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

Department of Civil Engineering
Ahsanullah University of Science and Technology

FALL 2022

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.

Updated by

Md. Munirul Islam
Assistant Professor

Farjana Akther Sweety
Lecturer
Department of Civil Engineering
Ahsanullah University of Science and Technology

Prepared by

Fatema-Tuz-Zahura
Assistant Professor

Urmi Devi
Lecturer
Department of Civil Engineering
Ahsanullah University of Science and Technology

INDEX

Page no.

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 (Structuralsteel frame, n.d.)

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

Figure 1.2(b): Arc bridge (Old trails archbridge, 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)

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:** A compact section reaches its cross-sectional material strength, or capacity, before local buckling occurs.
- **Non-compact:** In a non-compact section, only a portion of the cross-section reaches its yield strength before local buckling occurs.
- **Slender:** In a slender section, the cross section does not yield and the strength of the member is governed by local buckling.

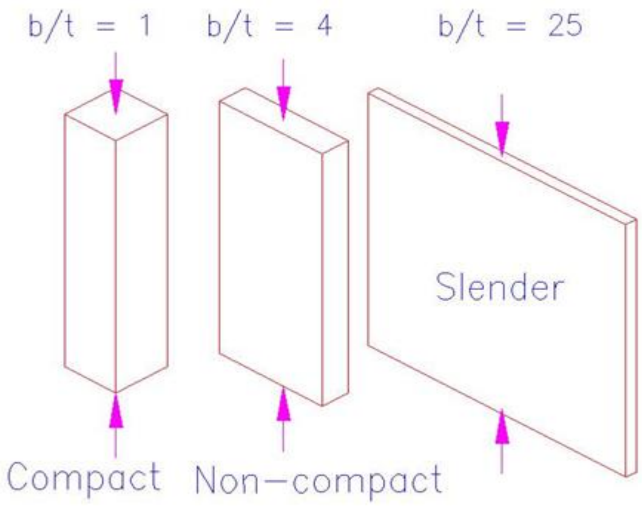


Figure 1.3: Compact, Non-compact & Slender Sections

In the following figure b indicates width, t indicates thickness of the sample & from the ratio of b/t it helps to identify the compact, non-compact & slender section.

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 CRS

Figure 1.4: Hot rolled steel

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.

Figure 1.5: Cold rolled steel

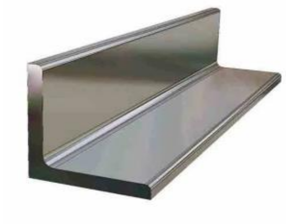
Difference between Hot-Rolled Steel and Cold-Rolled Steel

Category	**Hot-Rolled Steel**	**Cold- Rolled Steel**
Product Features	The original colour is a mixture of blue and black and it's a bit only.	There is a combination of white spots with grey colour and a bit of glossy.
Aesthetics	These types of steel are shaggy with no sharp edges.	These types of steel are not shaggy and have an aesthetic touch with a beautiful and smooth surface.
Features	At high temperatures, hot rolled can change features, shapes and sizes.	The features remain unchanged while the shape changes.
Preserve technique	Without the protective packaging outside, hot-rolled steel can be easily preserved.	To remove all the rust, exterior storage and packaging are needed.

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.	

