{1}{4}$; (cross sectional area, A = 1.31 inch^2 & minimum radius of gyration, r_z = 0.528 inch).

Check for Compression: Slenderness ratio, $\frac{KL}{r}=\frac{0.6\times 7.4535\times 12}{0.528}$ = 101.6386364

E = modulus of elasticity of steel = 29×10^6 psi = 29000 ksi

F_y = yield stress of the steel = 36 ksi (for A 36 steel)

$$C_c=\pi\sqrt{\frac{2E}{F_y}} = \pi\sqrt{\frac{2\times 29\times 10^3}{36}} = 126.0992836$$

F_a = allowable stress in compression (ksi)

$$\Rightarrow F_a=\frac{F_y\left[1-\frac{1}{2}\left(\frac{KL/r}{C_c}\right)^2\right]}{\frac{5}{3}+\frac{3}{8}\left(\frac{KL/r}{C_c}\right)-\frac{1}{8}\left(\frac{KL/r}{C_c}\right)^3} \quad \text{if } \frac{KL}{r}\le C_c$$

$$\Rightarrow F_a=\frac{36\times\left[1-\frac{1}{2}\left(\frac{101.6386364}{126.0992836}\right)^2\right]}{\frac{5}{3}+\frac{3}{8}\left(\frac{101.6386364}{126.0992836}\right)-\frac{1}{8}\left(\frac{101.6386364}{126.0992836}\right)^3} \quad \text{if } \frac{KL}{r}\le C_c$$

$$\Rightarrow F_a = 12.769265 \text{ ksi}$$

Allowable force in compression, $P_a = F_a \times A$ = 12.769265 ksi × 1.31 inch^2

= 16.72773715 kip. (which is greater than design compressive force 15.73 kip).

The L 3× $2\frac{1}{2}\times\frac{1}{4}$ section is OK for compressive force.

Check for Compression: Slenderness ratio, $\frac{KL}{r}=\frac{0.6\times7.4535\times12}{0.528}$ = 101.6386364 (which is less than 300).

F_t = allowable stress in tension (ksi)

$\Rightarrow F_t = 0.6F_y$ = 0.6 × 36 ksi = 21.6 ksi

Allowable force in tension, $P_t = F_t$ × A = 21.6 ksi × 1.31 inch^2 = 28.296 kip. (which is greater than design tensile force 18.099 kip). The L 3× $2\frac{1}{2}\times\frac{1}{4}$ section is OK for tensile force.

Top chord	L $3\frac{1}{2}\times3\times\frac{1}{4}$

2.9.2 Design of Bottom Chord:

From the design chart for truss member

For bottom chord (length, L = 6.6667 feet), maximum compressive force = − 32.667 kip & maximum tensile force = + 18.47 kip.

Select an angle section L 4×3× $\frac{5}{16}$; (cross sectional area, A = 2.09 inch^2 & minimum radius of gyration, r_z = 0.647 inch).

Check for Compression: Slenderness ratio, $\frac{KL}{r}=\frac{0.6\times6.6667\times12}{0.528}$ = 74.1885626

E = modulus of elasticity of steel = 29×10^6 psi = 29000 ksi

F_y = yield stress of the steel = 36 ksi (for A 36 steel)

$C_c=\pi\sqrt{\frac{2E}{F_y}} = \pi\sqrt{\frac{2\times29\times10^3}{36}}$ = 126.0992836

F_a = allowable stress in compression (ksi)

$$\Rightarrow F_a = \frac{F_y\left[1-\frac{1}{2}\left(\frac{KL/r}{C_c}\right)^2\right]}{\frac{5}{3}+\frac{3}{8}\left(\frac{KL/r}{C_c}\right)-\frac{1}{8}\left(\frac{KL/r}{C_c}\right)^3} \quad \text{if } \frac{KL}{r} \le C_c$$

$$\Rightarrow F_a = \frac{36\times\left[1-\frac{1}{2}\left(\frac{74.1885626}{126.0992836}\right)^2\right]}{\frac{5}{3}+\frac{3}{8}\left(\frac{74.1885626}{126.0992836}\right)-\frac{1}{8}\left(\frac{74.1885626}{126.0992836}\right)^3} \quad \text{if } \frac{KL}{r} \le C_c$$

$\Rightarrow F_a$ = 15.988919 ksi

Allowable force in compression, $P_a = F_a \times A$ = 15.988919 ksi × 2.09 inch^2 = 33.41684071 kip. (which is greater than design compressive force 32.667 kip). The L $4\times3\times\frac{5}{16}$ section is OK for compressive force.

Check for Compression: Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times6.6667\times12}{0.528}$ = 74.1885626 (which is less than 300).

F_t = allowable stress in tension (ksi)

$\Rightarrow F_t = 0.6F_y$ = 0.6 × 36 ksi = 21.6 ksi

Allowable force in tension, $P_t = F_t \times A$ = 21.6 ksi × 2.09 inch^2 = 45.144 kip. (which is greater than design tensile force 18.47 kip). The $L\ 4\times3\times\frac{5}{16}$ section is OK for tensile force.

Bottom chord	$L\ 4\times3\times\frac{5}{16}$

2.9.3 Design of Verticals:

From the design chart for truss member

For verticals U_3L_3 (length, L = 10 feet), maximum compressive force = − 5.23 kip & maximum tensile force = + 5.902 kip.

Select an angle section L $2\frac{1}{2}\times 2\frac{1}{2}\times\frac{3}{16}$; (cross sectional area, A = 0.902 inch^2 & minimum radius of gyration, r_z = 0.495 inch).

Check for Compression: Slenderness ratio, $\frac{KL}{r}=\frac{0.6\times 10\times 12}{0.495}$ = 145.4545455

E = modulus of elasticity of steel = 29×10^6 psi = 29000 ksi

F_y = yield stress of the steel = 36 ksi (for A 36 steel)

$$C_c=\pi\sqrt{\frac{2E}{F_y}} = \pi\sqrt{\frac{2\times 29\times 10^3}{36}} = 126.0992836$$

F_a = allowable stress in compression (ksi)

$$\Rightarrow F_a=\frac{12\pi^2 E}{23\left(\frac{KL}{r}\right)^2}=\frac{149000}{\left(\frac{KL}{r}\right)^2} \qquad \text{if } \frac{KL}{r}\geq C_c$$

$$\Rightarrow F_a=\frac{12\pi^2 E}{23\left(\frac{KL}{r}\right)^2}=\frac{149000}{(145.4545455)^2} \qquad \text{if } \frac{KL}{r}\geq C_c$$

$$\Rightarrow F_a = 7.042578121 \text{ ksi}$$

Allowable force in compression, P_a = F_a × A = 7.042578121 ksi × 0.902 inch^2 = 6.352405465 kip. (which is greater than design compressive force 5.23 kip). The L $3\times 2\frac{1}{2}\times\frac{3}{16}$ section is OK for compressive force.

Check for Compression: Slenderness ratio, $\frac{KL}{r}=\frac{0.6\times 10\times 12}{0.495}$ = 145.4545455 (which is less than 300).

F_t = allowable stress in tension (ksi)

$\Rightarrow F_t = 0.6F_y$ = 0.6 × 36 ksi = 21.6 ksi

Allowable force in tension, $P_t = F_t \times A = 21.6$ ksi × 0.905 inch^2 = 19.548 kip. (which is greater than design tensile force 5.902 kip). The L $3\times 2\frac{1}{2}\times\frac{3}{16}$ section is OK for tensile force.

Verticals	L $2\frac{1}{2}\times 2\frac{1}{2}\times\frac{3}{16}$

2.9.4 Design of Diagonals / Web Members:

From the design chart for truss member

For diagonals U_2L_3/U_4L_3 (length, L = 9.4286 feet), maximum compressive force = − 7.415 kip & maximum tensile force = + 15.2 kip.

Select an angle section L $3\times 2\frac{1}{2}\times\frac{3}{16}$; (cross sectional area, A = 0.996 inch^2 & minimum radius of gyration, r_z = 0.533 inch).

<u>Check for Compression:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times 9.4286\times 12}{0.533}$ = 127.3657036

E = modulus of elasticity of steel = 29×10^6 psi = 29000 ksi

F_y = yield stress of the steel = 36 ksi (for A 36 steel)

$$C_c = \pi\sqrt{\frac{2E}{F_y}} = \pi\sqrt{\frac{2\times 29\times 10^3}{36}} = 126.0992836$$

F_a = allowable stress in compression (ksi)

$$\Rightarrow F_a = \frac{12\pi^2 E}{23\left(\frac{KL}{r}\right)^2} = \frac{149000}{\left(\frac{KL}{r}\right)^2} \qquad \text{if } \frac{KL}{r} \geq C_c$$

$$\Rightarrow F_a = \frac{12\pi^2 E}{23\left(\frac{KL}{r}\right)^2} = \frac{149000}{(127.3657036)^2} \qquad \text{if } \frac{KL}{r} \geq C_c$$

$\Rightarrow F_a$ = 9.185044616 ksi

Allowable force in compression, $P_a = F_a \times A$ = 9.185044616 ksi × 0.996 inch^2 = 9.148304438 kip. (which is greater than design compressive force 7.415 kip). The L $3\times 2\frac{1}{2}\times\frac{3}{16}$ section is OK for compressive force.

<u>Check for Compression:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times 9.4286\times 12}{0.533}$ = 127.3657036 (which is less than 300).

F_t = allowable stress in tension (ksi)

$\Rightarrow F_t = 0.6F_y$ = 0.6 × 36 ksi = 21.6 ksi

Allowable force in tension, $P_t = F_t \times A$ = 21.6 ksi × 0.996 inch^2 = 21.5136 kip. (which is greater than design tensile force 15.2 kip). The L $3\times 2\frac{1}{2}\times\frac{3}{16}$ section is OK for tensile force.

Check whether the selected L $3\times 2\frac{1}{2}\times\frac{3}{16}$ section for diagonal member U_2L_3 & U_4L_3 is OK or not for the other diagonal members U_1L_2 & U_5L_4

For diagonals U_1L_2 / U_5L_4 (length, L = 7.4537 feet), maximum compressive force = − 5.867 kip & maximum tensile force = + 12.004 kip.

Select an angle section L $3\times 2\frac{1}{2}\times\frac{3}{16}$; (cross sectional area, A = 0.996 inch^2 & minimum radius of gyration, r_z = 0.533 inch).

<u>Check for Compression:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times 7.4537\times 12}{0.533}$ = 100.6878799

E = modulus of elasticity of steel = 29×10^6 psi = 29000 ksi

F_y = yield stress of the steel = 36 ksi (for A 36 steel)

$$C_c = \pi\sqrt{\frac{2E}{F_y}} = \pi\sqrt{\frac{2\times 29\times 10^3}{36}} = 126.0992836$$

F_a = allowable stress in compression (ksi)

$$\Rightarrow F_a = \frac{F_y\left[1-\frac{1}{2}\left(\frac{KL/r}{C_c}\right)^2\right]}{\frac{5}{3}+\frac{3}{8}\left(\frac{KL/r}{C_c}\right)-\frac{1}{8}\left(\frac{KL/r}{C_c}\right)^3} \quad \text{if } \frac{KL}{r} \le C_c$$

$$\Rightarrow F_a = \frac{36\times\left[1-\frac{1}{2}\left(\frac{127.3657036}{126.0992836}\right)^2\right]}{\frac{5}{3}+\frac{3}{8}\left(\frac{127.3657036}{126.0992836}\right)-\frac{1}{8}\left(\frac{127.3657036}{126.0992836}\right)^3} \quad \text{if } \frac{KL}{r} \le C_c$$

$\Rightarrow F_a$ = 9.2018618 ksi

Allowable force in compression, $P_a = F_a \times A$ = 9.2018618 ksi × 0.996 inch^2 = 9.1650543 kip. (which is greater than design compressive force 5.867 kip). The L $3\times 2\frac{1}{2}\times\frac{3}{16}$ section is OK for compressive force.

<u>Check for Compression:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times 9.4286\times 12}{0.533}$ = 127.3657036 (which is less than 300).

F_t = allowable stress in tension (ksi)

$\Rightarrow F_t = 0.6F_y$ = 0.6 × 36 ksi = 21.6 ksi

Allowable force in tension, $P_t = F_t \times A$ = 21.6 ksi × 0.996 inch^2 = 21.5136 kip. (which is greater than design tensile force 12.004 kip). The L $3\times 2\frac{1}{2}\times\frac{3}{16}$ section is OK for tensile force.

Diagonals	L $3\times 2\frac{1}{2}\times\frac{3}{16}$

Table 2.3: Design summary for truss members

Member type	Design section
Top chord	$L\ 3\frac{1}{2}\times 3\times \frac{1}{4}$
Bottom chord	$L\ 4\times 3\times \frac{5}{16}$
Verticals	$L\ 2\frac{1}{2}\times 2\frac{1}{2}\times \frac{3}{16}$
Web member / Diagonals	$L\ 3\times 2\frac{1}{2}\times \frac{3}{16}$

2.10 Design of Bracing Systems:

2.10.1 Vertical Bracing:

The members of the vertical bracing will be tied to each other at their crossing point. Therefore, half of their length will be considered in determining slenderness ratio $\left(\frac{KL}{r}\right)$. We will assume that, effective length factor, K = 0.70. The length of member of the vertical bracing (L) = $\frac{\sqrt{10^2+25^2}}{2}$ feet = 13.46291202 feet = 13.46291202 × 12 inch.

$$\Rightarrow \frac{KL}{r_{minimum}} < 400$$

$$\Rightarrow \frac{0.7\times 13.46291202\times 12}{r_{minimum}} < 400$$

$$\Rightarrow r_{minimum} > \frac{0.7\times 13.46291202\times 12}{400}$$

$$\Rightarrow r_{minimum} > 0.2827211524 \text{ inch}$$

From AISC chart for angles we select $L\ 1\frac{1}{2}\times 1\frac{1}{2}\times \frac{1}{4}$ for which r = $r_z = r_{minimum}$ = 0.292 inch.

2.10.2 Top Chord Bracing:

Similar to the vertical bracing, the members of the top chord bracing will also be tied to each other at their crossing point. Therefore, half of their length will be considered in determining slenderness ratio $\left(\frac{KL}{r}\right)$. We will assume that, effective length factor, K = 0.70. The length of member of the top chord bracing (L) = $\frac{\sqrt{(2\times 7.453)^2+25^2}}{2}$ feet = 14.55325424 feet = 14.55325424 × 12 inch.

$$\Rightarrow \frac{KL}{r_{minimum}} < 400$$

$$\Rightarrow \frac{0.7\times 14.55325424\times 12}{r_{minimum}} < 400$$

$$\Rightarrow r_{minimum} > \frac{0.7\times 14.55325424\times 12}{400}$$

$$\Rightarrow r_{minimum} > 0.3056183391 \text{ inch}$$

From AISC chart for angles we select $L\ 1\frac{3}{4}\times 1\frac{3}{4}\times\frac{1}{4}$ for which r = $r_z = r_{minimum}$ = 0.341 inch.

2.10.3 Bottom Chord Bracing:

If we consider the length of the struts equal to the bay distance, the $\left(\frac{KL}{r}\right)$ratio criterion will result too large section. To economize our design, we will use a lateral tie at the midspan of the struts very similar to the sagrods used for purlins (see figure below). For these lateral ties, we use steel rods same as the rods. The presence of the ties at the midspan will reduce the unsupported length of the struts by 50%. We will assume that, effective length factor, K = 0.70. The length of member of the bottom chord bracing (L) = $\frac{25}{2}$ feet = 12.5 feet = 12.5 × 12 inch.

$$\Rightarrow \frac{KL}{r_{minimum}} < 300$$

$$\Rightarrow \frac{0.7\times 12.5\times 12}{r_{minimum}} < 300$$

$$\Rightarrow r_{minimum} > \frac{0.7\times 12.5\times 12}{300}$$

$$\Rightarrow r_{minimum} > 0.350 \text{ inch}$$

From AISC chart for angles we select $L\ 2\times2\times\frac{5}{16}$ for which r = $r_z = r_{minimum}$ = 0.390 inch.

Ties for the bottom chord struts are arbitrarily chosen to be round steel bars of $\frac{1}{2}$ inch diameter. These will be connected to the bottom chord struts using standard $\frac{1}{2}$ inch nuts in a manner similar to the sagrods.

Table 2.4: Design summary for bracing systems

Bracing type	Design section
Vertical bracing	$L\ 1\frac{1}{2}\times1\frac{1}{2}\times\frac{1}{4}$
Top chord bracing	$L\ 1\frac{3}{4}\times1\frac{3}{4}\times\frac{1}{4}$
Bottom chord strut	$L\ 2\times2\times\frac{5}{16}$

2.11 Design of Truss Joints (Welded Connections):

There are two types of joints in the truss – joints where all members ends (such as L0, U3 & L6) and joints where there are one continuous members (such as L1, L2, L3, L4, L5, U1, U2, U4 & U5).

Gusset plate thickness (for a joint) = maximum thickness of the angle sections meeting at that joint + 1/8 inch.

Weld length for a member =

$$\frac{\text{maximum tensile or compressive force at that member (kip)}}{\text{allowable weld shear (ksi)}\times\text{effective throat size (inch)}} = ?\ \text{inch}$$

Weld Design of Joint L0:

Here, two members L0U1 ($L\ 3\frac{1}{2}\times3\times\frac{1}{4}$) & L0L1 ($L\ 4\times3\times\frac{5}{16}$) meets.

Gusset plate thickness (for joint L0) = maximum thickness of the angle sections ($\frac{5}{16}$ inch) meeting at that joint + $\frac{1}{8}$ inch = $\frac{7}{16}$ inch.

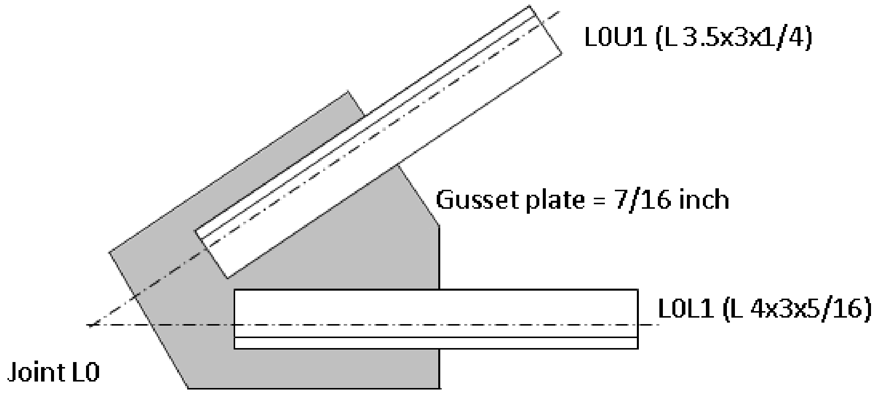


Figure 2.16: Joint L_0 of roof truss

Weld for L0L1:

Consider, L0L1 (L $4\times3\times\frac{5}{16}$) & gusset plate ($\frac{7}{16}$ inch)

$t_{max} = \frac{7}{16}$ inch and $t_{min} = \frac{5}{16}$ inch

Maximum thickness of the part being connected, $t_{max} = \frac{7}{16}$ inch. So, Minimum fillet weld size, $s_{min} = \frac{3}{16}$ inch (from table 1, chapter: welded connections)

Minimum thickness of the part being connected, $t_{min} = \frac{5}{16}$ inch. So, Maximum fillet weld size, $s_{max} = \left(\frac{5}{16} - \frac{1}{16}\right)$ inch $= \frac{4}{16}$ inch You can choose either $\frac{3}{16}$ inch or $\frac{4}{16}$ inch. Choose $\frac{3}{16}$ inch fillet weld.

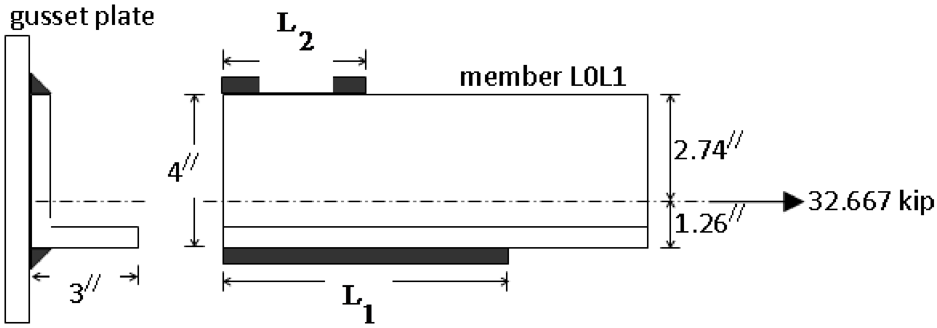


Figure 2.17: Weld design for L_0L_1

Electrode: E60XX (i.e., electrode material tensile strength (F_{EXX}) = 60 ksi).

Allowable shear in weld (F_v) = 0.3 × F_{E60XX} = 0.3 × 60 ksi = 18 ksi.

Fillet weld size chosen, s = $\frac{3}{16}$ inch

Effective throat size, (t_e) = s × cos 45^0 = $\frac{3}{16}$ × cos 45^0 inch.

Weld length required for member L0U1,

$$L_{L_1U_0} = \frac{\left|P_{maximum}\right|_{tensile\ or\ compressive}}{F_v \times t_e} = \frac{32.667\ kip}{18\ ksi \times \frac{3}{16} \cos 45^0\ inch}$$

$\Rightarrow L_{L_1U_0}$ = 13.68833 inch

$\Rightarrow L_1 + L_2 = L_{L_1U_0}$ = 13.68833 inch

Taking moment about L_2,

$\Rightarrow$ ($L_1 \times t_e \times F_v$) × (4$^{//}$) = (32.667 kip) × (2.74$^{//}$)

$\Rightarrow$ ($L_1 \times \frac{3}{16}$ cos 45^0 inch ×18 ksi) × (4$^{//}$) = (32.667 kip) × (2.74$^{//}$)

$\Rightarrow L_1$ = 9.376506 inch ≈ 9.50 inch

Taking moment about L_1,

$\Rightarrow$ ($L_2 \times t_e \times F_v$) × (4$^{//}$) = (18.099 kip) × (1.26$^{//}$)

$\Rightarrow$ ($L_2 \times \frac{3}{16}$ cos 45^0 inch ×18 ksi) × (4$^{//}$) = (32.667 kip) × (1.26$^{//}$)

$\Rightarrow L_2$ = 4.311824 inch ≈ 4.50 inch

Minimum weld length, $L_{minimum}$ = 4 s = 4 × $\frac{3}{16}$ = 0.75 inch

Both L_1 & L_2 > $L_{minimum}$; OK

Alternatively, $L_1 + L_2$ = 13.68833 inch & $\frac{L_1}{L_2} = \frac{2.74\,\text{inch}}{1.26\,\text{inch}}$; from which, L_1 = 9.376506 inch & L_2 = 4.311824 inch.

Weld for L0U1:

Consider, L0U1 ($L\ 3\frac{1}{2}\times 3\times \frac{1}{4}$) & gusset plate ($\frac{7}{16}$ inch)

$t_{max} = \frac{7}{16}\,\text{inch}$ and $t_{min} = \frac{1}{4}\,\text{inch}$

Maximum thickness of the part being connected, $t_{max} = \frac{7}{16}\,\text{inch}$. So, Minimum fillet weld size, $s_{min} = \frac{3}{16}\,\text{inch}$ (from table 1, chapter: welded connections)

Minimum thickness of the part being connected, $t_{min} = \frac{1}{4}\,\text{inch}$. So, Maximum fillet weld size, $s_{max} = \left(\frac{1}{4} - \frac{1}{16}\right)\text{inch} = \frac{3}{16}\,\text{inch}$ (from table 2, chapter: welded connections)

Use $\frac{3}{16}$ inch fillet weld.