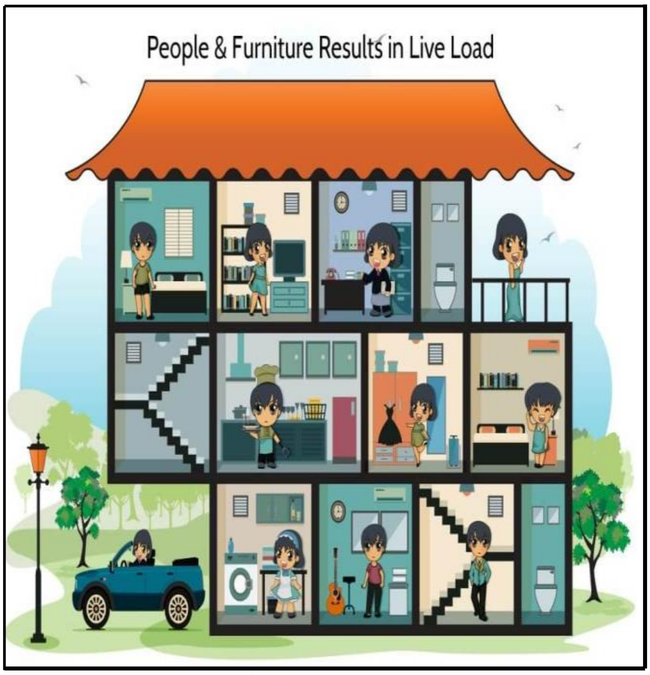
Symbol	Type of shape	Description	Figure
WT ST MT	Tee	WT shapes are cut from a wide flange. ST shapes are cut from American Standard Beams. MT shapes are cut from non-standard I-shapes.	
HSS TS	Hollow Steel Section Steel Tube	Either nomenclature is acceptable; however, HSS is more common.	
L	Angle	Angles 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:
    i. floor loads
    ii. Snow and ice
    iii. Rain especially on flat roofs because ponding develops causing deflections.
    iv. Traffic loads for bridges.
    v. Impact loads: such as falling objects or sudden car braking.
    vi. Lateral loads: such as wind, which changes with height, geographic location, surrounding structures
    vii. Earthquakes are another example of impact loads.
    viii. longitudinal loads: such as sudden stopping of trains or trucks on bridges.
    ix. Other live loads: soil pressure on walls or foundations, water on dams, explosions, thermal forces due to temperature changes… .. etc.



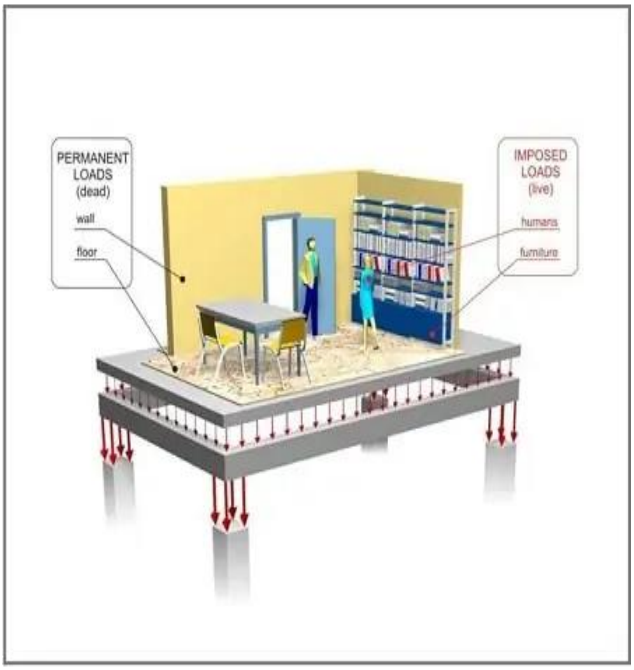


Figure 1.6: Dead load & Live load

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.

Figure 2.1: Truss Structure

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.

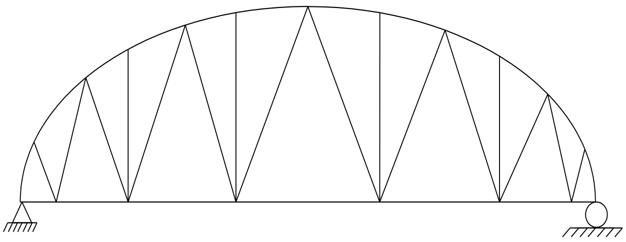
Load Distribution

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.

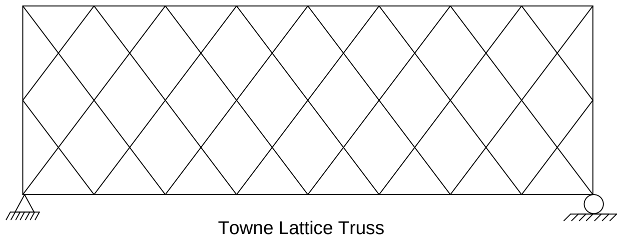
Accessibility

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.