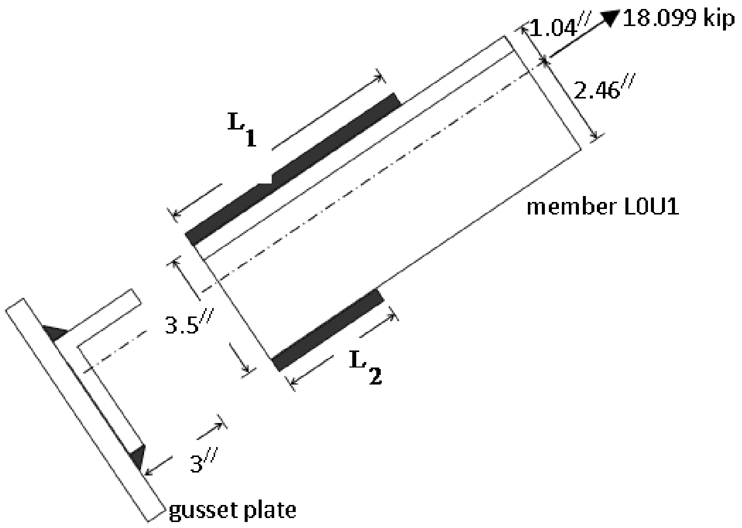


Figure 2.18: Weld design for L_0U_1

Electrode: E60XX (i.e., electrode material tensile strength (F_{EXX}) = 60 ksi).

Allowable shear in weld (F_v) = 0.3 × F_{E60XX} = 0.3 × 60 ksi = 18 ksi.

Fillet weld size chosen, s = $\frac{3}{16}$ inch

Effective throat size, (t_e) = s × cos 45^0 = $\frac{3}{16}$ × cos 45^0 inch.

Weld length required for member L0U1,

$$L_{L_0U_1} = \frac{\left|P_{maximum}\right|_{tensile\ or\ compressive}}{F_v \times t_e} = \frac{18.099\ kip}{18\ ksi \times \frac{3}{16}\cos 45^0\ inch}$$

$\Rightarrow L_{L_0U_1}$ = 7.583955 inch

$\Rightarrow L_1 + L_2 = L_{L_0U_1}$ = 7.583955 inch

Taking moment about L_2,

$\Rightarrow (L_1 \times t_e \times F_v) \times (3.5'') = (18.099\ kip) \times (2.46'')$

$\Rightarrow (L_1 \times \frac{3}{16}\cos 45^0\ inch \times 18\ ksi) \times (3.5'') = (18.099\ kip) \times (2.46'')$

$\Rightarrow L_1$ = 5.330437 inch ≈ 5.50 inch

Taking moment about L_1,

$\Rightarrow (L_2 \times t_e \times F_v) \times (3.5'') = (18.099\ kip) \times (1.04'')$

$\Rightarrow (L_2 \times \frac{3}{16}\cos 45^0\ inch \times 18\ ksi) \times (3.5'') = (18.099\ kip) \times (1.04'')$

$\Rightarrow L_2$ = 2.253518 inch ≈ 2.50 inch

Minimum weld length, $L_{minimum}$ = 4 s = 4 × $\frac{3}{16}$ = 0.75 inch

Both L_1 & $L_2 > L_{minimum}$; OK

Alternatively, $L_1 + L_2 = 7.583955$ inch & $\frac{L_1}{L_2} = \frac{2.46\,\text{inch}}{1.04\,\text{inch}}$; from which, $L_1 = 5.330437$ inch & $L_2 = 2.253518$ inch.

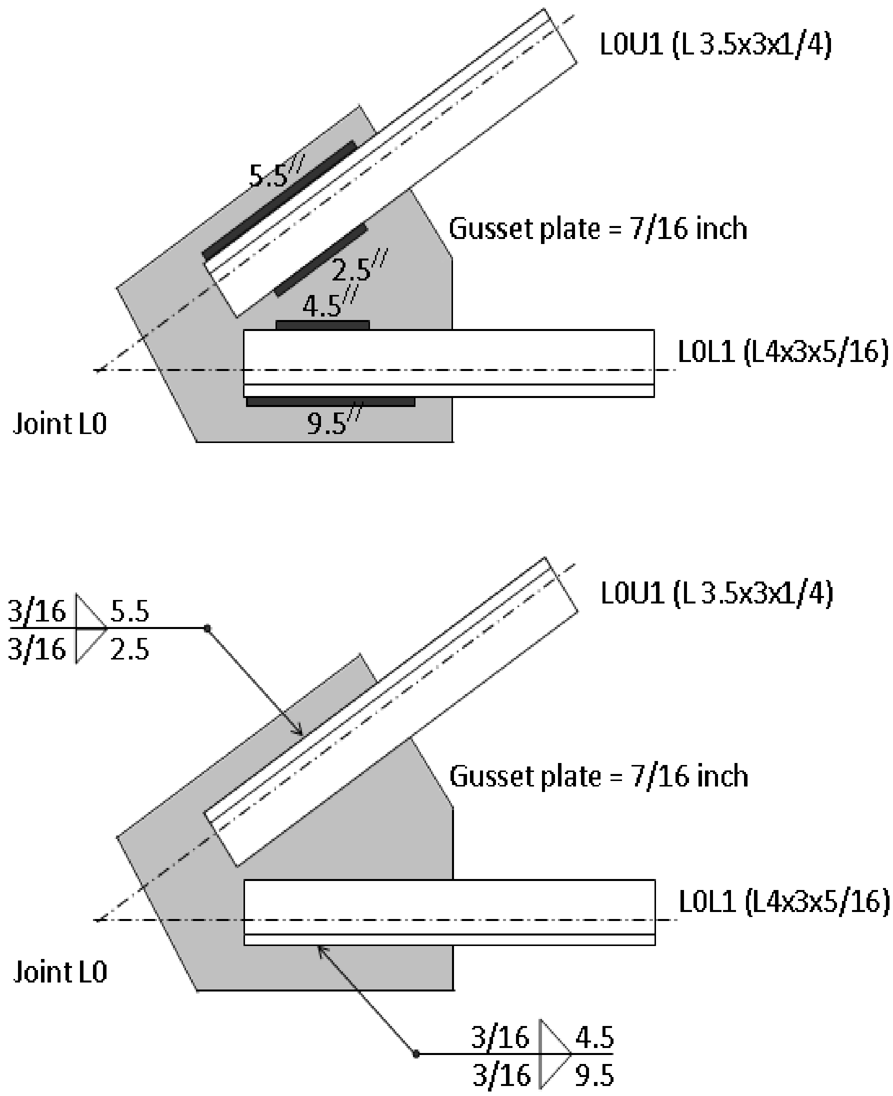


Figure 2.19: Weld design of joint L_0

Weld Design of Joint U1:

Here, two members L0U1 (L $3.5\times3\times\frac{1}{4}$), U1L1 (2.5×2.5×3/16) & U1L2 (3×2.5×3/16) meets. Gusset plate thickness (for joint L0) = maximum thickness of the angle sections ($\frac{1}{4}$ inch) meeting at that joint $+\frac{1}{8}$ inch = $\frac{6}{16}$ inches

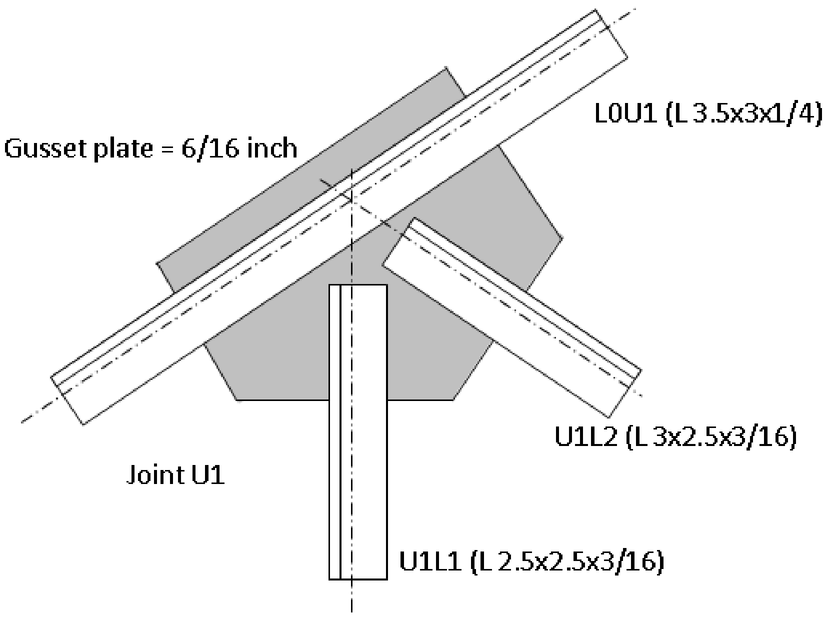


Figure 2.20: Weld design of joint U_1

<u>Weld for L0U1U2:</u>

Consider, L0U1U2 ($L\ 3\frac{1}{2}\times 3\times \frac{1}{4}$) & gusset plate ($\frac{6}{16}$ inch)

$t_{max} = \frac{6}{16}$ inch and $t_{min} = \frac{1}{4}$ inch

Maximum thickness of the part being connected, $t_{max} = \frac{6}{16}$ inch. So, Minimum fillet weld size, $s_{min} = \frac{3}{16}$ inch (from table 1, chapter: welded connections)

Minimum thickness of the part being connected, $t_{min} = \frac{1}{4}$ inch. So, Maximum fillet weld size, $s_{max} = \left(\frac{1}{4} - \frac{1}{16}\right)$ inch $= \frac{3}{16}$ inch (from table 2, chapter: welded connections)

Use $\frac{3}{16}$ inch fillet weld.

We are designing top chord as a continuous member. The length of weld required to hold the bottom chord with the gusset plate at joint L1 depends on the resultant (absolute value) of the axial forces in members L0U1 and U1U2. We have to consider three possible equilibrium conditions to determine the resultant force for design. These three equilibrium conditions are (1) Dead load only, (2) DL + Wind (L→R) and (3) DL + Wind (R→L). The process of finding the resultant for design is shown in tabular form below –

Equilibrium condition	L0U1 (member force, kip) $F_{L_0U_1}$	U1U2 (member force, kip) $F_{U_1U_2}$	Magnitude of the resultant, kip $\left\|F_{L_0U_1} - F_{U_1U_2}\right\|$
DL	− 15.73	− 12.587	$\lvert -15.73 - (-12.587) \rvert$ = 3.143
DL + W (L→R)	+ 0.265	+ 5.048	$\lvert +0.265 - (+5.048) \rvert$ = 4.783
DL + W (R→L)	+ 18.099	+ 12.115	$\lvert +18.099 - (+12.115) \rvert$ = 5.984

Observing the last column, we find that the design force is 5.984 kip. This force will be used to determine the weld length required to hold the top chord member with the gusset plate at joint U1.

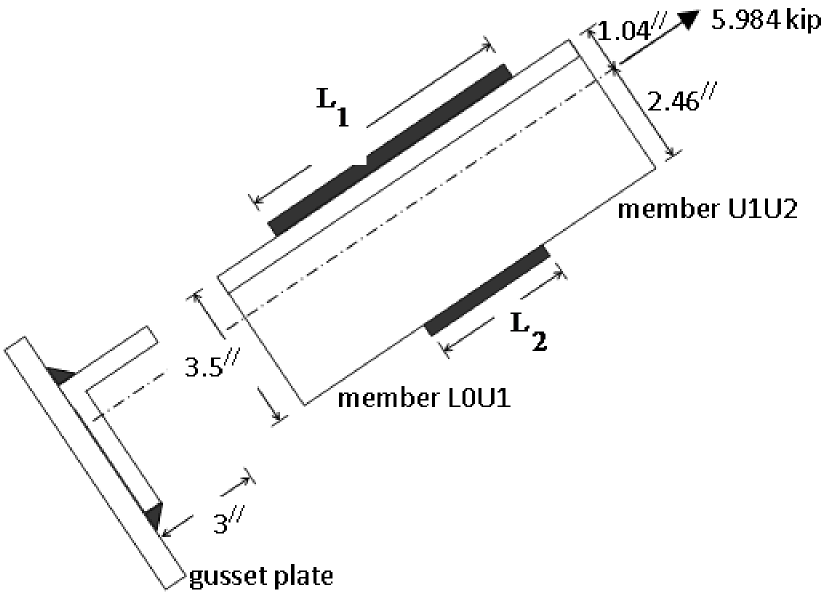


Figure 2.21: Weld design of member L_0U_1 and U_1U_2

Electrode: E60XX (i.e., electrode material tensile strength (F_{EXX}) = 60 ksi).

Allowable shear in weld (F_v) = 0.3 × F_{E60XX} = 0.3 × 60 ksi = 18 ksi.

Fillet weld size chosen, s = $\frac{3}{16}$ inch

Effective throat size, (t_e) = s × cos 45^0 = $\frac{3}{16}$ × cos 45^0 inch.

Weld length required for member L0U1U2,

$$L_{L_0U_1U_2} = \frac{\left|P_{maximum}\right|_{tensile\,or\,compressive}}{F_v \times t_e} = \frac{5.984\,kip}{18\,ksi \times \frac{3}{16}\cos 45^0\ inch}$$

$\Rightarrow L_{L_0U_1U_2}$ = 2.507453 inch

$\Rightarrow L_1 + L_2 = L_{L_0U_1U_2}$ = 2.507453 inch

Taking moment about L_2,

$\Rightarrow (L_1 \times t_e \times F_v) \times (3.5'') = (5.984\ kip) \times (2.46'')$

$\Rightarrow (L_1 \times \frac{3}{16}\cos 45^0\ inch \times 18\ ksi) \times (3.5'') = (5.984\ kip) \times (2.46'')$

$\Rightarrow L_1$ = 1.762381 inch ≈ 2 inch

Taking moment about L_1,

$\Rightarrow (L_2 \times t_e \times F_v) \times (3.5'') = (5.984\ kip) \times (1.04'')$

$\Rightarrow (L_2 \times \frac{3}{16}\cos 45^0\ inch \times 18\ ksi) \times (3.5'') = (5.984\ kip) \times (1.04'')$

$\Rightarrow L_2$ = 0.745071 inch ≈ 1 inch

Minimum weld length, $L_{minimum} = 4\ s = 4 \times \frac{3}{16}$ = 0.75 inch

Both L_1 & $L_2 > L_{minimum}$; OK

Alternatively, $L_1 + L_2$ = 2.507453 inch & $\frac{L_1}{L_2} = \frac{2.46\,inch}{1.04\,inch}$; from which, L_1 = 1.762381 inch & L_2 = 0.745071 inch.

<u>Weld for L1U1:</u>

Consider, L1U1 ($L\ 2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{16}$) & gusset plate ($\frac{6}{16}$ inch)

$t_{max} = \frac{6}{16}$ inch and $t_{min} = \frac{3}{16}$ inch

Maximum thickness of the part being connected, $t_{max} = \frac{6}{16}$ inch. So, Minimum fillet weld size, $s_{min} = \frac{3}{16}$ inch (from table 1, chapter: welded connections)

Minimum thickness of the part being connected, $t_{min} = \frac{3}{16}$ inch. So, Maximum fillet weld size, $s_{max} = \frac{3}{16}$ inch (from table 2, chapter: welded connections)

Use $\frac{3}{16}$ inch fillet weld.

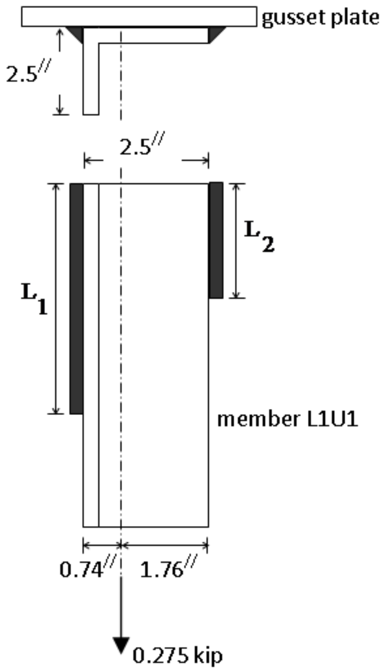


Figure 2.22: Weld design of member L_1U_1

Electrode: E60XX (i.e., electrode material tensile strength (F_{EXX}) = 60 ksi).

Allowable shear in weld (F_v) = 0.3 × F_{E60XX} = 0.3 × 60 ksi = 18 ksi.

Fillet weld size chosen, s = $\frac{3}{16}$ inch

Effective throat size, (t_e) = s × cos 45^0 = $\frac{3}{16}$ × cos 45^0 inch.

Weld length required for member L1U1,

$$L_{L_1U_1} = \frac{\left|P_{maximum}\right|_{tensile\,or\,compressive}}{F_v \times t_e} = \frac{0.275\,kip}{18\,ksi \times \frac{3}{16}\cos 45^0\ inch}$$

$\Rightarrow L_{L_1U_1} = 0.1152322$ inch

$\Rightarrow L_1 + L_2 = L_{L_1U_1} = 0.1152322$ inch

Taking moment about L_2,

$\Rightarrow (L_1 \times t_e \times F_v) \times (2.5^{\prime\prime}) = (0.275 \text{ kip}) \times (1.76^{\prime\prime})$

$\Rightarrow (L_1 \times \frac{3}{16} \cos 45^0 \text{ inch} \times 18 \text{ ksi}) \times (2.5^{\prime\prime}) = (0.275 \text{ kip}) \times (1.76^{\prime\prime})$

$\Rightarrow L_1 = 0.081123 \text{ inch} \approx 0.50$ inch

Taking moment about L_1,

$\Rightarrow (L_2 \times t_e \times F_v) \times (2.5^{\prime\prime}) = (0.275 \text{ kip}) \times (0.74^{\prime\prime})$

$\Rightarrow (L_2 \times \frac{3}{16} \cos 45^0 \text{ inch} \times 18 \text{ ksi}) \times (2.5^{\prime\prime}) = (0.275 \text{ kip}) \times (0.74^{\prime\prime})$

$\Rightarrow L_2 = 0.034108 \text{ inch} \approx 0.50$ inch

Minimum weld length, $L_{minimum} = 4 \text{ s} = 4 \times \frac{3}{16} = 0.75$ inch

Both L_1 & $L_2 > L_{minimum}$; OK

Alternatively, $L_1 + L_2 = 0.1152322$ inch & $\frac{L_1}{L_2} = \frac{2.46 \text{ inch}}{1.04 \text{ inch}}$; from which, $L_1 = 0.081123$ inch & $L_2 = 0.034108$ inch.

Weld for U1L2:

Consider, U1L2 ($L\ 3 \times 2\frac{1}{2} \times \frac{3}{16}$) & gusset plate ($\frac{6}{16}$ inch)

$t_{max} = \frac{6}{16}$ inch and $t_{min} = \frac{3}{16}$ inch

Maximum thickness of the part being connected, $t_{max} = \frac{6}{16}$ inch. So, Minimum fillet weld size, $s_{min} = \frac{3}{16}$ inch

Minimum thickness of the part being connected, $t_{min} = \frac{3}{16}$ inch. So, Maximum fillet weld size, $s_{max} = \frac{3}{16}$ inch

Use $\frac{3}{16}$ inch fillet weld.

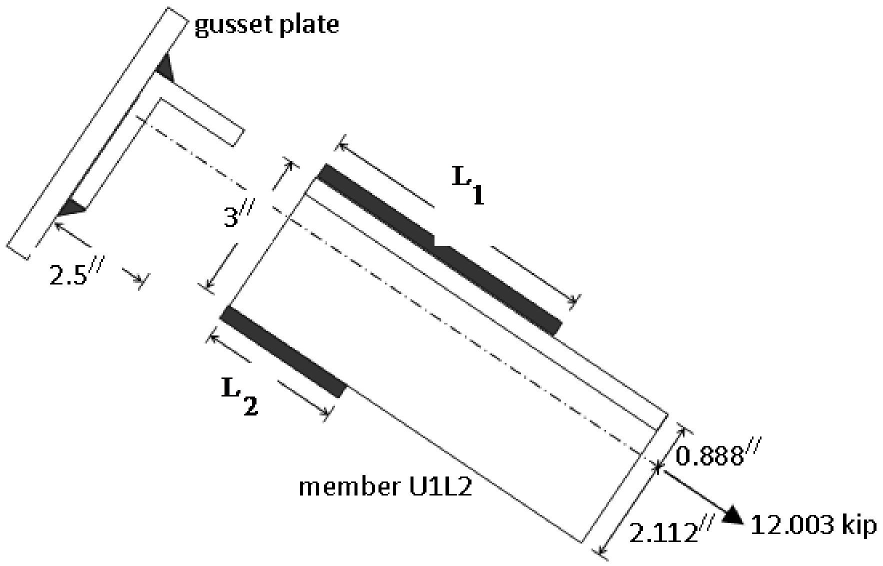


Figure 2.23: Weld design of member U_1L_2

Electrode: E60XX (i.e., electrode material tensile strength (F_{EXX}) = 60 ksi).
Allowable shear in weld (F_v) = 0.3 × F_{E60XX} = 0.3 × 60 ksi = 18 ksi.

Fillet weld size chosen, s = $\frac{3}{16}$ inch

Effective throat size, (t_e) = s × cos 45^0 = $\frac{3}{16}$ × cos 45^0 inch.

Weld length required for member L1U2,

$$L_{L_1U_1} = \frac{\left|P_{maximum}\right|_{tensile\ or\ compressive}}{F_v \times t_e} = \frac{12.003\,kip}{18\,ksi \times \frac{3}{16}\cos 45^0\ inch}$$

$\Rightarrow L_{L_1U_1}$ = 5.029571 inch

$\Rightarrow L_1 + L_2 = L_{L_1U_1}$ = 5.029571 inch

Taking moment about L_2,

$\Rightarrow (L_1 \times t_e \times F_v) \times (3^{//}) = (12.003 \text{ kip}) \times (2.112^{//})$

$\Rightarrow (L_1 \times \frac{3}{16} \cos 45^0 \text{ inch} \times 18 \text{ ksi}) \times (3^{//}) = (12.003 \text{ kip}) \times (2.112^{//})$

$\Rightarrow L_1 = 3.540818 \text{ inch} \approx 4 \text{ inch}$

Taking moment about L_1,

$\Rightarrow (L_2 \times t_e \times F_v) \times (3^{//}) = (12.003 \text{ kip}) \times (0.888^{//})$

$\Rightarrow (L_2 \times \frac{3}{16} \cos 45^0 \text{ inch} \times 18 \text{ ksi}) \times (3^{//}) = (12.003 \text{ kip}) \times (0.888^{//})$

$\Rightarrow L_2 = 1.488753 \text{ inch} \approx 1.5 \text{ inch}$

Minimum weld length, $L_{minimum} = 4\text{ s} = 4 \times \frac{3}{16} = 0.75 \text{ inch}$

Both L_1 & $L_2 > L_{minimum}$; OK

Alternatively, $L_1 + L_2 = 5.029571$ inch& $\frac{L_1}{L_2} = \frac{2.112 \text{ inch}}{0.888 \text{ inch}}$; from which, $L_1 = 3.540818$ inch & $L_2 = 1.488753$ inch.

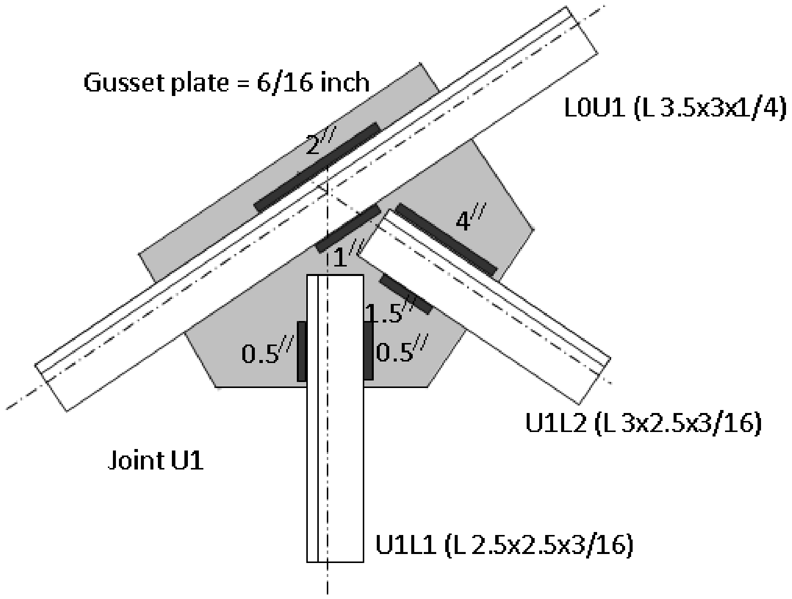


Figure 2.24(a): Weld design of joint U₁

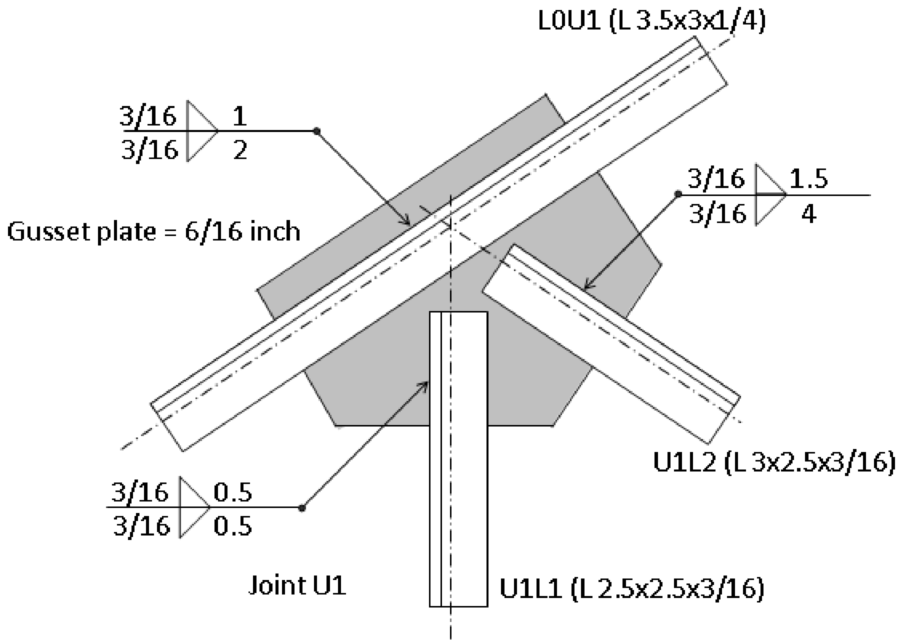


Figure 2.24(b): Weld design of joint U_1

2.12 Design of Anchorage and Support:

The truss is supported by reinforced concrete columns and footings, their reactions having been calculated earlier for point dead load and wind loads. The connections between the truss and support are designed in this section for the combined design loads.

Combination of Support Reactions from Dead Load and Wind Load:

The calculation for the design support reactions is carried out in the following tabular form.

Support	Support Reactions (kips)			Design Forces (kips)		
	Dead Load	Wind Load (→)	Wind Load (←)	Case1	Case2	Case3
L_0	14.73 ↑	40.50 ↓ 3.34 ←	44.02 ↓ 3.34 →	14.73 (C)	25.77 (T) 3.34 (S)	29.29 (T) 3.34 (S)
L_6	14.73 ↑	44.02 ↓	40.50 ↓	14.73 (C)	29.29 (T)	25.77 (T)

Since the truss is supported on base plates on concrete pedestals supported by masonry columns, the design in this study deals mainly with the connections between the truss and the columns. The column forces are nominal, therefore a 10”X20” masonry column is chosen. The maximum tensile stress on the column = 29.29/(10*20) = 0.146 ksi, which is within the allowable limit (Tensile strength 300 psi).

Assuming the base plate area = Ap and bearing pressure = 0.35 fc’ = 1.05 ksi

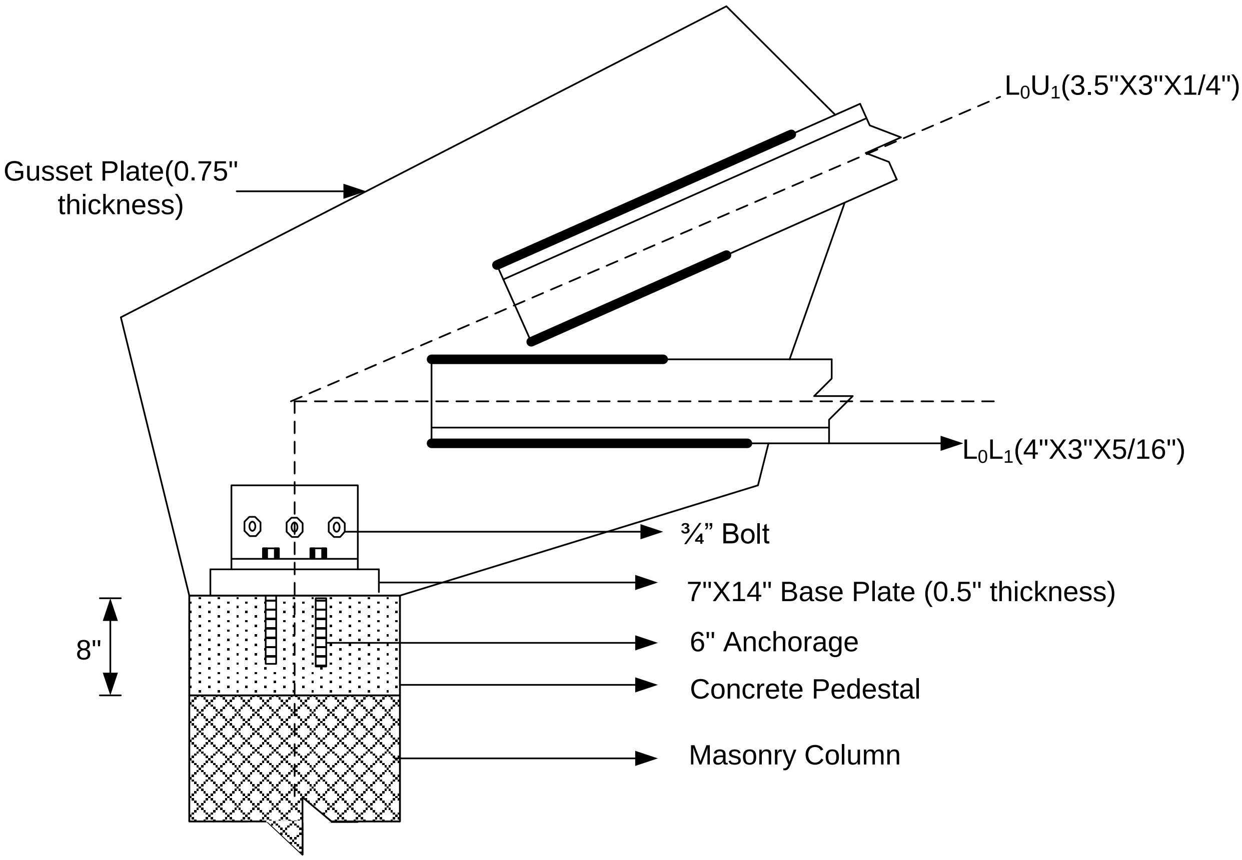
1.05Ap = 14.73

Ap = 14.73/1.05

$A_p = 14.03 \text{ in}^2$

Provide 7"X14" base plate (since the bottom cord members are 4" + 4" wide)

Since the free portion of the base plate is nominal, a thickness of 0.5" is more than adequate.



Hinge Support L_0

Figure 2.25: Design of anchorage at hinge support L_0

The base plate is supported on a 10"x20" concrete pedestal and connected to the column by four reinforcements to resist the entire tensile and shear force.

Allowable tensile stress = 0.5 fy = 20 ksi and allowable shear stress = 0.3 fy = 12 ksi

Required area (based on tensile force) = 29.29/(4*20) = 0.366 in^2

Required area (based on shear force) = 3.34/(4*12) = 0.07 in^2

Provide 4 #6 (i.e., ¾" diameter) anchor bolts (Area = 0.44 in^2 each).

Allowable tensile force per anchor = 0.44*20 = 8.8 kips

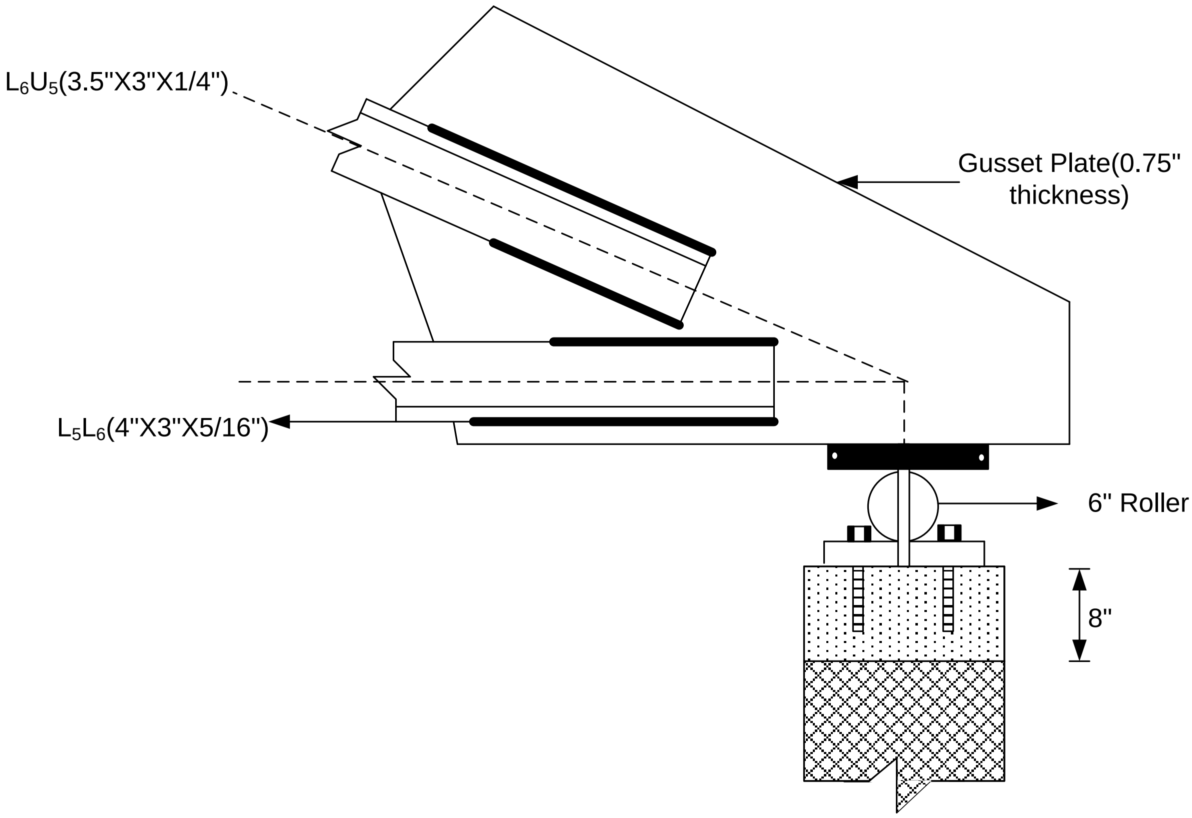
Allowable bond force per unit length = $35\sqrt{fc'} = 35\sqrt{3000}$ lb/in = 1.92 k/in

Development length = 8.8/1.92 = 4.59"

Provide anchorage of 6" for each bolt.

The base plate will be connected to the gusset plate by the section similar to the bottom cord (i.e., a 4X3X5/16 double angle section), also with ¾" diameter bolts to transfer the maximum support reaction (= 29.29 kips) by shear.

Required area = 29.29/12 = 2.44 in^2, i.e., provide 3-3/4" diameter bolts in double shear.



Roller Support L_6

Figure 2.26 Design of anchorage at roller support L_6

Part 3: Design of Steel Plate Girder

3.1 Introduction

When a member is required that is larger than that is available in rolled beams, it is necessary to build up a section which for the sake of economy has a general shape of an I-beam. This built up I-beam is called plate girder in which Section modulus is greater than any available rolled beam. The moment-resisting capacities of plate girders lie somewhere between those of deep standard rolled wide-flange shapes and those of trusses. Plate girders can be welded (Fig. 30(a) to 30(d)), riveted, or bolted (Fig. 30(e)). Riveted plate girders are practically obsolete. Very few bolted plate girders are designed nowadays. Therefore, in this manual design of welded plate girders has been covered. In this chapter, we consider large flexural members (girders) that are composed of plate elements in particular, with shapes built up from plates, however, both flanges and webs can be compact, noncompact, or slender. These built-up shapes usually are used when the bending moments are larger than standard hot-rolled shapes can resist, usually because of a large span. These girders are invariably very deep, resulting in noncompact or slender webs.

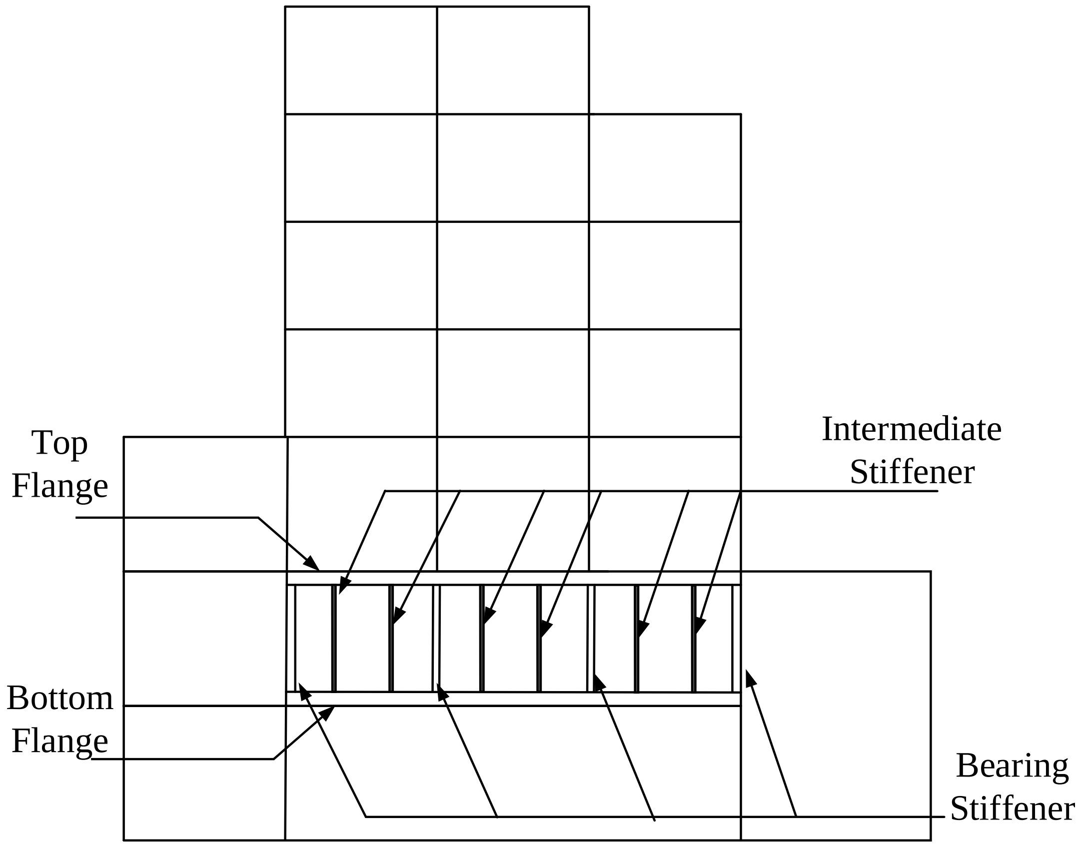


Figure 3.1: Plate girder in a multistory building

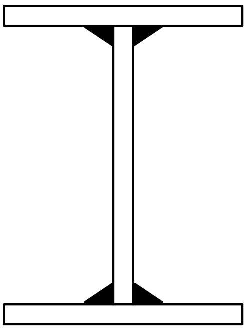


Fig.3.2(a): Welded plate girder

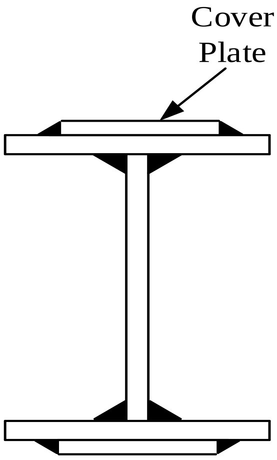


Fig. 3.2(b): Plate girder with cover plate

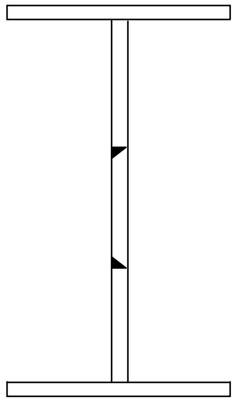


Fig. 3.3(c): Built-up girder with T sections

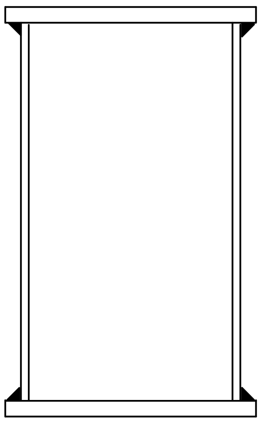


Fig. 3.2(d): Welded box girder

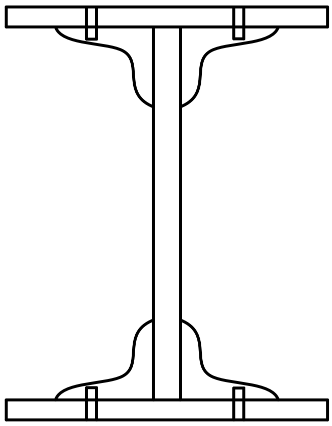


Fig. 3.2(e): Riveted or bolted plate girder

3.2 Advantages and Disadvantages of Plate girder:

Plate girders are used in both buildings and bridges. In buildings, when large column-free spaces are designed to be used as an assembly hall, for example, the plate girder is often the economical solution. In such cases, the designer must choose between a plate girder and a truss. Plate girders, in general, have the following advantages over trusses:

- Connections are less critical for plate girders than for trusses.
- Fabrication cost of plate girders is less than that of trusses.
- Plate girders can be erected more rapidly and more cheaply than trusses.
- Depth of a plate girder is less than the height of a comparable truss. Consequently, plate girders need less vertical clearance than trusses. This makes them very attractive for multilevel highway bridges.
- Plate girders generally vibrate less than trusses under moving loads.
- Painting of plate girders is easier than painting of trusses. This means less maintenance cost for plate girders.

In contrast, plate girders in general are heavier than trusses, especially for very long spans.

3.3 Types of Plate Girder:

There are different types of plate girder that are used in buildings and bridges.

- Box Girder: Providing improved torsional stiffness for long span bridges.
- Hybrid Girder: Providing variable material strength in accordance with stresses. In order to reduce the girder weight and possibly achieve maximum economy, hybrid plate girders are sometimes used. In a hybrid girder, flange plates are made of higher strength steel than that of the web
- Delta girder: Delta girder, may be used for more stability of the compression flange.

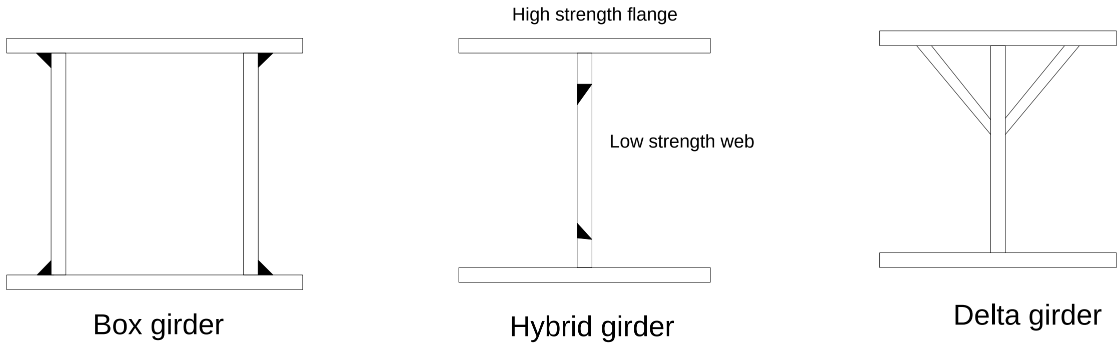
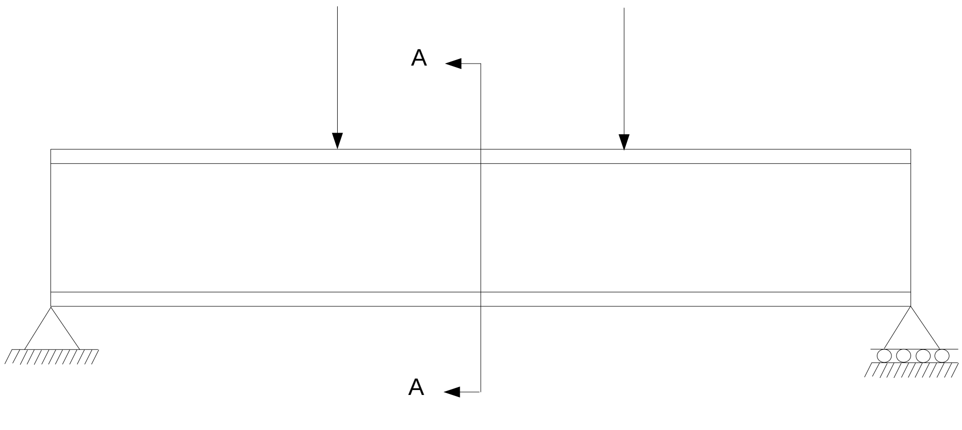


Figure 3.3: Types of plate girder

3.4 Essential Elements of I-section Plate Girder:

In a built-up I section, there are some elements that need to be designed.

- Top flange
- Bottom flange
- Web
- Intermediate stiffener
- Bearing stiffener
- Welding



(a)

Figure 3.4(a): Plate girder without stiffeners

<u>Top and bottom flange plate:</u> Plate girders basically carry the loads by bending. The bending moment is mostly carried by flange plates.

<u>Web:</u> A web plate is needed to unify the two flange plates and to carry the shear.

<u>Intermediate Stiffener:</u> In addition to flange plates and a web plate, a plate girder often consists of stiffeners. Thin web plates are susceptible to unstable behavior. Thick web plates make the girder unnecessarily heavy. A relatively thin web plate strengthened by stiffeners often yields the lightest plate girder. Therefore, intermediate stiffeners are provided to stiffen the web plate against buckling and to resist compressive forces transmitted from the web during tension-field action.

<u>Bearing Stiffener:</u> Bearing stiffeners should always be provided in pairs at the ends of plate girders and if required at points of application of concentrated loads. These bearing stiffeners should extend roughly to the edges of the flange plates.