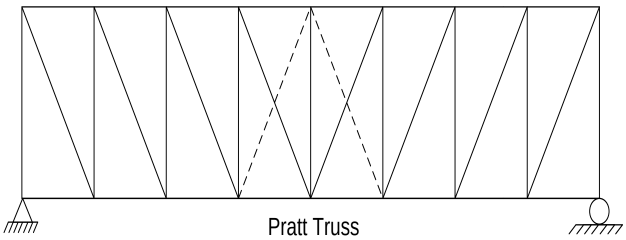
2.4 Types of Truss:



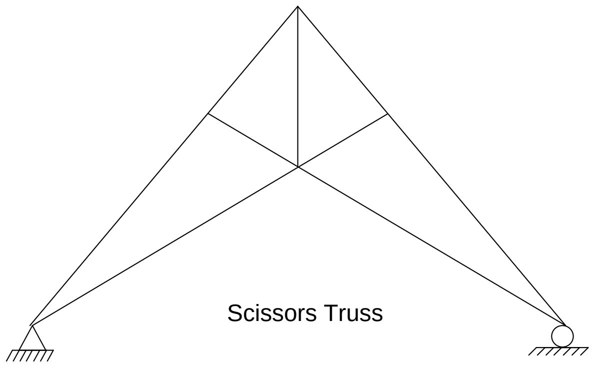
Bowstring Truss



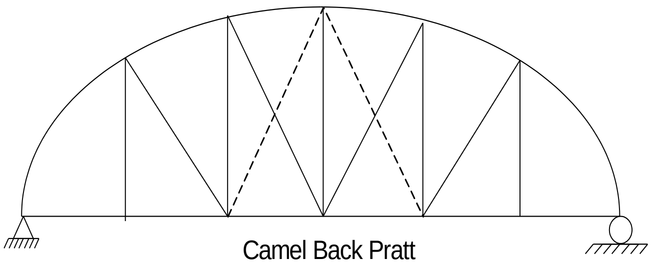
Towne Lattice Truss



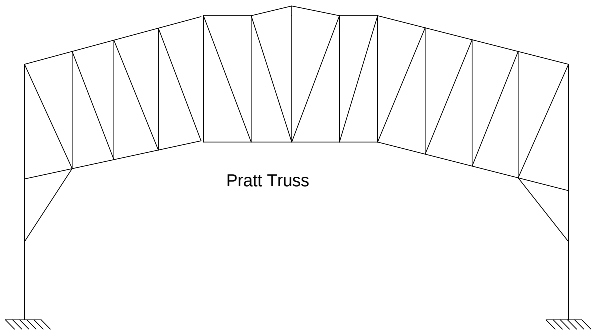
Pratt Truss



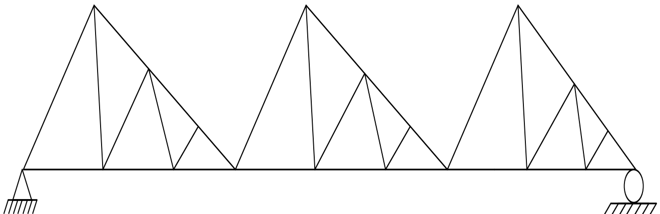
Scissors Truss



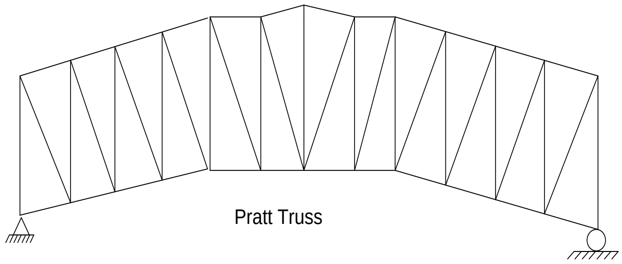