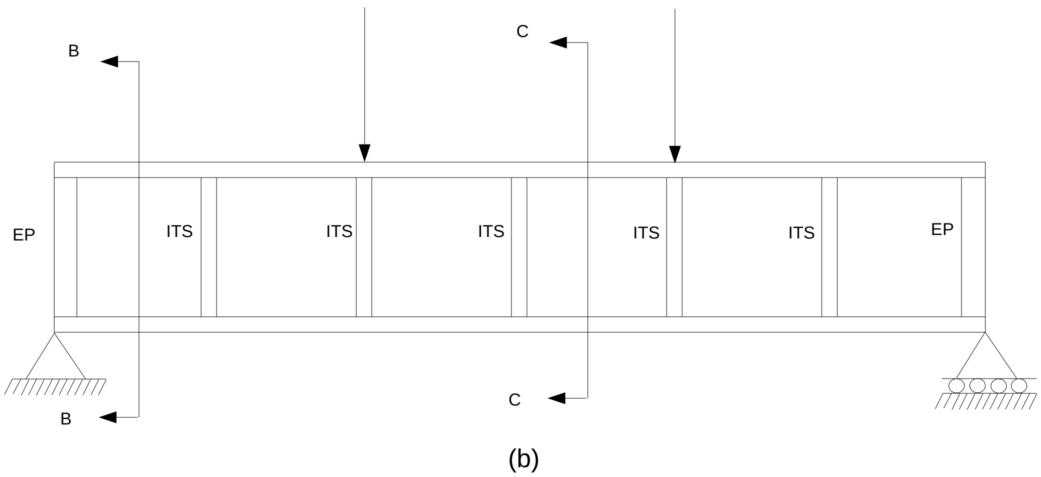


Figure 3.4(b): Plate girder with intermediate and bearing stiffeners

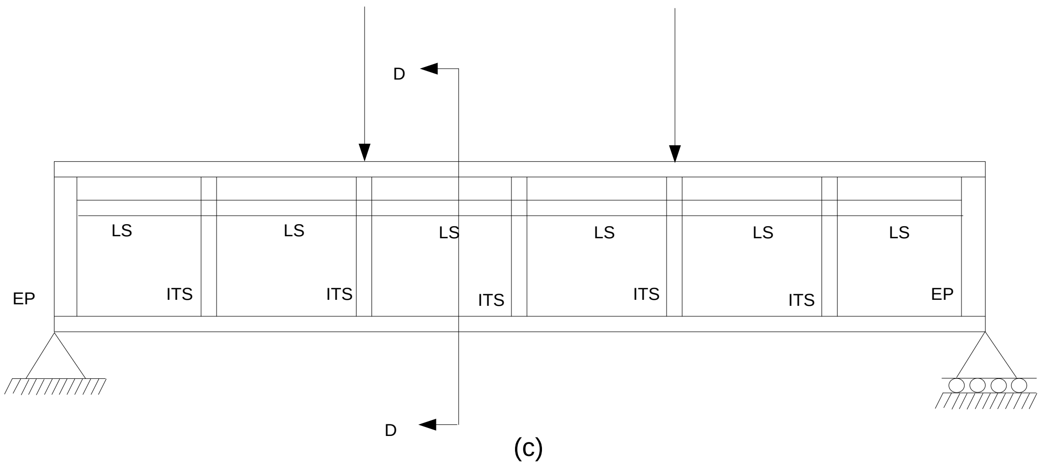


Figure 3.4(c): Plate girder with intermediate, bearing and lateral stiffeners

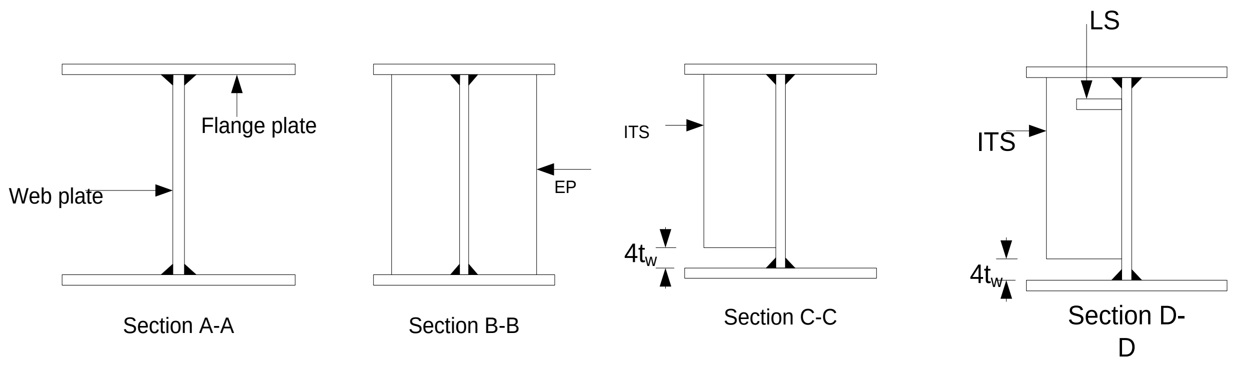


Figure 3.4(d): Forms of plate girder

3.5 Post-Buckling Behavior of the Web Plate:

A relatively thin web plate strengthened by stiffeners often yields the lightest plate girder. Stiffened plate girders are designed on the basis of the ultimate strength concept. As the magnitude of the load on the girder is increased, the web panels between adjacent vertical stiffeners buckle due to diagonal compression resulting from shear. If the plate girder has properly designed stiffeners, the instability of the web plate panels, bounded on all sides by the transverse stiffeners of flanges, will not result in its failure. In fact, after the web panels buckle in shear, the plate girder behaves like the Pratt truss shown in Fig. 3.5(a). It will then be able to carry additional loads. A stiffened plate girder has considerable post-buckling strength.

The Pratt truss of Fig. 3.5(a) is subjected to a concentrated load applied at its midspan. In this truss, the vertical members are in compression and the diagonals are in tension. The post-buckling behavior of the plate girder is analogous to the behavior of this truss. As shown in Fig. 33(b), after the shear instability of the web takes place, a redistribution of stresses occurs; the stiffeners behave like axially compressed members, and shaded portions of the web behave like tension diagonals in the truss of Fig. 33(a). This truss-like behavior is called *tension-field action* in the literature. The post-buckling strength of the web plate may be three or four times its initial buckling strength. Consequently, designs on the basis of tension-field action can yield better economy.

A tension field ordinarily cannot be fully developed in an end panel. This can be understood by considering the horizontal components of the tension fields shown in Figure 3.5(b). (The vertical components are resisted by the stiffeners.) The tension field in panel *CD* is balanced on the left side in part by the tension field in panel *BC*. Thus, interior panels are anchored by adjacent panels. Panel *AB*, however, has no such anchorage on the left side. Hence the anchorage for panel BC must be provided on the left side by a beam-shear panel rather than the tension-field panel shown.

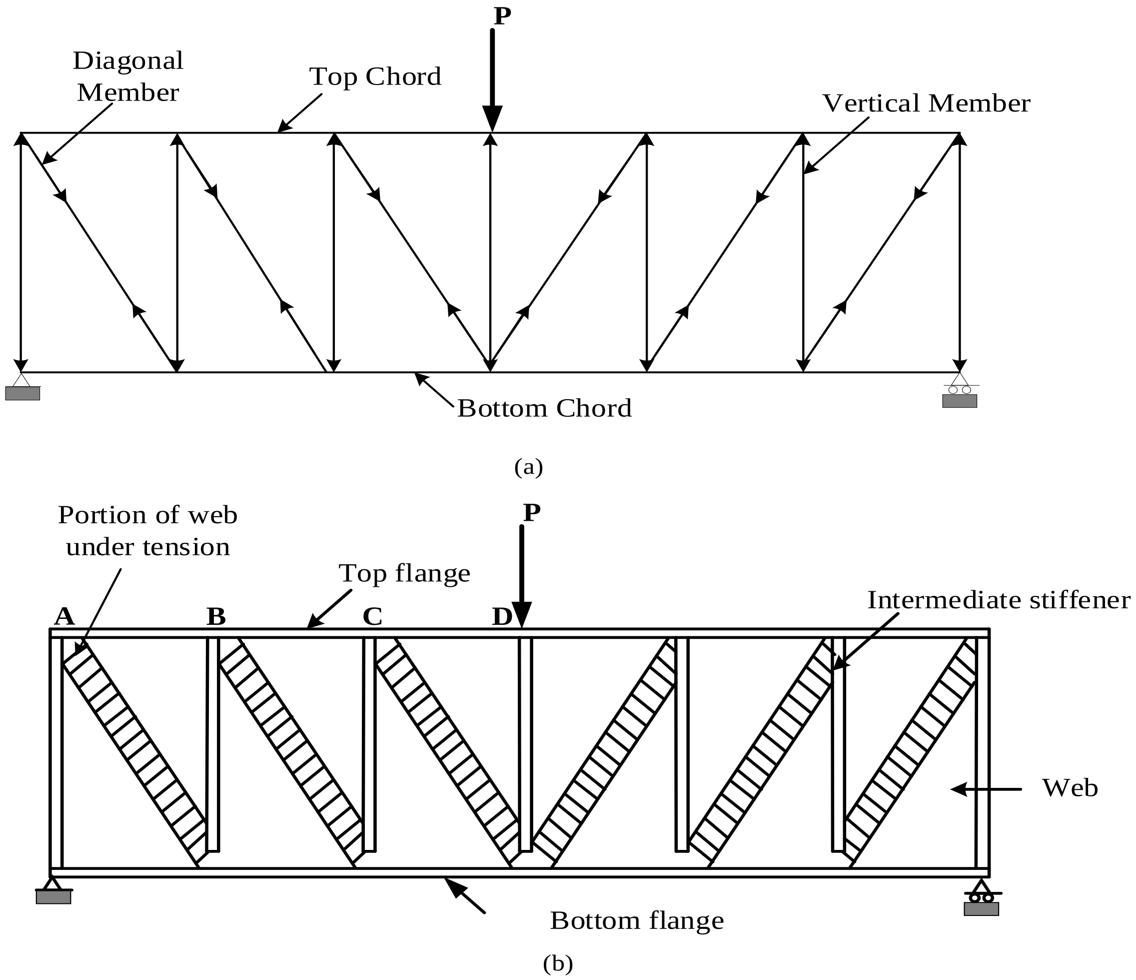


Figure 3.5: Analogy between a truss and a stiffened plate girder

AISC G3.1 lists all of the conditions under which a tension field cannot be used:

a. In end panels

b. When. $\frac{a}{h} < 3$ or. $\frac{a}{h} < (\frac{260}{h/t_w})^2$ (Each of these cases corresponds to $k_v = 5$)

c. When $\frac{2A_w}{(A_{fc}+A_{ft})} > 2.5$

d. When $\frac{h}{b_{fc}}$ or $\frac{h}{b_{ft}} > 6$

Where,

A_w =area of the web

A_{fc} =area ofthe compression flange

A_{ft} =area of the tension flange

b_{fc} =width ofthe compression flange

b_{ft} =width of the tension flange

3.6 Requirements for different components of the plate girder

3.6.1 Proportions of Plate Girders:

Whether a girder web is noncompact or slender depends on h/t_w, the width-to-thickness ratio of the web, where h is the depth ofthe web from inside face of flange to inside face of flange and t_w is the web thickness. From AISC B4 the web of a doubly symmetric I-shaped section is noncompact if

$$3.76\sqrt{\frac{E}{F_y}} < \frac{h}{t_w} \leq 5.70\sqrt{\frac{E}{F_y}} \quad (3.1)$$

and the web is slender if

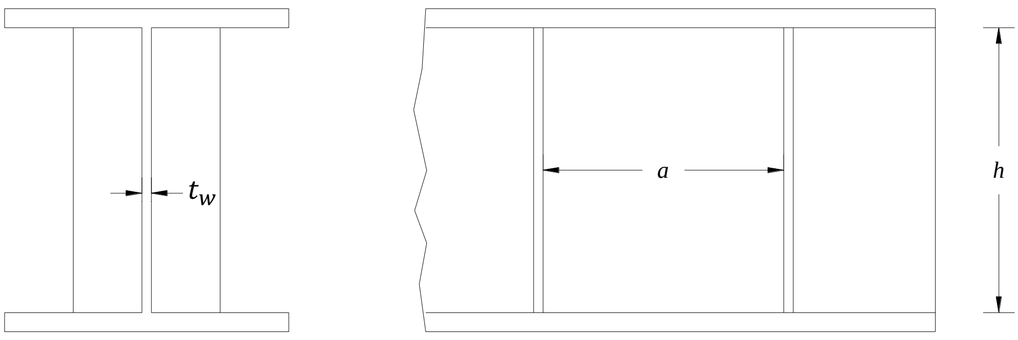
$$\frac{h}{t_w} > 5.70\sqrt{\frac{E}{F_y}} \quad (3.2)$$

To prevent vertical buckling of the compression flange into the web, AISC F13.2 imposes an upper limit on the web slenderness. The limiting value of h/t_w is a function of the aspect ratio, a/h, ofthe girder panels, which is the ratio of intermediate stiffener spacing to web depth

For $\frac{a}{h} \leq 1.5$, $\left(\frac{h}{t_w}\right)_{\max} = 12.0\sqrt{\frac{E}{F_y}}$ (3.3)

For $\frac{a}{h} > 1.5$, $\left(\frac{h}{t_w}\right)_{\max} = \frac{0.40E}{F_y}$ (3.4)

Where, a is the clear distance between stiffeners.



Section- Front View Section- Side View

Figure 3.6: Front and side view of the plate girder

3.6.2 Requirement for Flexural Strength

The nominal flexural strength M_n of a plate girder is based on one of the limit states of tension flange yielding, compression flange yielding or local buckling (FLB), or lateral torsional buckling (LTB). The aspects related to flexural strength are discussed in AISC chapter F.

Tension Flange Yielding

AISC F5 gives the nominal flexural strength based on tension flange yielding as

$$M_n = F_y S_{xt} \quad (3.5)$$

where S_{xt} =elastic section modulus referred to the tension side.

Compression Flange Yielding

The compression flange nominal strength is given by,

$$M_n = R_{pg} F_{cr} S_{xc} \tag{3.6}$$

Where,

R_{pg} =bending strength reduction factor

F_{cr} =critical compressive flange stress, based on either yielding or local buckling

S_{xc} =elastic section modulus referred to the compression side

The bending strength reduction factor is given by

$$R_{pg} = 1 - \frac{a_w}{1{,}200 + 300 a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \leq 1.0 \tag{3.7}$$

where

$$a_w = \frac{h_c\, t_w}{b_{fc} t_{fc}} \leq 10 \tag{3.8}$$

b_{fc} =width of the compression flange

t_{fc} =thickness ofthe compression flange

The critical compression flange stress F_{cr} depends on whether the flange is compact, noncompact, or slender. The AISC Specification uses the generic notation λ, λ_p, and λ_r to define the flange width - to - thickness ratio and its limits. From AISC Table B4.1b,

$$\lambda = \frac{b_f}{2t_f} \tag{3.9}$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} \tag{3.10}$$

$$\lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}} \tag{3.11}$$

$$k_c = \frac{4}{\sqrt{h/t_w}} \text{ but } (0.35 \leq k_c \leq 0.76) \tag{3.12}$$

$F_L = 0.7F_y$ for girders with slender webs.

If $\lambda \leq \lambda_p$, the flange is compact. The limit state ofyielding will control and $F_{cr} = F_y$, resulting in $M_n = R_{pg} F_y S_{xc}$

If $\lambda_p < \lambda \leq \lambda_r$, the flange is noncompact. Inelastic FLB will control and

$$F_{cr} = F_y - 0.3F_y \left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \tag{3.13}$$

If $\lambda > \lambda_r$, the flange is slender, elastic FLB will control and

$$F_{cr} = \frac{0.9Ek_c}{\left(\frac{b_f}{2t_f}\right)^2} \tag{3.14}$$

Lateral Torsional Buckling

Whether lateral-torsional buckling will occur depends on the amount of lateral support i.e. unbraced length L_b. If the unbraced length is small enough, yielding or flange local buckling will occur before lateral-torsional buckling. The length parameters are L_p and L_r, where

$$L_p = 1.1r_t\sqrt{\frac{E}{F_y}} \tag{3.15}$$

$$L_r = \pi r_t\sqrt{\frac{E}{0.7F_y}} \tag{3.16}$$

Here,

r_t =radius of gyration about the weak axis for a portion of the cross section consisting of the compression flange and one third of the compressed part of the web. For a doubly symmetric girder, this dimension will be one sixth of the web depth.

If $L_b \leq L_p$, there is no lateral torsional buckling.

If $L_p < L_b \leq L_r$, Failure will be by inelastic LTB, and (3.17)

$$F_{cr} = C_bF_y - 0.3F_y \left(\frac{L_b - L_p}{L_r - L_p}\right) \leq F_y$$

$If\, L_b > L_r$, failure will be by elastic LTB, and (3.18)

$$F_{cr} = \frac{C_b\pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \leq F_y$$

C_b =factor to account for nonuniform bending within the unbraced length L_b.If the moment within the unbraced length L_b is uniform (constant), there is no moment gradient and $C_b = 1.0$

3.6.3 Requirement for Shear Strength:

The AISC Specification covers shear strength in Chapter G. In that coverage the constants k_v and C_v are used. AISC defines k_v, which is a plate buckling coefficient,

$$k_v = 5 + \frac{5}{(a/h)^2} \quad (3.19)$$

$$= 5 \text{ if } \frac{a}{h} > 3$$

$$= 5 \text{ if } \frac{a}{h} > \left(\frac{260}{h/t_w}\right)^2$$

$$= 5 \text{ in unstiffened webs with } \frac{h}{t_w} < 260$$

For C_v, which can be defined as the ratio of the critical web shear stress to the web shear yield stress,

$$\text{If } \frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E}{F_y}}, \quad C_v = 1.0 \quad (3.20)$$

$$\text{If } 1.10\sqrt{\frac{k_v E}{F_y}} < \frac{h}{t_w} \leq 1.37\sqrt{\frac{k_v E}{F_y}}, C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \quad (3.21)$$

$$\text{If } \frac{h}{t_w} > 1.37\sqrt{\frac{k_v E}{F_y}}, \quad C_v = \frac{1.51 k_v E}{(h/t_w)^2 F_y} \quad (3.22)$$

Whether the shear strength is based on web shear yielding or web shear buckling depends on the web width to thickness ratio h/t_w. If

$$\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E}{F_y}}$$

the strength is based on shear yielding, and $V_n = 0.6F_y A_w$ (3.23)

where A_w =cross-sectional area of the web. If

$$\frac{h}{t_w} > 1.10\sqrt{\frac{k_v E}{F_y}},$$

the strength will be based on shear buckling or shear buckling plus tension field action. If tension field behavior exists,

AISC Equation G3-2 can also be written as

$$V_n = 0.6F_y A_w C_v + 0.6F_y A_w \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \quad (3.24)$$

The first term in the equation gives the web shear buckling strength and the second term gives the post buckling strength. If there is no tension field action, the second term is omitted, resulting in

$$V_n = 0.6F_y A_w C_v \tag{3.25}$$

Solution of AISC Equations G2-1 (without tension field) and G3-2 (with tension field) is facilitated by curves given in Part 3 of the Manual. Tables 3-16a and 3-16b present curves that relate the variables of these two equations for steel with a yield stress of 36 ksi and Tables 3-17a and 3-17b do the same for steels with a yield stress of 50 ksi.

3.6.4 Requirements for Intermediate Stiffener

Without a Tension Field:

The requirements for stiffeners when a tension field is not present are given in AISC G2.2. The required moment of inertia of a pair of stiffeners about an axis through the web is

$$I_{st} \geq b t_w^3 j \tag{3.26}$$

Where,

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \tag{3.27}$$

b =smaller of a and h.

With a Tension Field

The requirements for stiffeners where tension field action is used are given in AISC G3.3. The first requirement is for the proportions of the stiffener.

$$(\frac{b}{t})_{st} \leq 0.56\sqrt{\frac{E}{F_{yst}}} \tag{3.28}$$

Where,

$(\frac{b}{t})_{st}$ =width to thickness ratio of the stiffener cross section

F_{yst} =yield stress ofthe stiffener

The second requirement is for the moment of inertia of the stiffener or pair of stiffeners.

$$I_{st} \geq I_{st1} + (I_{st2} - I_{st1})\frac{V_r - V_{c1}}{V_{c2} - V_{c1}} \tag{3.29}$$

Where,

I_{st1} =required moment of inertia as calculated for the no tension field case

I_{st2} = moment of inertia required to develop the buckling plus post buckling shear strength

$$= \frac{h^4 p_{st}^{1.3}}{40} \left(\frac{F_{yw}}{E} \right)^{1.5} \tag{3.30}$$

$$p_{st} = \max \left(\frac{F_{yw}}{F_{yst}}, 1 \right)$$

F_{yw} =yield stress ofthe girder web

V_r = the larger of the required shear strengths (V_u for LRFD, V_a for ASD) on each side of the stiffener; that is, in the adjacent web panels

V_{c1} = the smaller of the available shear strengths (($\varphi_v V_n$ for LRFD, V_n/Ω_v for ASD) in the adjacent panels, calculated with no tension field action

$V_{c2} = the$ smaller of the available shear strengths (($\varphi_v V_n$ for LRFD, V$/\Omega_v$ for ASD) in the adjacent panels, calculated with tension field action

3.6.5 Requirements for Bearing Stiffener

Although the web can be proportioned to directly resist any applied concentrated loads, bearing stiffeners are usually provided. If stiffeners are used to resist the full concentrated load, the limit states of web yielding, web crippling, and side-sway web buckling do not need to be checked. The nominal bearing strength of a stiffener is given in AISC J7 as,

$$R_n = 1.8 F_y A_{pb} \tag{3.31}$$

For LRFD, the resistance factor is $\varphi = 0.75$. For ASD, the safety factor is $\Omega = 2.00$.

Full depth stiffeners should be used in pairs and analyzed as axially loaded columns subject to the following guidelines:

- The cross section of the axially loaded member consists of the stiffener plates and a length of the web This length can be no greater than 12 times the web thickness for an end stiffener or 25 times the web thickness for an interior stiffener.
- The effective length should be taken as 0.75 times the actual length that is, $KL = 0.75h$.
- The nominal axial strength is based on the provisions of AISC J4.4, “Strength of Elements in Compression which are as follows:

$$\text{For } \frac{KL}{r} \leq 25 \tag{3.32}$$

$$P_n = F_y A_g$$

This is the squash load for the stiffener that causes compression yielding with no buckling. For LRFD, the resistance factor for this limit state is $\varphi = 0.90$, for ASD, the safety factor is $\Omega = 1.67$. For $\frac{KL}{r} > 25$, the usual requirements for compression members apply

- The weld connecting the stiffener to the web should have the capacity to transfer the unbalanced force. Conservatively, the weld can be designed to carry the entire concentrated load. If the stiffener bears on the compression flange, it need not be welded to the flange.

Although no width to thickness ratio limit is given in the Specification for bearing stiffeners, the requirement of AISC Equation G3-3 for intermediate stiffeners can be used as a guide in proportioning bearing stiffeners:

$$\left(\frac{b}{t}\right)_{st} \leq 0.56\sqrt{\frac{E}{F_{yst}}}$$

3.7 Design Procedure

The primary task in plate girder design is to determine the size of the web and the flanges. If a variable moment of inertia is desired, decisions must be made regarding the method of varying the flange size—that is, whether to use cover plates or different thicknesses of flange plate at different points along the length of the girder. A decision about whether to use intermediate stiffeners must be made early in the process because it will affect the web thickness. If bearing stiffeners are needed, they must be designed. Finally, the various components must be connected by properly designed welds. The following step-by-step procedure is recommended.

1. Select the overall depth. The optimum depth of a girder depends on the situation. Some authors recommend an overall depth of $\frac{1}{10}$ to $\frac{1}{12}$ of the span length. Others suggest a range of $\frac{1}{6}$ to $\frac{1}{20}$ give procedures for determining the depth that incorporate the required moment strength and a specified $\frac{h}{t_w}$ ratio. As with any beam design, constraints on the maximum depth could establish the depth by default.

2. Select a trial web size. The web depth can be estimated by subtracting twice the flange thickness from the overall depth selected. Of course, at this stage of the design, the flange thickness must also be estimated. The web thickness t_w can then be found by using the following limitations as a guide. Once h and t_w have been selected, determine whether the web width to thickness ratio qualifies this member as a slender-web flexural member. If so, the provisions of AISC F5 can be used. (If the web is noncompact, AISC F5 can still be used, but it will be conservative)

3. Estimate the flange size: The required flange area can be estimated from a simple formula derived as follows.

$$A_f = \frac{M_{nreq}}{hR_{pg}F_{cr}} - \frac{t_w h}{6}$$

If we assume that $R_{pg} = 1.0$ and $F_{cr} = F_y$, the required area of one flange is

$$\boxed{A_f = \frac{M_{nreq}}{hF_y} - \frac{A_w}{6}} \quad \text{…………………(3.33)}$$

Where,
M_{nreq} =Required nominal flexural strength
$= M_U/\varphi_b$ for LRFD
$= \Omega_b M_a$ for ASD

A_w = web area

Once the required flange area has been determined, select the width and thickness. If the thickness originally used in the estimate of the web depth is retained, no adjustment in the web depth will be needed. Otherwise changes have to be made regarding previously selected components. At this point, an estimated girder weight can be computed, then M_{nreq} should be recomputed.

4. Check the bending strength of the trial section.

5. Determine intermediate stiffener spacings and check the shear strength of the trial section. The design curves in Part 3 of the AISC Manual can be used for this purpose or AISC Equation G3-1 and G3-2

6. Design intermediate stiffeners. If there is not a tension field, the intermediate stiffeners must be proportioned to satisfy the moment of inertia requirement of AISC Equation G2-7. If there is a tension field, the width to thickness ratio limit of AISC Equation G3-3 and the moment of inertia requirement of AISC Equation G3-4 must be satisfied.

7. Design bearing stiffeners. To determine whether bearing stiffeners are needed, check the web resistance to concentrated loads (web yielding, web crippling, and web sidesway buckling). Alternatively, bearing stiffeners can be provided to fully resist the concentrated loads, and the web need not be checked. If bearing stiffeners are used, the following design procedure can be used.

a. Try a width that brings the edge ofthe stiffener near the edge of the flange and a thickness that satisfies AISC Equation G3-3:

$$(\frac{b}{t})_{st} \leq 0.56\sqrt{\frac{E}{F_{yst}}}$$

b. Compute the cross - sectional area needed for bearing strength. Compare this area with the trial area and revise if necessary.

c. Check the stiffener-web assembly as a compression member.

8. Design the flange-to-web welds, stiffener-to-web welds, and any other connections (flange segments, web splices, etc.)

3.8 Design Example

Problem Description

Fig 3.7: Beam Layout

Design the internal plate girder where,
Thickness of the slab = 5 in
maximum permissible depth of beam = 65 in
Concentrated load, $P_D = 78$ kip & $P_L = 58$ kip are applied at the midpoint
Steel used = A36 ($F_y = 36$ ksi)
Electrode: E70XX (for welding)

Assume that the girder has lateral support at the ends and at the point of application of concentrated load. The girder is restrained against rotation at these points.

Solution:
The tributary are for the internal beam, A = 0.5 (60+40)(10)2 = 1000 ft²

Assume floor finish = 40 psf and Live load on slab= 75 psf

∴ Service dead load, w_D = (150×5/12+40) $(\frac{1000}{60})$ $(\frac{1}{1000})$ = 1.70 k/ft

∴ Service live load, w_L= 75 $(\frac{1000}{60})$ $(\frac{1}{1000})$ = 1.25 k/ft

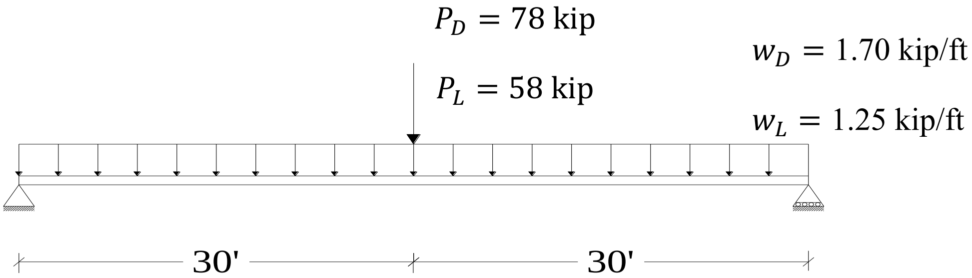


Figure 3.8: Plate girder to be designed under service loads

Factored loads = 1.2 × Dead Load + 1.6 × Live Load

∴ Total concentrated load = 1.2 × 78 + 1.6 × 58 = 186.4 kip

∴ Total uniform load = 1.2 × 1.70 + 1.6 × 1.25 = 4.04 kip/ft

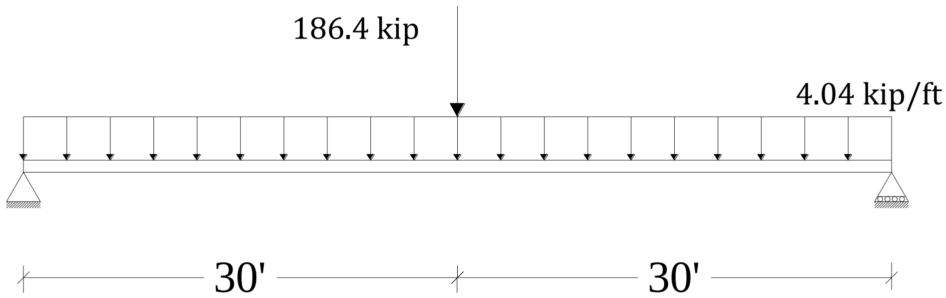


Figure 3.9: Factored Loads (excluding girder self-weight)

Selection of Web Plate:

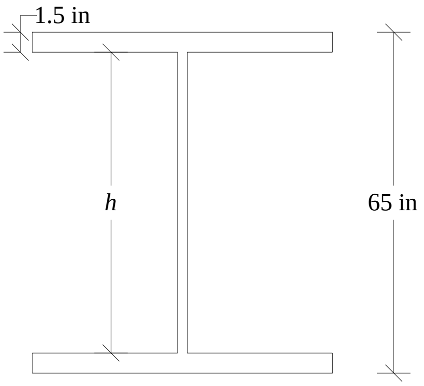


Figure 3.10: Cross-section of the plate

Determine the overall depth.

$$\frac{\text{Span Length}}{10} = \frac{60(12)}{10} = 72 \text{ in.}$$

$$\frac{\text{Span Length}}{10} = \frac{60(12)}{12} = 60 \text{ in.}$$

Use the maximum permissible depth of 65 inches.

Try a flange thickness of $t_f = 15$ inches and a web depth of

$h = 65 - 2(1.5) = 62$ in.

To determine the web thickness, first the limiting values of h/t_w have to be examined

From Equations 3.3 and 3.4:

For $\frac{a}{h} \leq 1.5$,

$$\left(\frac{\mathrm{h}}{\mathrm{t_w}}\right)_{\max} = 12.0\sqrt{\frac{E}{F_y}} = 12.0\sqrt{\frac{29{,}000}{36}} = 340.6$$

$$\min\ t_w = \frac{62}{340.6} = 0.182 \text{ in.}$$

For $\frac{a}{h} > 1.5$,

$$\left(\frac{h}{t_w}\right)_{\max} = \frac{0.40E}{F_y} = \frac{0.40(29{,}000)}{36} = 322.2$$

$\min t_w = \frac{62}{322.2} = 0.192$ in.

$\boxed{\textbf{Try a } ^{5}/_{16} \times 62 \textbf{ web plate.}}$

Determine whether the web is slender

$$\frac{h}{t_w} = \frac{62}{5/16} = 198.4$$

$$5.70\sqrt{\frac{E}{F_y}} = 5.70\sqrt{\frac{29{,}000}{36}} = 161.8 < 198.4$$

∴The web is slender. So, the AISC provisions for plate girder can be used.

Selection of Flange Size

Determine the required flange size. From Fig.3.9, the maximum factored load bending moment is

$$M_u = \frac{186.4(60)}{4} + \frac{4.040(60)^2}{8} = 4614 \text{ ft-kips}$$

∴ The required flange area is, $A_f = \frac{M_{nreq}}{hF_y} - \frac{A_w}{6}$

$$= \frac{M_u/\varphi_b}{hF_y} - \frac{A_w}{6}$$

$$= \frac{(4614 \times 12)/0.90}{62(36)} - \frac{62(5/16)}{6} = 24.33 \text{ in}^2$$

If the original estimate of the flange thickness is retained, the required width is

$$b_f = \frac{A_f}{t_f} = \frac{24.33}{1.5} = 16.2 \text{ in.}$$

$\boxed{\textbf{Try a } 1\,^{1}/_{2} \times 18 \textbf{ flange plate.}}$

The girder weight can now be computed.

Web area: $62(5/16) = 19.38$ in.^{2}
Flange area: $2(15 \times 18) = 54.00$ in.^{2}
Total: 73.38 in^2

Weight = $\frac{73.38}{144}$ =$(490) = 250$ lb/ft (Self-weight of structural steel = 490 lb/ft^3)

The adjusted bending moment is

$M_u = 4614 + \frac{(1.2 \times 0.250)(60)^2}{8} = 4749$ ft-kips

Figure 3.11 shows the trial section, and Figure 3.12 shows the shear and bending moment diagrams for the factored loads, which include the girder weight of $250\ \mathrm{lb/ft}$.

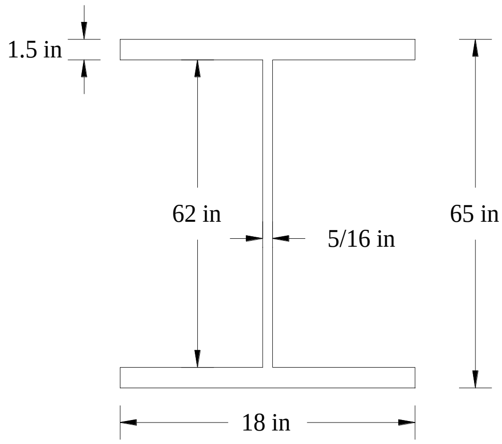


Figure 3.11: Cross-section of the plate

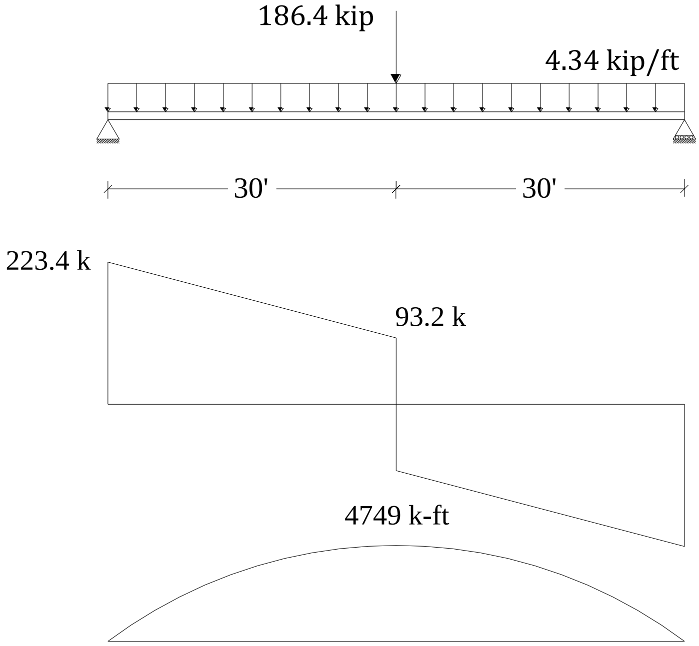


Figure 3.12: SFD and BMD for the recomputed factored

Check the flexural strength of the trial section.

From Figure 3.11, the moment of inertia about the axis of bending is

$$l_x = \frac{(5/16)(62)^3}{12} + 2(1.5)(18)(31.75)^2 = 60{,}640\ \mathrm{in}^4$$

and the elastic section modulus is

$$S_X = \frac{I_x}{c} = \frac{60{,}640}{32.5} = 1866\ \mathrm{in}^3$$

An examination of AISC Equations F5-7 and F5-10 shows that for a girder with a symmetrical cross section, the flexural strength will **never be controlled by tension flange yielding.**

Check for compression flange buckling:

Determine whether the compression flange is compact, noncompact, or slender. (use Equations 3.9 to 3.12)

$$\lambda = \frac{b_f}{2t_f} = \frac{18}{2(15)} = 6.0$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.3s\sqrt{\frac{29{,}000}{36}} = 10.79$$

Since $\lambda < \lambda_p$, there is no flange local buckling. The compression flange strength is therefore based on yielding, and $F_{cr} = F_y = 36$ ksi

To compute the plate girder strength reduction factor R_{pg}, the value of a_w will be needed:

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} = \frac{62(5/16)}{18(1.5)} = 0.7176 < 10$$

From Equation-7

$$R_{pg} = 1 - \frac{a_w}{1{,}200 + 300a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \leq 1.0$$

$$= 1 - \frac{0.7176}{1200 + 300(0.7176)}\left(198.4 - 5.7\sqrt{\frac{29000}{36}}\ \right) \leq 1.0$$

$$= 0.9814$$

From AISC Equation F5-7, the nominal flexural strength for the compression flange is

$$M_n = R_{pg} F_{cr} S_{\text{xc}} = 0.9814(36)(1866) = 65{,}930 \text{ in.-kips} = 5494 \text{ft-kips}$$

and the design strength is

$$\varphi_b M_n = 0.90(5494) = 4945 \text{ ft-kips} > 4749 \text{ ft-kips (OK)}$$

Although this capacity is somewhat more than needed, the excess will compensate for the weight of stiffeners and other incidentals that have not yet been accounted for.

Check for lateral torsional buckling:

$$I_y = \frac{1}{12}(1.5)(18)^3 + \frac{1}{12}(62/6)\left(\frac{5}{16}\right)^3 = 729.03\ in^4$$

$$A = 18(1.5) + 10.333\left(\frac{5}{16}\right) = 30.23\ in^2$$

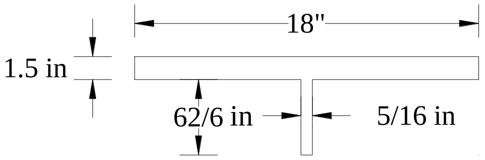


Figure 3.13: Zone corresponding to lateral torsional

$r_t = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{729.03}{30.23}} = 4.91\ in$

Check the unbraced length.

$L_b = 30$ ft

$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} = 1.1(4.91)\sqrt{\frac{29{,}000}{36}} = 153.29$ in. $= 12.77$ ft

$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} = \pi(4.91)\sqrt{\frac{29{,}000}{0.7(36)}} = 523.28$ in. $= 43.6$ ft

Since $L_p < L_b < L_r$, the girder is subject to inelastic lateral torsional buckling. From Equation-17,

$F_{cr} = C_b F_y - 0.3F_y(\frac{L_b - L_p}{L_r - L_p}) \leq F_y$

For the computation of C_b, refer to Figure 3.12, which shows the loading, shear, and bending moment diagrams based on factored loads. The unsupported segment is divided into four equal spaces, we get points A, B, and C located at 7.5 ft, 15 ft and 22.2 ft from the left end of the girder. The corresponding bending moments are

$M_A = 223.4(7.5) - 4.34(7.5)^2/2 = 1553.43$ ft-kips

$M_B = 223.4(15) - 4.34(15)^2/2 = 2862.75$ ft-kips

$M_C = 223.4(22.5) - 4.34(22.5)^2/2 = 3927.94$ ft-kips

From AISC Equation F1-l,

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C}$$

$$= \frac{12.5(4749)}{2.5(4749) + 3(1553.43) + 4(2862.75) + 3(3927.94)} = 1.5$$

$F_{cr} = C_b F_y - 0.3F_y(\frac{L_b - L_p}{L_r - L_p}) \leq F_y$

$= 1.5 \times 36 - (0.3 \times 36)\ \frac{30-12.77}{43.58-12.77} = 47.96$ ksi

Since 47.96 $ksi > F_y = 36$ ksi, use $F_{cr} = 36$ ksi (same as for the other limit states).

The nominal flexural strength is,$\varphi_b M_n = 4945$ ft-kips > 4749 ft-kips (OK)

Use a ${}^{5}/_{16} \times 62$ web and 1 ${}^{1}/_{2} \times 18$ flanges, as shown in Fig.4

Select intermediate stiffener spacing and check the corresponding shear strength

The shear is maximum at the support, but tension field action cannot be used in an end panel. Table 3-16a in Part 3 of the AISC Manual can used to obtain the required size of the end panel. The curves will be entered with values of h/t_w and the required $\varphi_v V_n / A_w$,

Where,

$$\frac{h}{t_w} = 198.4$$

$$A_w = 62(\frac{5}{16}) = 19.38 \text{ in.}^2$$

Required $\frac{\varphi_v V_n}{A_w} = \frac{V_u}{A_w} = \frac{223.4}{19.38} = 11.5$ ksi

Using $h/t_w = 198$ and $\varphi_v V_n / A_w = 12$ ksi, we get a value of a/h of approximately 0.60. The corresponding intermediate stiffener spacing is

$a = 0.60h = 0.60(62) = 37.2$ in.

Although the required stiffener spacing is a clear distance, the use of center to center distances is somewhat simpler and will be slightly conservative. In addition, because of the approximations involved in using the curves, we will be conservative in rounding the value of a. Use a distance of 36 inches from the center of the end bearing stiffener to the center of the first intermediate stiffener.

Now, $a = 36\ in \quad \therefore \frac{a}{h} = \frac{36}{62} = 0.58$

From Equation-19,

$$k_v = 5 + \frac{5}{(a/h)^2} = 19.86$$

$$\frac{h}{t_w} = 198.4$$

$$1.10\sqrt{\frac{k_v E}{F_y}} = 1.10\sqrt{\frac{19.86(29000)}{36}} = 139.13$$

$$1.37\sqrt{\frac{k_v E}{F_y}} = 1.37\sqrt{\frac{19.86(29000)}{36}} = 173.28$$

$$\therefore \frac{h}{t_w} > 1.37\sqrt{\frac{k_v E}{F_y}}$$

So, from Equation-22, $C_v = \frac{1.51 k_v E}{(h/t_w)^2 F_y} = 0.61$

As no tension field action occurs at the end panel, from equation-25

$V_n = 0.6 F_y A_w C_v$ = 0.6 (36) (19.375) (0.61) =255.285 kip

φV_n= 0.9 (255.285) = 229.76 kip >223.4 kip (OK)

Determine the intermediate stiffener spacings needed for shear strength outside the end panels. At a distance of 36 inches from the left end, the shear is

$$V_u = 223.4 - 4.34 \ (\frac{36}{12}) = 210.4 \text{ kips}$$

$$\text{Required } \frac{\varphi_v V_n}{A_w} = \frac{210.4}{19.38} = 10.86 \text{ ksi}$$

Tension field action can be used outside the end panels, so the curves of AISC Table 3-16b will be used. For $h/t_w = 198$ and $\varphi_v V_n/A_w = 11$ ksi,

$$\frac{a}{h} = 1.60$$

The required stiffener spacing is, $a = 1.60h = 1.60(62) = 99.2$ in.

This maximum spacing will apply for the remaining distance to the centerline of the girder. This distance is

$30(12) - 36 = 324$ in.

The number of spaces required is, $\frac{324}{99.2} = 3.27$

Use four spaces. This results in a spacing of, $\frac{324}{4} = 81$ in

Before proceeding, check the conditions of AISC G3.1 to be sure that tension field action can be used for this girder and this stiffener spacing.

$$a. \frac{a}{h} = \frac{81}{62} = 1.306 < 3 \text{ (OK)}$$

$$b. \frac{a}{h} < (\frac{260}{h/t_w})^2 = (\frac{260}{198.4})^2 = 1.717 \text{ (OK)}$$

(Conditions a and *b* are automatically satisfied by staying within the boundaries defined by the upper curve and the lower solid curve of Manual Table 3-16b.)

$$c. \frac{2A_w}{(A_{fc}+A_{ft})} = \frac{2(19.38)}{(27+27)} = 0.7178 < 2.5 \text{ (OK)}$$

$$d. \frac{h}{b_{fc}} = \frac{h}{b_{ft}} = \frac{62}{18} = 3.444 < 6 \text{ (OK)}$$

The following spacings will be used from each end of the girder: one at 36 inches and four at 81 inches, as shown in Figure 3.14.

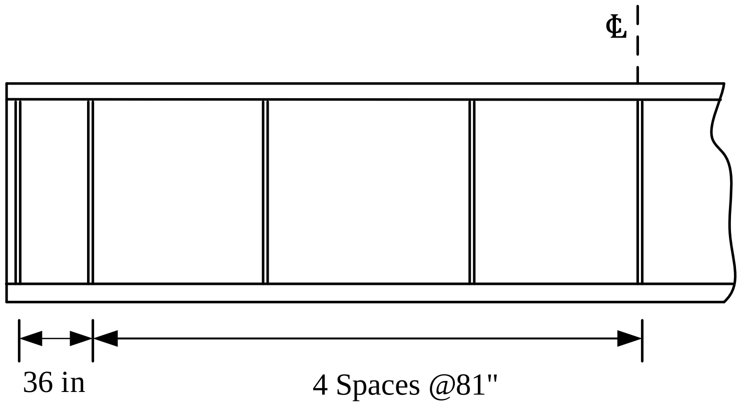


Figure 3.14: Spacing of Intermediate stiffeners

Size of Intermediate Stiffeners:

The first stiffener, placed at 36 inches, defines the left boundary of the first tension field panel. This stiffener must therefore be proportioned to satisfy the requirements of AISC G3.3. To determine a trial width for all stiffeners, consider the available space. The maximum possible width is

$\frac{b_f - t_w}{2} = \frac{18-5/16}{2}$ = 8.84 in. Try $b = 4in.$

Using 3.28, minimum required thickness:

$$(\frac{b}{t})_{st} \leq 0.56\sqrt{\frac{E}{F_{yst}}}$$

$$\frac{4}{t} \leq 0.56\sqrt{\frac{29{,}000}{36}}$$

$t \geq 0.252$ in.

From 3.29 the required moment of inertia

$$l_{st} \geq l_{st1} + (I_{st2} - I_{st1})\frac{V_r - V_{c1}}{V_{c2} - V_{c1}}$$

Now,

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5$$

$$= \frac{2.5}{(36/62)^2} - 2 = 5.415 > 0.5$$

From 3.26, The required moment of inertia calculated for the no tension field case,

$I_{st1} = bt_w^3 j = \min(a, h) t_w^3 j$

$= \min(36{,}62)(5/16)^3(5.415) = 62\,(5/16)^3(5.415) = 5.949\ in^4$

I_{st2} = moment of inertia required to develop the buckling plus post buckling shear strength

$$= \frac{h^4 \rho_{st}^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5}$$

$$p_{st} = \max\left(\frac{F_{yw}}{F_{yst}}, 1\right) = \max\left(\frac{36}{36}, 1\right) = 1$$

$$I_{st2} = \frac{(62)^4(1)^{1.3}}{40}\left(\frac{36}{29{,}000}\right)^{1.5} = 16.16\ in^4$$

To the left of this stiffener, the stiffener spacing is 36 inches, and to the right, the spacing is 81 in. The longer panel will have the smaller strength for both the tension field and the no tension field cases defined by V_{c1} and V_{c2}. From AISC Manual Table 3-16a (no tension field action), for $h/t_w = 198.4$ and $a/h = 81/62 = 1.306$,

$\frac{\varphi_v V_n}{A_w}$ = 5 ksi (by interpolation)

For $A_w = ht_w = 62(5/16) = 19.38$,

$\varphi_v V_n = V_{c1} = 5A_w = 5(19.38) = 96.9$ kips

From Table 3-16b (tension field action), for $h/t_w = 198.4$ and $a/h = 1.306$,

$\frac{\varphi_v V_n}{A_w}$ = 12.3 ksi (by interpolation)

$\varphi_v V_n = V_{c2} = 12.3A_w = 12.3(19.38) = 238$ kips

From Figure 3.12, the larger required strength in the two adjacent panels is

$V_r = V_u = 223.4$ kips.

From AISC Equation G3-4,

$$I_{st} \geq I_{st1} + (l_{st2} - I_{st1})\frac{V_r - V_{c1}}{V_{c2} - V_{c1}}$$

$= 5.949 + (16.16 - 5.949)\,\frac{223.4 - 96.9}{238 - 96.9} = 15.1\ in^4$

Try two 3/8 × 4 plates

From Figure 3.15 and the parallel-axis theorem,

$I_{st} = \Sigma(\bar{I} \times Ad^2)$

$= \frac{(3/8)(4)^3}{12} + (3/8)(4)(2 + 5/32)^2 \times 2$ stiffeners

$= 17.9\ in^4 > 15.1\ in^4$ (OK)

We will use this size for all of the intermediate stiffeners. To determine the length of the stiffeners, first compute the distance between the stiffener to web weld and the web to flange weld (see Figure 3.15):

Minimum distance $= 4t_w = 4(\frac{5}{16}) = 1.25$ in.

Maximum distance $= 6t_w = 6(\frac{5}{16}) = 1.875$ in.

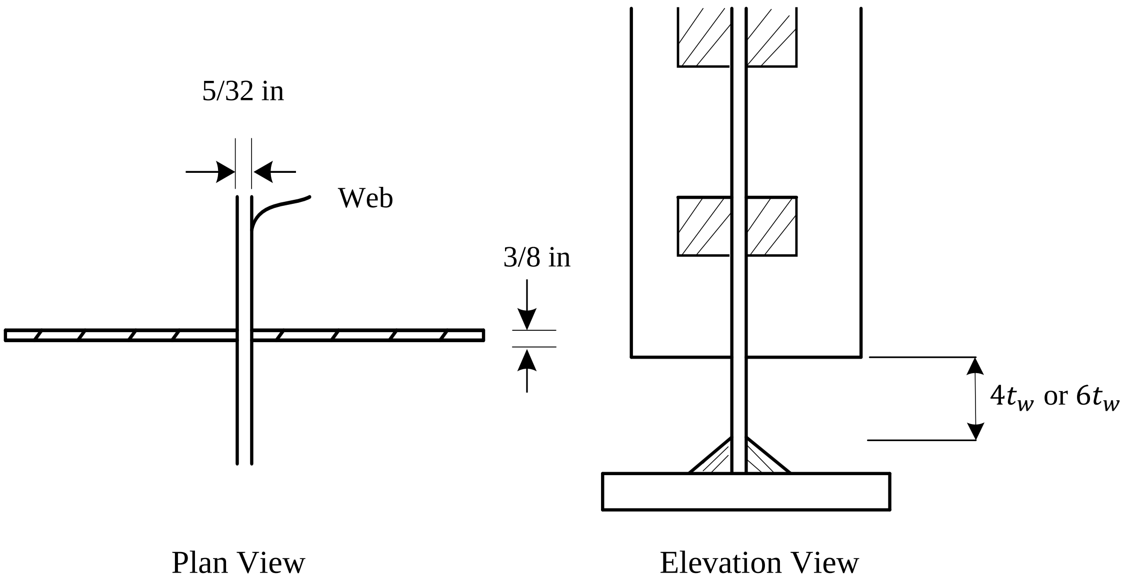


Figure 3.15: Plan and Elevation View of Intermediate Stiffeners

If we assume a flange to web weld size o 5/16 in. and 1.25 in. between welds, the approximate length of the stiffener is

$h -$ weld size $- 1.25 = 62 - 0.3125 - 1.25$

$= 60.44$ in. ∴Use 60 in.

Use two plates $\frac{3}{8} \times 4 \times 5'$ for the intermediate stiffeners.

Size of Bearing Stiffeners

Bearing stiffeners will be provided at the supports and at midspan. Since there will be a stiffener at each concentrated load, there is no need to investigate the resistance of the web to these loads. If the stiffeners were not provided, the web would need to be protected by providing sufficient bearing length, l_b, as required by AISC Equations J10-2 through J10-7.

Try a stiffener width, b of 8 inches. The total combined width will be

$2(8) + 5/16$ (the web thickness) = 16.31 inches, or slightly less than the flange width of 18 inches. From Equation-28

$$(\frac{b}{t})_{s7} \leq 0.56\sqrt{\frac{E}{F_{yst}}}$$

$$t \geq \frac{b}{0.56}\sqrt{\frac{F_{yst}}{E}} = \frac{8}{0.56}\sqrt{\frac{36}{29{,}000}} = 0.503\ in.$$

Try two $3/4 \times 8$ stiffeners. Assume a 3/16 in. web to flange weld and a 1/2 in. cut-out in the stiffener. Check the stiffener at the support.

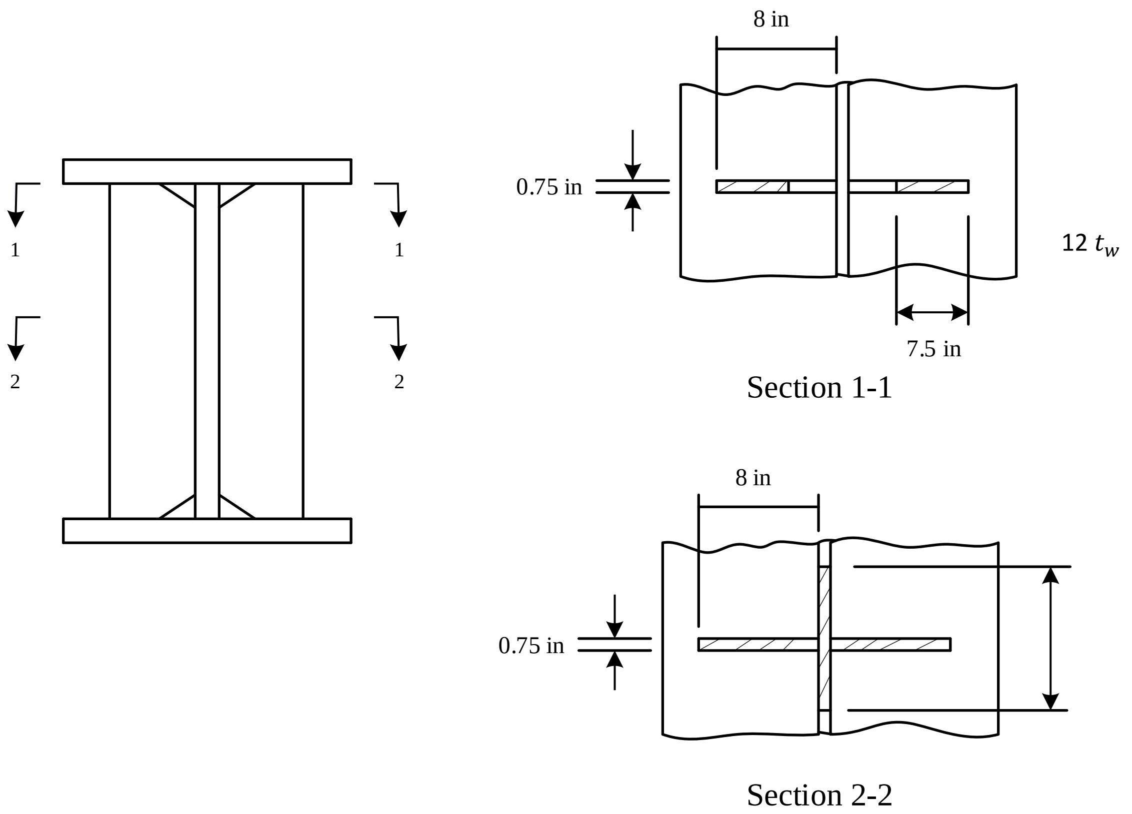


Figure 3.16: Bearing Stiffener at the Support

The bearing strength is

$R_n = 1.8F_yA_{pb} = 1.8(36)(0.75)(8 - 0.5) \times 2 = 729.0$ kips

$\varphi R_n = 0.75(729.0) = 547$ kips > 223.4 kips (OK)

Check the stiffener as a column.

The length of web acting with the stiffener plates to form a compression member is 12 times the web thickness for an end stiffener (AISC J10.8). As shown in Figure 3.16, this length is $12(5/16) = 3.75$ in. If we locate the stiffener centrally within this length, the point of support (location of the girder reaction) will be approximately $\frac{3.75}{2}$ =1.875 inches from the end of the girder. Use 3 inches, as shown in Figure 3.16, but base the computations on a total length of web of 3.75 inches, which gives

$$A = 2(8)\left(\frac{3}{4}\right) + \left(\frac{5}{16}\right)(3.75) = 13.17\ in.^2$$

$$I = \frac{3.75(5/16)^3}{12} + 2\,\frac{0.75(8)^3}{12} + 8\ \left(\frac{3}{4}\right)\left(4 + \frac{5}{32}\right)^2 = 271.3\ in.^4$$

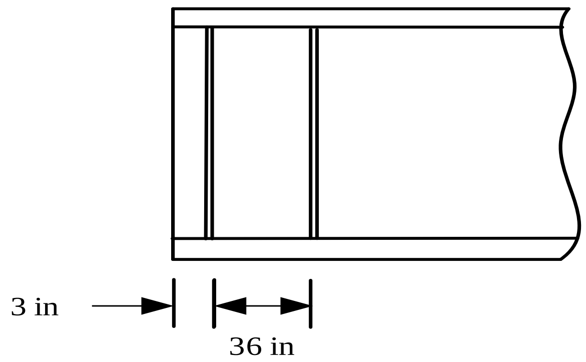


Figure 3.17: Side view of the plate girder

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{271.3}{13.17}} = 4.539\ \text{in.}$$

$$\frac{KL}{r} = \frac{Kh}{r} = \frac{0.7.5(62)}{4.539} = 10.24 < 25$$

$$P_n = F_y A_g = 36(13.17) = 474.1\ \text{kips}$$

$$\varphi P_n = 0.90(474.1) = 427\ \text{kips} > 223.4\ \text{kips (OK)}$$

Since the load at midspan is smaller than the reaction, use the same stiffener at midspan.

So, for bearing stiffeners use two plates of $\mathbf{3/4 \times 8 \times 5' - 2''}$

Connection Design

At this point, all components of the girder have been sized. The connections of these elements will now be designed. E70 electrodes, with a design strength of 1.392 kips/inch per sixteenth of an inch in size, will be used for all welds.

Flange to web welds

For the flange to web welds, compute the horizontal shear flow at the flange to web junction:

Maximum $V_u = 223.4$ kips

Q =flange area $\times \bar{y}$ (see Figure 3.11)

$= 1.5(18)(31.75) = 857.2\ in.^3$

$I_x = 60{,}640\ in.^4$

Maximum $\frac{V_u Q}{I_x} = \frac{223.4(857.2)}{60{,}640} = 3.158$ kips/in.

For the plate thicknesses being welded, the minimum weld size, w is $3/16$ in. If intermittent welds are used, their minimum length is

$L_{\text{min}} = 4 \times w \geq 1.5$ in.

$= 4\left(\frac{3}{16}\right) = 0.75$ in. ∴ Use 1.5 in.

Try 3/16-in. × $1\,{}^1/_2$ -in. fillet welds:

Weld shear strength: φR_n = 0.75(0.707wF_{nw}) (3.34)

Where, F_{nw} in a fillet weld is 0.6 times the tensile strength of the weld metal

∴ Weld shear strength per sixteenth of an inch: φR_n = 0.75(0.707wF_{nw})

= 0.75 (0.707) (1/16) (0.6×70) =1.392 kips/inch

∴ Capacity per inch $= 1.392 \times 3$ sixteenths $\times 2$ welds $= 8.352$ kips/in.

Check the capacity of the base metal (From AISC manual equations J4-3 and J4-4). The web is the thinner of the connected parts and controls.

The shear yield design strength per unit length is

$\varphi R_n = 1.0(0.6F_y t) = 0.6F_y t$ for a one-inch length (3.35)

$\varphi R_n = 0.6F_y t = 0.6(36)\left(\frac{5}{16}\right) = 6.750 kips/in.$

The base metal shear rupture strength per unit length is

$\varphi R_n = 0.45F_u t$ (3.36)

$= 0.45(58)\left(\frac{5}{16}\right) = 8.156$ kips/in.

The base metal shear strength is therefore 6.750 kips/in. $<$ 8.352 kips/in.

Use a total weld capacity of 6.750 kips/in. The capacity of a 1.5 in. length of a pair of welds is

6.750(1.5) = 10.13 kips

To determine the spacing, let

$$\frac{10.13}{s} = \frac{V_u Q}{I_x}$$

where s is the center-to-center spacing of the welds in inches and

$$s = \frac{10.13}{V_u Q/I_x} = \frac{10.13}{3.158} = 3.21 \text{ in.}$$

Using a center-to-center spacing of 3 inches will give a clear spacing of $3 - 1.5 = 1.5$ inches. The AISC Specification refers to spacing of intermittent fillet welds in Section F13 and Section E6.

$d \leq 0.75\sqrt{\frac{E}{F_y}}t_f$, but no greater than 12 in.

Adapting these limits to the present case yields

$$0.75\sqrt{\frac{E}{F_y}}t_f = 0.75\sqrt{\frac{29{,}000}{36}}(1.5) = 31.9 \text{ in.} > 12\text{in.}$$

The maximum permissible clear spacing is therefore 12 inches, and the required clear spacing of 1.5 inches is satisfactory.

There is no minimum spacing given in the Specification, but the AISC publication, "Detailing for Steel Construction," (AISC, 2009) states that intermittent welds are more economical than continuous welds only if the center to center spacing is more than twice the length of the weld. In this example, the spacing is equal to twice the length, so either type could be used.

Although the 3-inch center to center spacing can be used for the entire length of the girder, an increased spacing can be used where the shear is less than the maximum of 223.4 kips. Three different spacings will be investigated:

1. The closest required spacing of 3 inches.

2. The maximum permissible center to center spacing of $12 + 1.5 = 13.5$ in.

3. An intermediate spacing of 5 inches.

The 5-inch spacing can be used when

$$\frac{V_u Q}{I_x} = \frac{10.13}{s} \text{ or } V_u = \frac{10.13 I_x}{Qs} = \frac{10.13(60{,}640)}{857.2(5)} = 143.3 \text{ kips}$$

Refer to Figure 3.12 and let x be the distance from the left support, giving

$$V_u = 223.4 - 4.34x = 143.3 \text{ kips}$$

$$\therefore x = 18.46 \text{ ft}$$

The 13.5-inch spacing can be used when

$$V_u = \frac{10.13 I_x}{Qs} = \frac{10.13(60{,}640)}{857.2(13.5)} = 53.08 \text{ kips}$$

Figure 3.12, shows that the shear never gets this small, so the maximum spacing never controls.

Use 3/16-in. × $1\,{}^{1}\!/_{2}$ in. fillet welds for the flange to web welds, spaced as shown in Figure 3.18.

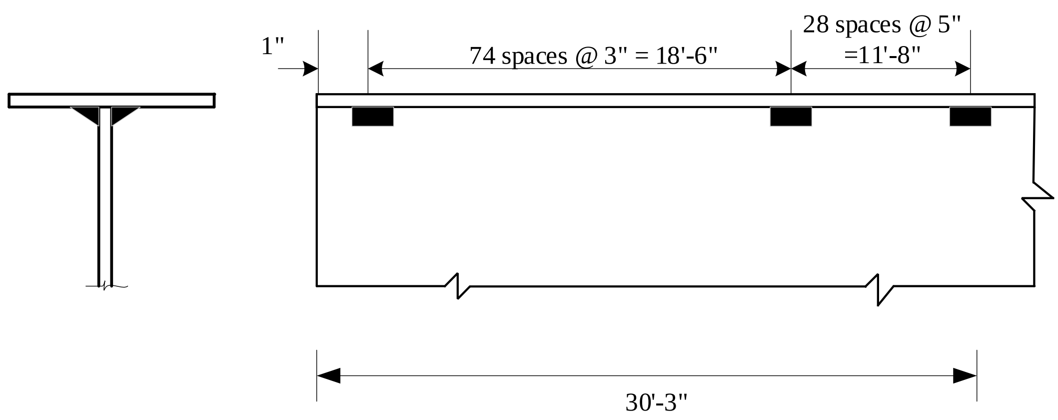


Figure 3.18: Weld between Flange and web

For the intermediate stiffener welds:

Minimum weld size = $\frac{3}{16}$ in. (based on the web thickness of $\frac{5}{16}$ in.)

Minimum length = $4(\frac{3}{16}) = 0.75$ in. $<$ 1.5 in. Use 1.5 in.

Use two welds per stiffener for a total of four. The capacity per inch for two $3/16$ in. fillet welds per stiffener plate is

$1.392 \times 3 \times 2 = 8.352$ kips/in.

Check the shear strength of the stiffener (the thinner of the two connected parts).

From Equation-35, the shear yield strength per unit length is

$\varphi R_n = 0.6 F_y t = 0.6(36)(\frac{1}{4}) = 5.400$ kips/in.

From Equation-36, the base metal shear rupture strength per unit length is

$\varphi R_n = 0.45 F_u t = 045(58)(\frac{1}{4}) = 6.525$ kips/in.

The base metal shear strength is therefore 5.400 kips/in. per stiffener. This is less than the shear strength of two welds (using two welds for each plate), so use a weld strength of 5.400 kips/in. For the two plates (four welds), use

$5.400 \times 2 = 10.80$ kips/in.

Proportioning the intermediate stiffeners by the AISC rules does not require the computation of any forces, but a force must be transmitted from the stiffener to the web, and the connection should be designed for this force. Basler (1961) recommends the use of a shear flow of

$$f = 0.045h\sqrt{\frac{F_y^3}{E}}\ \text{kips/in.} \tag{3.37}$$

$$f = 0.045h\sqrt{\frac{F_y^3}{E}} = 0.045(62)\sqrt{\frac{(36)^3}{29{,}000}} = 3.539\ \text{kips/in.}$$

Use intermittent welds. The capacity of a 1.5 in. length of the 4 welds is 1.5(10.80) =16.20 kips

Equating the shear strength per inch and the required strength gives

$\frac{16.20}{s} = 3.539$ kips/in. or $s = 4.58$ in.

From AISC G2.2, the maximum clear spacing is 16 times the web thickness but no greater than 10 inches, or

$16\ t_w = 16(\frac{5}{16}) = 5$ in.

Use a center to center spacing of $4\,{}^{1}/_{2}$ inches, resulting in a clear spacing of

$4.5 - 1.5 = 3$ in. < 5 in. (OK)

Use 3/16-in. $\times$ $1\,{}^{1}/_{2}$-in. fillet welds for intermediate stiffeners, spaced as shown in Figure 3.19.

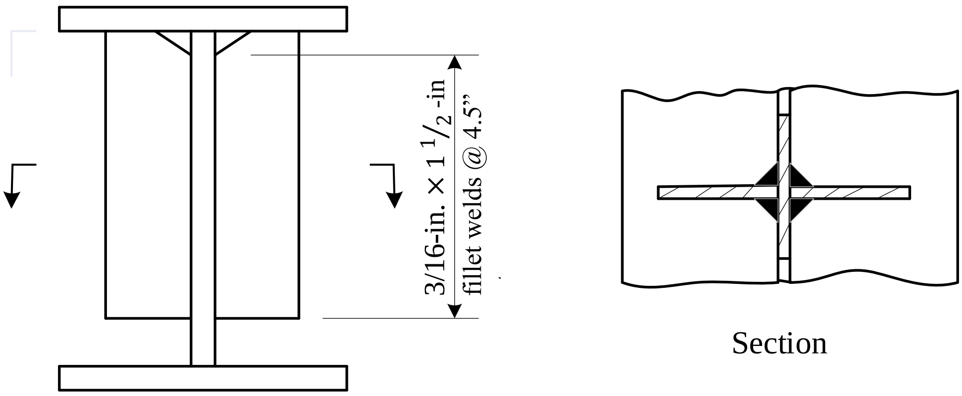


Figure 3.19: Weld between web and intermediate stiffener

For the bearing stiffener welds:

Minimum weld size $= \frac{3}{16}$ in. (based on the thickness $t_w = \frac{5}{16}$ in.)

Minimum length $= 4(\frac{3}{16}) = 0.75$ in. < 1.5 in. Use 1.5 in.

Use two welds per stiffener for a total of four. The capacity per inch for two $3/16$ - inch fillet welds per stiffener plate is

$1.392 \times 3 \times 2 = 8.352$ kips/in.

Check the shear strength of the web. From the design of the flange to web welds, the base metal shear strength is 6.750 kips/in. per stiffener. This is less than the shear strength of two welds (using two welds for each plate), so use a weld strength of 6.750 kips/in.

For the two plates (four welds), use $6.750 \times 2 = 13.50$ kips/in.

The capacity of a 1.5 in. length of four welds is

$1.5(13.50) = 20.25$ kips

For the end bearing stiffener, the applied load per inch is

$$\frac{\text{Reaction}}{\text{Length avai1able for weld}} = \frac{223.4}{62\text{-}2(0.5)} = 3.662 \text{ kips/in}$$

From $20.\ 25/s = 3.662 \therefore s = 5.53$ inches. (Note that a smaller weld spacing is required for the intermediate stiffeners)

Use 3/16-in.$\times$ $1\,^1/_2$-in. fillet welds for all bearing stiffeners, spaced as shown in Figure 3.20.

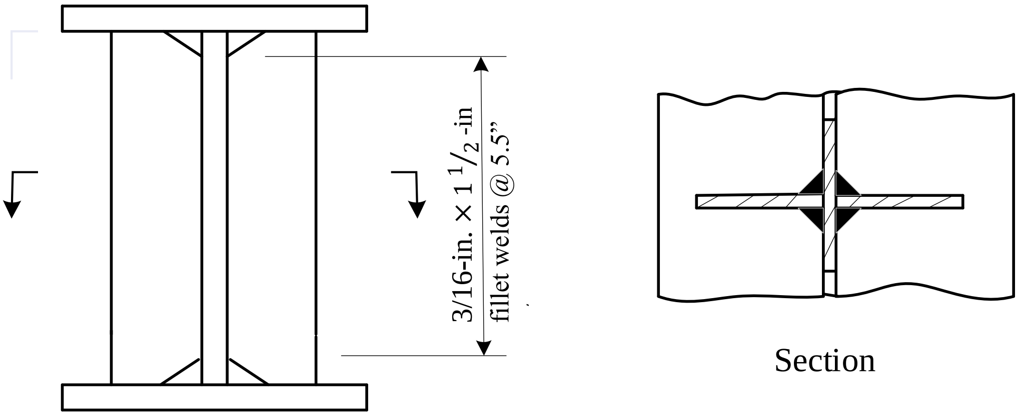


Figure 3.20: Weld between web and bearing stiffener

References:

- American Institute of Steel Construction. 2011a. Steel Construction Manual. 14th ed. Chicago.
- Old trails arch bridge [Online Image], Retrieved December 17, 2017, from https://bridgehunter.com/ca/san-bernardino/old-trails-arch/
- Segui, W.T., 2012. Steel design. Cengage Learning.
- Structural steel frame [Online Image], Retrieved December 28, 2017, from http://www.girder-slab.com/features.asp
- Structural steel shapes [Online Image], Retrieved December 17, 2017, from https://www.archtoolbox.com/materials-systems/metals/structsteelshapes.html
- Trenton through truss bridge [Online Image], 2008, Retrieved December 17, 2017, from https://bridgehunter.com/ut/cache/5034C/

Appendix.

AISC, Plate Sizes

Note to tables I &II: the first length given obtainable from most, and usually from all, of the mills rolling the given width. The second given is the maximum obtainable from any mill, and such lengths are subject to substantial extras. For plates of large sizes, Designers should consult fabricators regarding possibilities of fabrication, shipment and erection.

Table I: Plates (available sizes)

Length, in feet, of Universal Mill plates obtainable in the respective widths shown										
Thickness, inches	Width, Inches									
	6-12	13-20	21-26	27-30	31-36	37-42	43-46	47-48	49-58	59-60
$\frac{1}{4}$	65-80	60-125	60-125	60-125	60-125	40-100	90-100	90-100	40-65	60-
$\frac{3}{8}$	65-80	60-125	60-125	60-125	60-125	60-125	90-125	90-125	80-90	70-
$\frac{1}{2}$	65-80	60-125	60-125	60-125	60-125	60-125	90-125	90-125	85-110	60-
$\frac{3}{4}$	60-80	60-125	60-125	60-125	60-125	55-125	90-125	90-125	80-120	40-
1	60-80	60-125	60-125	60-125	60-125	40	90-125	90-125	70-95	40-
$1\frac{1}{4}$	60-75	8-125	48-125	49-125	49-125	38-125	90-115	75-90	60-75	40-
$1\frac{1}{2}$	40-60	48-120	46-125	46-125	45-125	33-105	90-95	65-90	50-75	35-
$1\frac{3}{4}$	35-60	41-110	40-125	40-125	38-110	28-90	80-90	55-90	45-55	30-
2	30-60	36-90	35-125	35-110	34-95	24-75	70-90	45-90	40-45	25-

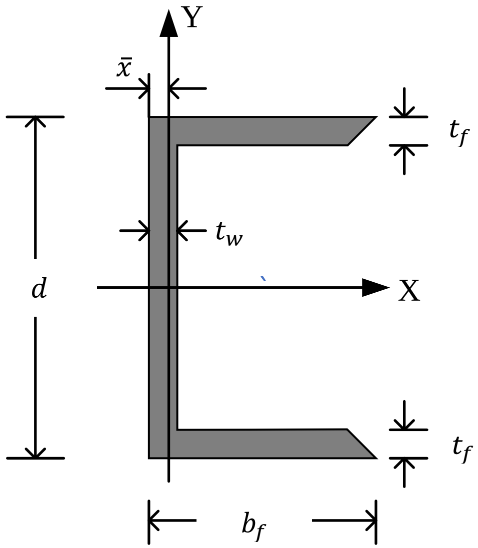
AISC, Plate Sizes

Table II: Plates (available sizes)

Length, in feet, of Universal Mill plates obtainable in the respective widths shown										
Thickness, inches	Width, Inches									
	24-36	37-48	49-60	61-78	79-96	97-114	115-132	133-150	151-168	169-186
$\frac{3}{4}$	40-45	40-50	40-50	35-55	30-48	27-38	21-30	×	×	×
$\frac{3}{8}$	38-50	40-70	40-70	35-70	30-65	30-52	26-48	17-30	24-	21-
$\frac{1}{2}$	36-50	40-70	40-70	35-70	30-70	30-55	36-50	20-37	33-	27-
$\frac{3}{4}$	36-50	37-70	35-70	35-70	30-70	30-55	35-48	19-45	45-	39-
1	36-50	34-70	30-70	32-70	25-66	25-53	35-48	18-45	45-	41-
$1\frac{1}{4}$	30-50	30-70	25-70	25-65	20-60	20-45	31-45	17-45	42-	38-
$1\frac{1}{2}$	25-40	30-70	23-60	21-60	16-56	15-45	30-45	16-42	41-	33-
$1\frac{3}{4}$	25-40	30-60	22-52	18-59	14-50	12-45	28-44	15-42	40-	31-
2	20-35	25-55	20-49	16-52	13-47	11-45	24-43	14-42	39-	29-

Source: American Institute of steel construction (AISC), 9th edition, Page 59.

American Standard Channel Section (C- Section) Properties



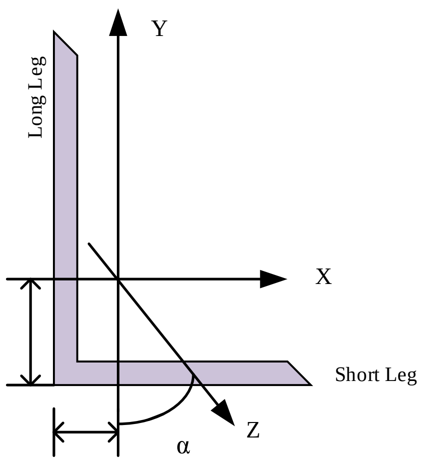
Designation	Nominal weight per foot $lb.$	Area A (in^2)	Depth d (in)	Flange width b_f (in)	Web thickness t_w (in)	Flange thickness t_f (in)	$\bar{x}$ (in)	Axis X-X			Axis Y-Y		
								I_x (in^4)	S_x (in^3)	r_x (in)	I_y (in^4)	S_y (in^3)	r_y (in)
C15×50	50.0	14.7	15.0	3.72	0.716	0.650	0.799	404	53.8	5.24	11.0	3.77	0.865
C15×40	40.0	11.8	15.0	3.52	0.520	0.650	0.778	348	46.5	5.43	9.17	3.34	0.883
C15×33.9	33.9	10.0	15.0	3.40	0.400	0.650	0.788	315	42.0	5.61	8.07	3.09	0.901
C12×30	30.0	8.81	12.0	3.17	0.510	0.501	0.674	162	27.0	4.29	5.12	2.05	0.762
C12×25	25.0	7.34	12.0	3.05	0.387	0.501	0.674	144	24.0	4.43	4.45	1.87	0.779
C12×20.7	20.7	6.08	12.0	2.94	0.282	0.501	0.698	129	21.5	4.61	3.86	1.72	0.797
C10×30	30.0	8.81	10.0	3.03	0.673	0.436	0.649	103	20.7	3.43	3.93	1.65	0.668

Source: AISC Shape Database, 14th edition

Designation	Nominal weight per foot $lb.$	Area A (in^2)	Depth d (in)	Flange width b_f (in)	Web thickness t_w (in)	Flange thickness t_f (in)	$\bar{x}$ (in)	Axis X-X			Axis Y-Y		
								I_x (in^4)	S_x (in^3)	r_x (in)	I_y (in^4)	S_y (in^3)	r_y (in)
C10×25	25.0	7.35	10.0	2.89	0.526	0.436	0.617	91.1	18.2	3.52	3.34	1.47	0.675
C10×20	20.0	5.87	10.0	2.74	0.379	0.436	0.606	78.9	15.8	3.67	2.80	1.31	0.690
C10×15.3	15.3	4.48	10.0	2.60	0.240	0.436	0.634	67.3	13.5	3.88	2.27	1.15	0.711
C9×20	20.0	5.87	9.00	2.65	0.448	0.413	0.583	60.9	13.5	3.22	2.41	1.17	0.640
C9×15	15.0	4.40	9.00	2.49	0.285	0.413	0.586	51.0	11.3	3.40	1.91	1.01	0.659
C9×13.4	13.4	3.94	9.00	2.43	0.233	0.413	0.601	47.8	10.6	3.48	1.75	0.954	0.666
C8×18.75	18.75	5.51	8.00	2.53	0.487	0.390	0.565	43.9	11.0	2.82	1.97	1.01	0.598
C8×13.75	13.75	4.03	8.00	2.34	0.303	0.390	0.554	36.1	9.02	2.99	1.52	0.848	0.613
C8×11.5	11.5	3.37	8.00	2.26	0.220	0.390	0.572	32.5	8.14	3.11	1.31	0.775	0.623
C7×14.75	14.75	4.33	7.00	2.30	0.419	0.366	0.532	27.2	7.78	2.51	1.37	0.772	0.561
C7×12.25	12.25	3.59	7.00	2.19	0.314	0.366	0.525	24.2	6.92	2.59	1.16	0.696	0.568
C7×9.8	9.80	2.87	7.00	2.09	0.210	0.366	0.541	21.2	6.07	2.72	0.957	0.617	0.578
C6×13	13.0	3.82	6.00	2.16	0.437	0.343	0.514	17.3	5.78	2.13	1.05	0.638	0.524
C6×10.5	10.5	3.07	6.00	2.03	0.314	0.343	0.500	15.1	5.04	2.22	0.860	0.561	0.529
C6×8.2	8.20	2.39	6.00	1.92	0.200	0.343	0.512	13.1	4.35	2.34	0.687	0.488	0.536
C5×9	9.00	2.64	5.00	1.89	0.325	0.320	0.478	8.89	3.56	1.84	0.624	0.444	0.486
C5×6.7	6.70	1.97	5.00	1.75	0.190	0.320	0.484	7.48	2.99	1.95	0.470	0.372	0.489
C4×7.25	7.25	2.13	4.00	1.72	0.321	0.296	0.459	4.58	2.29	1.47	0.425	0.337	0.447
C4×6.25	6.25	1.77	4.00	1.65	0.247	0.272	0.435	4.00	2.00	1.50	0.345	0.284	0.441
C4×5.4	5.40	1.58	4.00	1.58	0.184	0.296	0.457	3.85	1.92	1.56	0.312	0.277	0.444
C4×4.5	4.50	1.38	4.00	1.58	0.125	0.296	0.493	3.65	1.83	1.63	0.289	0.265	0.457
C3×6	6.00	1.76	3.00	1.60	0.356	0.273	0.455	2.07	1.38	1.09	0.300	0.263	0.413
C3×5	5.00	1.47	3.00	1.50	0.258	0.273	0.439	1.85	1.23	1.12	0.241	0.228	0.405
C3×4.1	4.10	1.20	3.00	1.41	0.170	0.273	0.437	1.65	1.10	1.18	0.191	0.196	0.398
C3×3.5	3.50	1.09	3.00	1.37	0.132	0.273	0.443	1.57	1.04	1.20	0.169	0.182	0.394

Source: AISC Shape Database, 14th edition

American Standard Angle Section (L- Section) Properties



Designation	Nominal weight per foot $lb.$	Area A (in^2)	$\bar{x}$ (in)	$\bar{y}$ (in)	Axis X-X			Axis Y-Y			Axis Z-Z	
					I_x (in^4)	S_x (in^3)	r_x (in)	I_y (in^4)	S_y (in^3)	r_y (in)	r_z (in)	$\tan\alpha$
L8×8×1-1/8	56.9	16.8	2.40	2.40	98.1	17.5	2.41	98.1	17.5	2.41	1.56	1.00
L8×8×1	51.0	15.1	2.36	2.36	89.1	15.8	2.43	89.1	15.8	2.43	1.56	1.00
L8×8×7/8	45.0	13.3	2.31	2.31	79.7	14.0	2.45	79.7	14.0	2.45	1.57	1.00
L8×8×3/4	38.9	11.5	2.26	2.26	69.9	12.2	2.46	69.9	12.2	2.46	1.57	1.00
L8×8×5/8	32.7	9.69	2.21	2.21	59.6	10.3	2.48	59.6	10.3	2.48	1.58	1.00
L8×8×9/16	29.6	8.77	2.19	2.19	54.2	9.33	2.49	54.2	9.33	2.49	1.58	1.00
L8×8×1/2	26.4	7.84	2.17	2.17	48.8	8.36	2.49	48.8	8.36	2.49	1.59	1.00
L8×6×1	44.2	13.1	1.65	2.65	80.9	15.1	2.49	38.8	8.92	1.72	1.28	0.542
L8×6×7/8	39.1	11.5	1.60	2.60	72.4	13.4	2.50	34.9	7.94	1.74	1.28	0.546
L8×6×3/4	33.8	9.99	1.56	2.55	63.5	11.7	2.52	30.8	6.92	1.75	1.29	0.550
L8×6×5/8	28.5	8.41	1.51	2.50	54.2	9.86	2.54	26.4	5.88	1.77	1.29	0.554
L8×6×9/16	25.7	7.61	1.49	2.48	49.4	8.94	2.55	24.1	5.34	1.78	1.30	0.556

Source: AISC Shape Database, 14th edition

Designation	Nominal weight per foot $lb.$	Area A (in^2)	$\bar{x}$ (in)	$\bar{y}$ (in)	Axis X-X			Axis Y-Y			Axis Z-Z	
					I_x (in^4)	S_x (in^3)	r_x (in)	I_y (in^4)	S_y (in^3)	r_y (in)	r_z (in)	$\tan\alpha$
L8×6×1/2	23.0	6.80	1.46	2.46	44.4	8.01	2.55	21.7	4.79	1.79	1.30	0.557
L8×6×7/16	20.2	5.99	1.44	2.43	39.3	7.06	2.56	19.3	4.23	1.80	1.31	0.559
L8×4×1	37.4	11.1	1.04	3.03	69.7	14.0	2.51	11.6	3.94	1.03	0.844	0.247
L8×4×7/8	33.1	9.79	0.997	2.99	62.6	12.5	2.53	10.5	3.51	1.04	0.846	0.252
L8×4×3/4	28.7	8.49	0.949	2.94	55.0	10.9	2.55	9.37	3.07	1.05	0.850	0.257
L8×4×5/8	24.2	7.16	0.902	2.89	47.0	9.20	2.56	8.11	2.62	1.06	0.856	0.262
L8×4×9/16	21.9	6.49	0.878	2.86	42.9	8.34	2.57	7.44	2.38	1.07	0.859	0.264
L8×4×1/2	19.6	5.80	0.854	2.84	38.6	7.48	2.58	6.75	2.15	1.08	0.863	0.266
L8×4×7/16	17.2	5.11	0.829	2.81	34.2	6.59	2.59	6.03	1.90	1.09	0.867	0.268
L7×4×3/4	26.2	7.74	1.00	2.50	37.8	8.39	2.21	9.00	3.01	1.08	0.855	0.324
L7×4×5/8	22.1	6.50	0.958	2.45	32.4	7.12	2.23	7.79	2.56	1.10	0.860	0.329
L7×4×1/2	17.9	5.26	0.910	2.40	26.6	5.79	2.25	6.48	2.10	1.11	0.866	0.334
L7×4×7/16	15.7	4.63	0.886	2.38	23.6	5.11	2.26	5.79	1.86	1.12	0.869	0.337
L7×4×3/8	13.6	4.00	0.861	2.35	20.5	4.42	2.27	5.06	1.61	1.12	0.873	0.339
L6×6×1	37.4	11.0	1.86	1.86	35.4	8.55	1.79	35.4	8.55	1.79	1.17	1.00
L6×6×7/8	33.1	9.75	1.81	1.81	31.9	7.61	1.81	31.9	7.61	1.81	1.17	1.00
L6×6×3/4	28.7	8.46	1.77	1.77	28.1	6.64	1.82	28.1	6.64	1.82	1.17	1.00
L6×6×5/8	24.2	7.13	1.72	1.72	24.1	5.64	1.84	24.1	5.64	1.84	1.17	1.00
L6×6×9/16	21.9	6.45	1.70	1.70	22.0	5.12	1.85	22.0	5.12	1.85	1.18	1.00
L6×6×1/2	19.6	5.77	1.67	1.67	19.9	4.59	1.86	19.9	4.59	1.86	1.18	1.00
L6×6×7/16	17.2	5.08	1.65	1.65	17.6	4.06	1.86	17.6	4.06	1.86	1.18	1.00
L6×6×3/8	14.9	4.38	1.62	1.62	15.4	3.51	1.87	15.4	3.51	1.87	1.19	1.00
L6×6×5/16	12.4	3.67	1.60	1.60	13.0	2.95	1.88	13.0	2.95	1.88	1.19	1.00
L6×4×7/8	27.2	8.00	1.12	2.12	27.7	7.13	1.86	9.70	3.37	1.10	0.854	0.421
L6×4×3/4	23.6	6.94	1.07	2.07	24.5	6.23	1.88	8.63	2.95	1.12	0.856	0.428
L6×4×5/8	20.0	5.86	1.03	2.03	21.0	5.29	1.89	7.48	2.52	1.13	0.859	0.435
L6×4×9/16	18.1	5.31	1.00	2.00	19.2	4.81	1.90	6.86	2.29	1.14	0.861	0.438

Source: AISC Shape Database, 14th edition

Designation	Nominal weight per foot $lb.$	Area A (in^2)	$\bar{x}$ (in)	$\bar{y}$ (in)	Axis X-X			Axis Y-Y			Axis Z-Z	
					I_x (in^4)	S_x (in^3)	r_x (in)	I_y (in^4)	S_y (in^3)	r_y (in)	r_z (in)	$\tan\alpha$
L6×4×1/2	16.2	4.75	0.981	1.98	17.3	4.31	1.91	6.22	2.06	1.14	0.864	0.440
L6×4×7/16	14.3	4.18	0.957	1.95	15.4	3.81	1.92	5.56	1.83	1.15	0.867	0.443
L6×4×3/8	12.3	3.61	0.933	1.93	13.4	3.30	1.93	4.86	1.58	1.16	0.870	0.446
L6×4×5/16	10.3	3.03	0.908	1.90	11.4	2.77	1.94	4.13	1.34	1.17	0.874	0.449
L6×3-1/2×1/2	15.3	4.50	0.829	2.07	16.6	4.23	1.92	4.24	1.59	0.968	0.756	0.343
L6×3-1/2×3/8	11.7	3.44	0.781	2.02	12.9	3.23	1.93	3.33	1.22	0.984	0.763	0.349
L6×3-1/2×5/16	9.80	2.89	0.756	2.00	10.9	2.72	1.94	2.84	1.03	0.991	0.767	0.352
L5×5×7/8	27.2	8.00	1.56	1.56	17.8	5.16	1.49	17.8	5.16	1.49	0.971	1.00
L5×5×3/4	23.6	6.98	1.52	1.52	15.7	4.52	1.50	15.7	4.52	1.50	0.972	1.00
L5×5×5/8	20.0	5.90	1.47	1.47	13.6	3.85	1.52	13.6	3.85	1.52	0.975	1.00
L5×5×1/2	16.2	4.79	1.42	1.42	11.3	3.15	1.53	11.3	3.15	1.53	0.980	1.00
L5×5×7/16	14.3	4.22	1.40	1.40	10.0	2.78	1.54	10.0	2.78	1.54	0.983	1.00
L5×5×3/8	12.3	3.65	1.37	1.37	8.76	2.41	1.55	8.76	2.41	1.55	0.986	1.00
L5×5×5/16	10.3	3.07	1.35	1.35	7.44	2.04	1.56	7.44	2.04	1.56	0.990	1.00
L5×3-1/2×3/4	19.8	5.85	0.993	1.74	13.9	4.26	1.55	5.52	2.20	0.974	0.744	0.464
L5×3-1/2×5/8	16.8	4.93	0.947	1.69	12.0	3.63	1.56	4.80	1.88	0.987	0.746	0.472
L5×3-1/2×1/2	13.6	4.00	0.901	1.65	10.0	2.97	1.58	4.02	1.55	1.00	0.750	0.479
L5×3-1/2×3/8	10.4	3.05	0.854	1.60	7.75	2.28	1.59	3.15	1.19	1.02	0.755	0.485
L5×3-1/2×5/16	8.70	2.56	0.829	1.57	6.58	1.92	1.60	2.69	1.01	1.02	0.758	0.489
L5×3-1/2×1/4	7.00	2.07	0.804	1.55	5.36	1.55	1.61	2.20	0.816	1.03	0.761	0.491
L5×3×1/2	12.8	3.75	0.746	1.74	9.43	2.89	1.58	2.55	1.13	0.824	0.642	0.357
L5×3×7/16	11.3	3.31	0.722	1.72	8.41	2.56	1.59	2.29	1.00	0.831	0.644	0.361
L5×3×3/8	9.80	2.86	0.698	1.69	7.35	2.22	1.60	2.01	0.874	0.838	0.646	0.364
L5×3×5/16	8.20	2.41	0.673	1.67	6.24	1.87	1.61	1.72	0.739	0.846	0.649	0.368
L5×3×1/4	6.60	1.94	0.648	1.64	5.09	1.51	1.62	1.41	0.600	0.853	0.652	0.371
L4×4×3/4	18.5	5.44	1.27	1.27	7.62	2.79	1.18	7.62	2.79	1.18	0.774	1.00
L4×4×5/8	15.7	4.61	1.22	1.22	6.62	2.38	1.20	6.62	2.38	1.20	0.774	1.00

Source: AISC Shape Database, 14th edition

Designation	Nominal weight per foot $lb.$	Area A (in^2)	$\bar{x}$ (in)	$\bar{y}$ (in)	Axis X-X			Axis Y-Y			Axis Z-Z	
					I_x (in^4)	S_x (in^3)	r_x (in)	I_y (in^4)	S_y (in^3)	r_y (in)	r_z (in)	$\tan\alpha$
L4×4×1/2	12.8	3.75	1.18	1.18	5.52	1.96	1.21	5.52	1.96	1.21	0.776	1.00
L4×4×7/16	11.3	3.30	1.15	1.15	4.93	1.73	1.22	4.93	1.73	1.22	0.777	1.00
L4×4×3/8	9.80	2.86	1.13	1.13	4.32	1.50	1.23	4.32	1.50	1.23	0.779	1.00
L4×4×5/16	8.20	2.40	1.11	1.11	3.67	1.27	1.24	3.67	1.27	1.24	0.781	1.00
L4×4×1/4	6.60	1.93	1.08	1.08	3.00	1.03	1.25	3.00	1.03	1.25	0.783	1.00
L4×3-1/2×1/2	11.9	3.50	0.994	1.24	5.30	1.92	1.23	3.76	1.50	1.04	0.716	0.750
L4×3-1/2×3/8	9.10	2.68	0.947	1.20	4.15	1.48	1.25	2.96	1.16	1.05	0.719	0.755
L4×3-1/2×5/16	7.70	2.25	0.923	1.17	3.53	1.25	1.25	2.52	0.980	1.06	0.721	0.757
L4×3-1/2×1/4	6.20	1.82	0.897	1.14	2.89	1.01	1.26	2.07	0.794	1.07	0.723	0.759
L4×3×5/8	13.6	3.99	0.867	1.37	6.01	2.28	1.23	2.85	1.34	0.845	0.631	0.534
L4×3×1/2	11.1	3.25	0.822	1.32	5.02	1.87	1.24	2.40	1.10	0.858	0.633	0.542
L4×3×3/8	8.50	2.49	0.775	1.27	3.94	1.44	1.26	1.89	0.851	0.873	0.636	0.551
L4×3×5/16	7.20	2.09	0.750	1.25	3.36	1.22	1.27	1.62	0.721	0.880	0.638	0.554
L4×3×1/4	5.80	1.69	0.725	1.22	2.75	0.988	1.27	1.33	0.585	0.887	0.639	0.558
L3-1/2×3-1/2×1/2	11.1	3.25	1.05	1.05	3.63	1.48	1.05	3.63	1.48	1.05	0.679	1.00
L3-1/2×3-1/2×7/16	9.80	2.89	1.03	1.03	3.25	1.32	1.06	3.25	1.32	1.06	0.681	1.00
L3-1/2×3-1/2×3/8	8.50	2.50	1.00	1.00	2.86	1.15	1.07	2.86	1.15	1.07	0.683	1.00
L3-1/2×3-1/2×5/16	7.20	2.10	0.979	0.979	2.44	0.969	1.08	2.44	0.969	1.08	0.685	1.00
L3-1/2×3-1/2×1/4	5.80	1.70	0.954	0.954	2.00	0.787	1.09	2.00	0.787	1.09	0.688	1.00
L3-1/2×3×1/2	10.2	3.02	0.869	1.12	3.45	1.45	1.07	2.32	1.09	0.877	0.618	0.713
L3-1/2×3×7/16	9.10	2.67	0.846	1.09	3.10	1.29	1.08	2.09	0.971	0.885	0.620	0.717
L3-1/2×3×3/8	7.90	2.32	0.823	1.07	2.73	1.12	1.09	1.84	0.847	0.892	0.622	0.720
L3-1/2×3×5/16	6.60	1.95	0.798	1.05	2.33	0.951	1.09	1.58	0.718	0.900	0.624	0.722
L3-1/2×3×1/4	5.40	1.58	0.773	1.02	1.92	0.773	1.10	1.30	0.585	0.908	0.628	0.725
L3-1/2×2-1/2×1/2	9.40	2.77	0.701	1.20	3.24	1.41	1.08	1.36	0.756	0.701	0.532	0.485
L3-1/2×2-1/2×3/8	7.20	2.12	0.655	1.15	2.56	1.09	1.10	1.09	0.589	0.716	0.535	0.495
L3-1/2×2-1/2×5/16	6.10	1.79	0.632	1.13	2.20	0.925	1.11	0.937	0.501	0.723	0.538	0.500

Source: AISC Shape Database, 14th edition

Designation	Nominal weight per foot $lb.$	Area A (in^2)	$\bar{x}$ (in)	$\bar{y}$ (in)	Axis X-X			Axis Y-Y			Axis Z-Z	
					I_x (in^4)	S_x (in^3)	r_x (in)	I_y (in^4)	S_y (in^3)	r_y (in)	r_z (in)	$\tan\alpha$
L3-1/2×2-1/2×1/4	4.90	1.45	0.607	1.10	1.81	0.753	1.12	0.775	0.410	0.731	0.541	0.504
L3×3×1/2	9.40	2.76	0.929	0.929	2.20	1.06	0.895	2.20	1.06	0.895	0.580	1.00
L3×3×7/16	8.30	2.43	0.907	0.907	1.98	0.946	0.903	1.98	0.946	0.903	0.580	1.00
L3×3×3/8	7.20	2.11	0.884	0.884	1.75	0.825	0.910	1.75	0.825	0.910	0.581	1.00
L3×3×5/16	6.10	1.78	0.860	0.860	1.50	0.699	0.918	1.50	0.699	0.918	0.583	1.00
L3×3×1/4	4.90	1.44	0.836	0.836	1.23	0.569	0.926	1.23	0.569	0.926	0.585	1.00
L3×3×3/16	3.71	1.09	0.812	0.812	0.948	0.433	0.933	0.948	0.433	0.933	0.586	1.00
L3×2-1/2×1/2	8.50	2.50	0.746	0.995	2.07	1.03	0.910	1.29	0.736	0.718	0.516	0.666
L3×2-1/2×7/16	7.60	2.22	0.724	0.972	1.87	0.921	0.917	1.17	0.656	0.724	0.516	0.671
L3×2-1/2×3/8	6.60	1.93	0.701	0.949	1.65	0.803	0.924	1.03	0.573	0.731	0.517	0.675
L3×2-1/2×5/16	5.60	1.63	0.677	0.925	1.41	0.681	0.932	0.888	0.487	0.739	0.518	0.679
L3×2-1/2×1/4	4.50	1.32	0.653	0.900	1.16	0.555	0.940	0.734	0.397	0.746	0.520	0.683
L3×2-1/2×3/16	3.39	1.00	0.627	0.874	0.899	0.423	0.947	0.568	0.303	0.753	0.521	0.687
L3×2×1/2	7.70	2.26	0.580	1.08	1.92	1.00	0.922	0.667	0.470	0.543	0.425	0.413
L3×2×3/8	5.90	1.75	0.535	1.03	1.54	0.779	0.937	0.539	0.368	0.555	0.426	0.426
L3×2×5/16	5.00	1.48	0.511	1.01	1.32	0.662	0.945	0.467	0.314	0.562	0.428	0.432
L3×2×1/4	4.10	1.20	0.487	0.980	1.09	0.541	0.953	0.390	0.258	0.569	0.431	0.437
L3×2×3/16	3.07	0.917	0.462	0.952	0.847	0.414	0.961	0.305	0.198	0.577	0.435	0.442
L2-1/2×2-1/2×1/2	7.70	2.26	0.803	0.803	1.22	0.716	0.735	1.22	0.716	0.735	0.481	1.00
L2-1/2×2-1/2×3/8	5.90	1.73	0.758	0.758	0.972	0.558	0.749	0.972	0.558	0.749	0.481	1.00
L2-1/2×2-1/2×5/16	5.00	1.46	0.735	0.735	0.837	0.474	0.756	0.837	0.474	0.756	0.481	1.00
L2-1/2×2-1/2×1/4	4.10	1.19	0.711	0.711	0.692	0.387	0.764	0.692	0.387	0.764	0.482	1.00
L2-1/2×2-1/2×3/16	3.07	0.901	0.687	0.687	0.535	0.295	0.771	0.535	0.295	0.771	0.482	1.00
L2-1/2×2×3/8	5.30	1.55	0.578	0.826	0.914	0.546	0.766	0.513	0.361	0.574	0.419	0.612
L2-1/2×2×5/16	4.50	1.32	0.555	0.803	0.790	0.465	0.774	0.446	0.309	0.581	0.420	0.618
L2-1/2×2×1/4	3.62	1.07	0.532	0.779	0.656	0.381	0.782	0.372	0.253	0.589	0.423	0.624
L2-1/2×2×3/16	2.75	0.818	0.508	0.754	0.511	0.293	0.790	0.292	0.195	0.597	0.426	0.628

Source: AISC Shape Database, 14th edition

Designation	Nominal weight per foot $lb.$	Area A (in^2)	$\bar{x}$ (in)	$\bar{y}$ (in)	Axis X-X			Axis Y-Y			Axis Z-Z	
					I_x (in^4)	S_x (in^3)	r_x (in)	I_y (in^4)	S_y (in^3)	r_y (in)	r_z (in)	$\tan\alpha$
L2-1/2×1-1/2×1/4	3.19	0.947	0.372	0.866	0.594	0.364	0.792	0.160	0.142	0.411	0.321	0.354
L2-1/2×1-1/2×3/16	2.44	0.724	0.347	0.839	0.464	0.280	0.801	0.126	0.110	0.418	0.324	0.360
L2×2×3/8	4.70	1.37	0.632	0.632	0.476	0.348	0.591	0.476	0.348	0.591	0.386	1.00
L2×2×5/16	3.92	1.16	0.609	0.609	0.414	0.298	0.598	0.414	0.298	0.598	0.386	1.00
L2×2×1/4	3.19	0.944	0.586	0.586	0.346	0.244	0.605	0.346	0.244	0.605	0.387	1.00
L2×2×3/16	2.44	0.722	0.561	0.561	0.271	0.188	0.612	0.271	0.188	0.612	0.389	1.00
L2×2×1/8	1.65	0.491	0.534	0.534	0.189	0.129	0.620	0.189	0.129	0.620	0.391	1.00

Source: AISC Shape Database, 14th edition

Basic Wind Speeds for Selected Locations in Bangladesh
(BNBC-1993 Table 6.2.8)

Location	Basic Wind Speed (km/h)	Location	**Basic Wind Speed (km/h)**
Angarpota	150	Lalmonirhat	204
Bagerhat	252	Madaripur	220
Bandarban	200	Magura	208
Barguna	260	Manikganj	185
Barisal	256	Meherpur	185
Bhola	225	Maheshkhali	260
Bogra	198	Moulvibazar	168
Brahmanbaria	180	Munshiganj	184
Chandpur	160	Mymensingh	217
Chapai Nawabganj	130	Naogaon	175
Chittagong	260	Narail	222
Chuadanga	198	Narayanganj	195
Comilla	196	Narsinghdi	190
Cox’s Bazar	260	Natore	198
Dahagram	150	Netrokona	210
Dhaka	210	Nilphamari	140
Dinajpur	130	Noakhali	184
Faridpur	202	Pabna	202
Feni	205	Panchagarh	130
Gaibandha	210	Patuakhali	260
Gazipur	215	Pirojpur	260
Gopalganj	242	Rajbari	188
Habiganj	172	Rajshahi	155
Hatiya	260	Rangamati	180
Ishurdi	225	Rangpur	209
Joypurhat	180	Satkhira	183
Jamalpur	180	Shariatpur	198
Jessore	205	Sherpur	200
Jhalakati	260	Sirajganj	160
Jhenaidah	208	Srimangal	160
Khagrachhari	180	St. Martin’s Island	260
Khulna	238	Sunamganj	195
Kutubdia	260	Sylhet	195
Kishoreganj	207	Sandwip	260
Kurigram	210	Tangail	160
Kushtia	215	Teknaf	260
Lakshmipur	162	Thakurgaon	130

Structure Importance Coefficients, C_I for Wind Loads (BNBC-1993 Table 6.2.9)

Structure Importance Category	Structure Importance Coefficient, C_I
Essential facilities	1.25
Hazardous facilities	1.25
Special occupancy structures	1.00
Standard occupancy structures	1.00
Low-risk structures	0.80

Exposure (Defined by BNBC-1993)

Exposure: This refers to the conditions of the terrain surrounding the building site. The terrain exposure in which a building or structure is to be sited shall be assessed as being one of the following categories:

- **Exposure A:** Urban and sub-urban areas, industrial area, wooded areas, hilly or other terrain covering at least 20% of the area with obstructions of 6 meters or more in height and extending from the site at least 500 meters or 10 times the height of the structure, whichever is greater.
- **Exposure B:** Open terrain with scattered obstructions having heights generally less than 10 meters extending 800 meters or more from the site in any full quadrant. This category includes air fields, open park lands, sparsely built-up outskirts of towns, flat open country and grasslands.
- **Exposure C:** Flat and un-obstructed open terrain, coastal areas and riversides facing large bodies of water, over 1.5 km or more in width. Exposure C extends inland from the shoreline 400 meter or 10 times the height of the structure, whichever is greater.

Combined Height and Exposure Coefficient, C_z

Height above ground level, z (meter)	Exposure A	Exposure B	Exposure C
0-4.5	0.368	0.801	1.196
6.0	0.415	0.866	1.263
9.0	0.497	0.972	1.370
12.0	0.565	1.055	1.451
15.0	0.624	1.125	1.517
18.0	0.677	1.185	1.573
21.0	0.725	1.238	1.623
24.0	0.769	1.286	1.667
27.0	0.810	1.330	1.706
30.0	0.849	1.371	1.743
35.0	0.909	1.433	1.797
40.0	0.965	1.488	1.846
45.0	1.017	1.539	1.890
50.0	1.065	1.586	1.930
60.0	1.155	1.671	2.002
70.0	1.237	1.746	2.065
80.0	1.313	1.814	2.120
90.0	1.383	1.876	2.171
100.0	1.450	1.934	2.217
110.0	1.513	1.987	2.260
120.0	1.572	2.037	2.299
130.0	1.629	2.084	2.337
140.0	1.684	2.129	2.371
150.0	1.736	2.171	2.404
160.0	1.787	2.212	2.436
170.0	1.835	2.250	2.465
180.0	1.883	2.287	2.494
190.0	1.928	2.323	2.521
200.0	1.973	2.357	2.547
220.0	2.058	2.422	2.596
240.0	2.139	2.483	2.641
260.0	2.217	2.541	2.684
280.0	2.910	2.595	2.724
300.0	2.362	2.647	2.762

Gust Response Factors, G_h and G_z
BNBC-1993, Table-6.2.11

Height above ground level (metres)	G_h and G_z		
	Exposure A	Exposure B	Exposure C
0-4.5	1.654	1.321	1.154
6.0	1.592	1.294	1.140
9.0	1.511	1.258	1.121
12.0	1.457	1.233	1.107
15.0	1.418	1.215	1.097
18.0	1.388	1.201	1.089
21.0	1.363	1.189	1.082
24.0	1.342	1.178	1.077
27.0	1.324	1.170	1.072
30.0	1.309	1.162	1.067
35.0	1.287	1.151	1.061
40.0	1.268	1.141	1.055
45.0	1.252	1.133	1.051
50.0	1.238	1.126	1.046
60.0	1.215	1.114	1.039
70.0	1.196	1.103	1.033
80.0	1.180	1.095	1.028
90.0	1.166	1.087	1.024
100.0	1.154	1.081	1.020
110.0	1.114	1.075	1.016
120.0	1.134	1.070	1.013
130.0	1.126	1.065	1.010
140.0	1.118	1.061	1.008
150.0	1.111	1.057	1.005
160.0	1.104	1.053	1.003
170.0	1.098	1.049	1.001
180.0	1.092	1.046	1.000
190.0	1.087	1.043	1.000
200.0	1.082	1.040	1.000
220.0	1.073	1.035	1.000
240.0	1.065	1.030	1.000
260.0	1.058	1.026	1.000
280.0	1.051	1.022	1.000
300.0	1.045	1.018	1.000

For main force wind force resistance system, use building or structure height *h* for *z*

Overall Pressure Coefficient, $\bar{C}_p$ for Rectangular Building with Flat Roofs

BNBC-1993, Table-6.2.15

h/B	*L/B*					
	0.1	0.5	0.65	1.0	2.0	≥ 3.0
≤ 0.5	1.40	1.45	1.55	1.40	1.15	1.10
10.0	1.55	1.85	2.00	1.70	1.30	1.15
20.0	1.80	2.25	2.55	2.00	1.40	1.20
≥ 40.0	1.95	2.50	2.80	2.20	1.60	1.25

Note:

1. These coefficients are to be used with Method-2 given in Sec.2.4.6.6a(ii). Use $\bar{C}_p = \pm 0.7$ for roof in all cases.
2. Linear interpolation may be made for intermediate values of *h/B* and *L/B*.

External Pressure Coefficients, C_{pe} for Roof

BNBC-1993, Sec-2.6.6.7

Wind Direction	Windward Side								Leeward Side
	h/L	θ (degrees)							
		0°	10°-15°	20°	30°	40°	50°	> 60°	
Normal to ridge	<0.3	-0.7	0.2* -0.9*	0.2	0.3	0.4	0.5	0.01 θ	-0.7 for all values of h/L and θ
	0.5	-0.7	-0.9	-0.75	-0.2	0.3	0.5	0.01 θ	
	1	-0.7	-0.9	-0.75	-0.2	0.3	0.5	0.01 θ	
	> 1.5	-0.7	-0.9	-0.9	-0.9	-0.35	0.2	0.01 θ	
Parallel to ridge	h/B or h/L≤ 2.5	-0.7							-0.7
	h/B or h/L>2.5	-0.8							-0.8

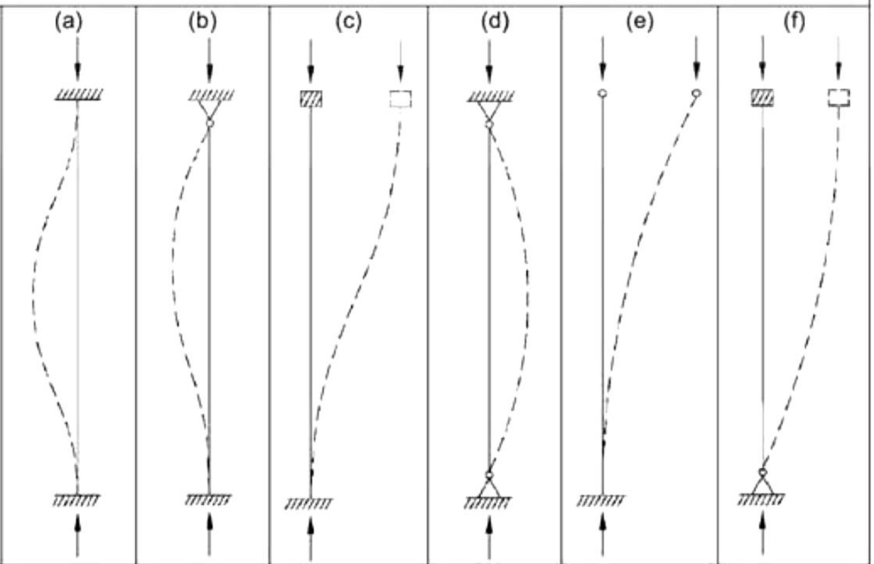
Minimum Size of Fillet Weld
AISC, Table-J2.4

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Filet Weld, in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

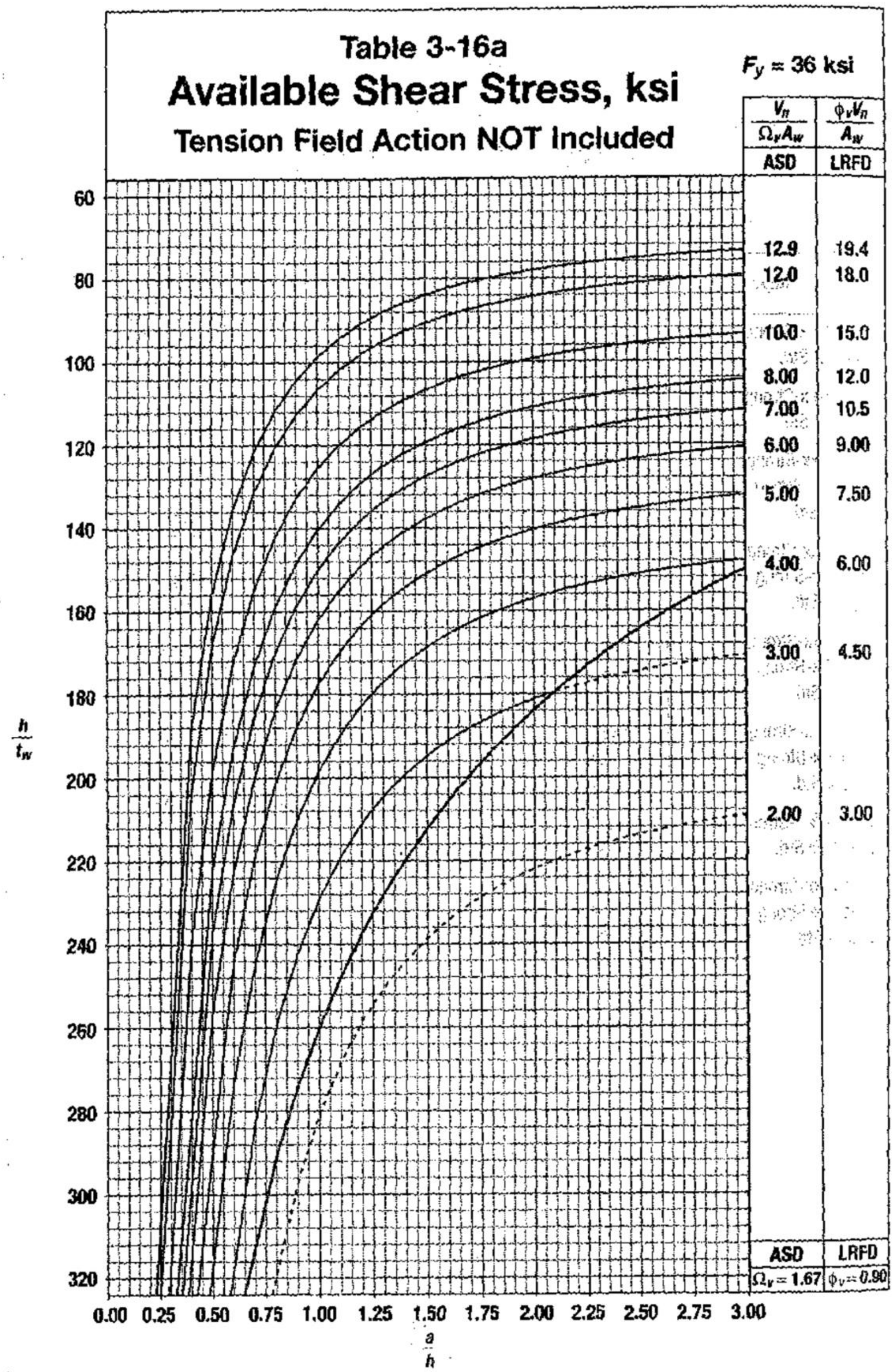
Maximum Size of Fillet Weld

Thickness along edge of the material, in. (mm)	Maximum Size of Filet Weld, in. (mm)
Less than 1/4 (6)	Thickness of the material
≥ 1/4 (6)	Thickness of the material-1/16

Approximate Values of Effective Length Factor, K
AISC, Table-C-A-7.1

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.1	2.0
End condition code	Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free					

Available Shear Stress in Plate Girders
AISC table, 3-16 to 3-17



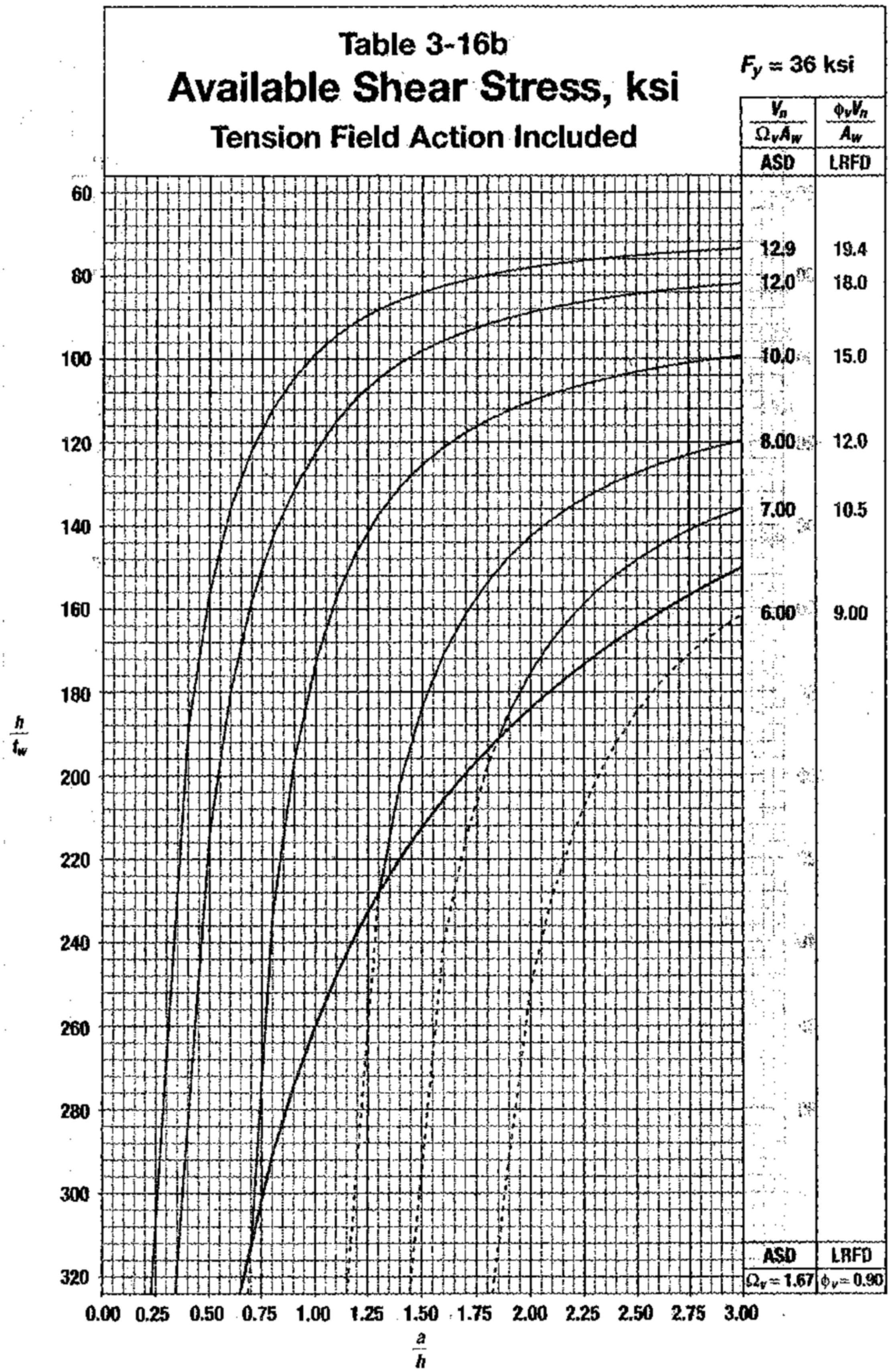


Table 3-16b
Available Shear Stress, ksi
Tension Field Action Included
$F_y = 36$ ksi
$\frac{V_n}{\Omega_v A_w}$
$\frac{\phi_v V_n}{A_w}$
ASD
LRFD
12.9
19.4
12.0
18.0
10.0
15.0
8.00
12.0
7.00
10.5
6.00
9.00
ASD
LRFD
$\Omega_v = 1.67$
$\phi_v = 0.90$
$\frac{h}{t_w}$
60
80
100
120
140
160
180
200
220
240
260
280
300
320
0.00
0.25
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
$\frac{a}{h}$

Table 3-17a
Available Shear Stress, ksi
Tension Field Action NOT Included

$F_y = 50$ ksi

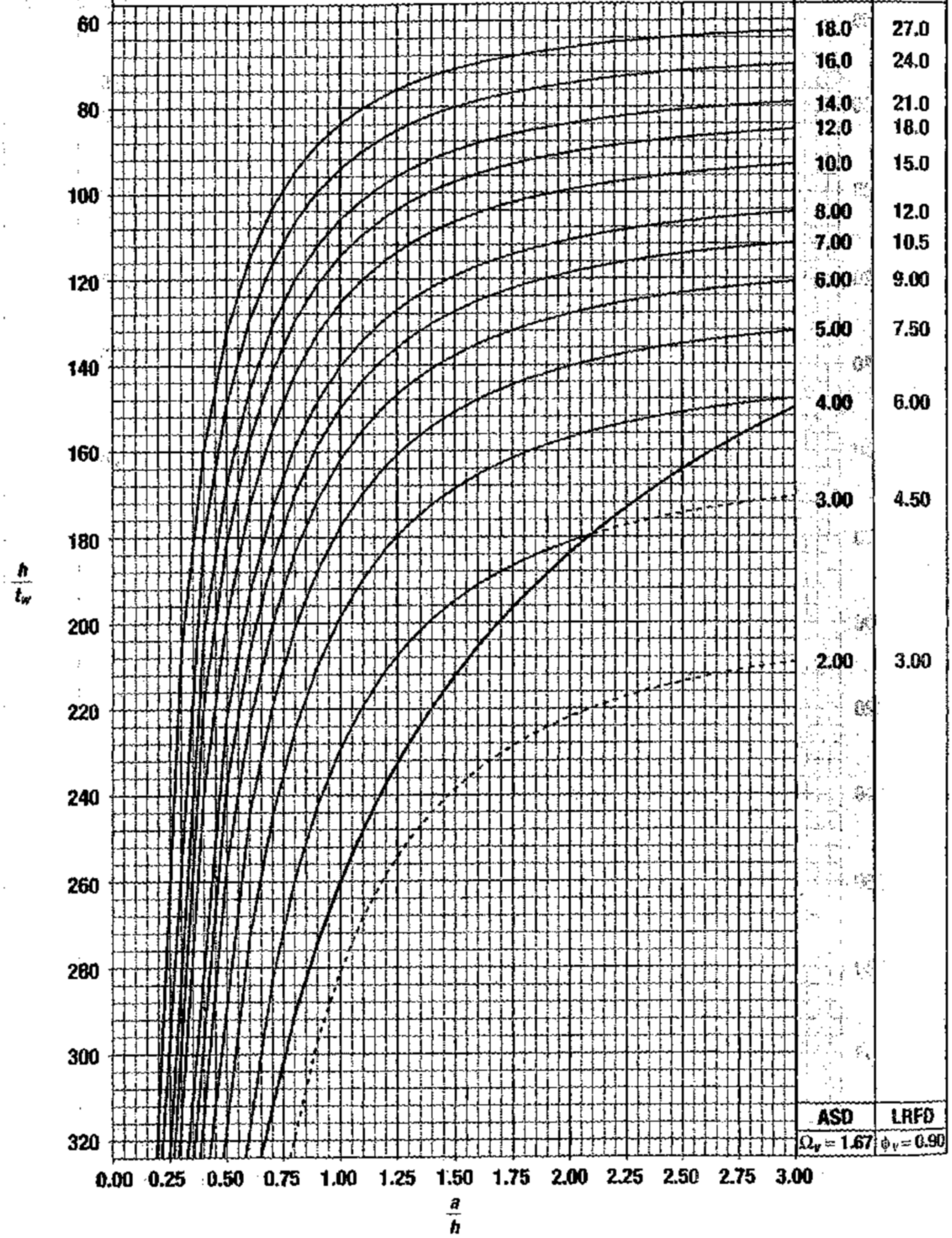


Table 3-17b

Available Shear Stress, ksi

Tension Field Action Included