Camel Back Pratt



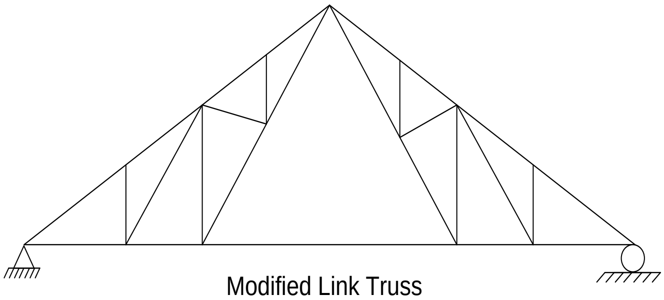
Pratt Truss

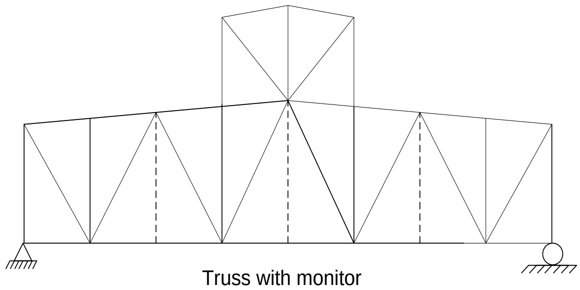


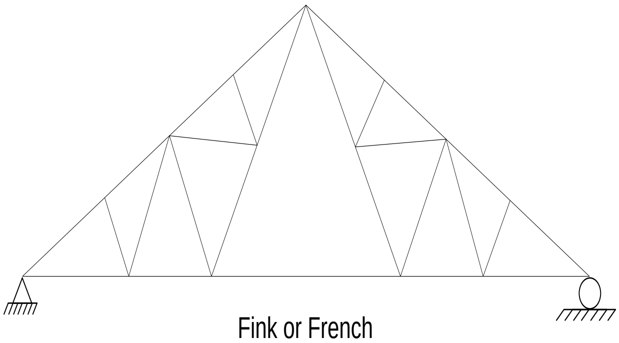
Saw-Tooth Truss

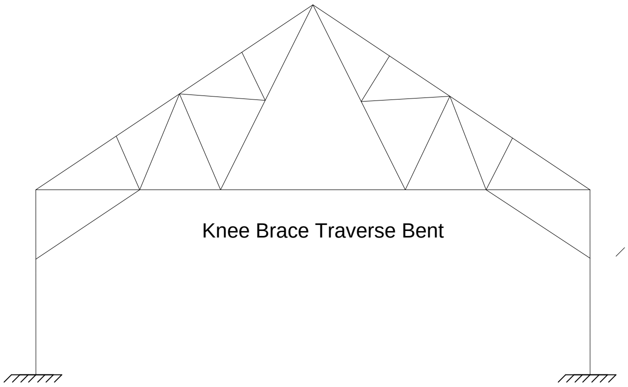


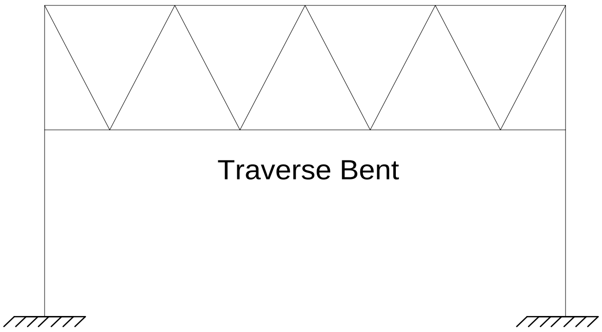
Pratt Truss

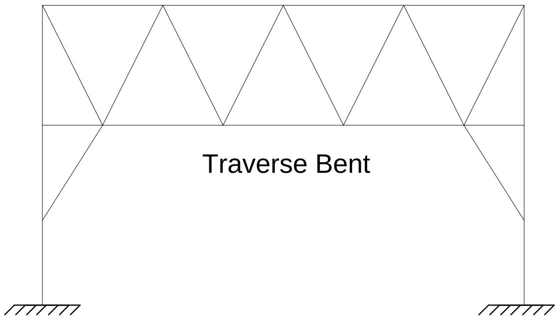

Modified Link Truss

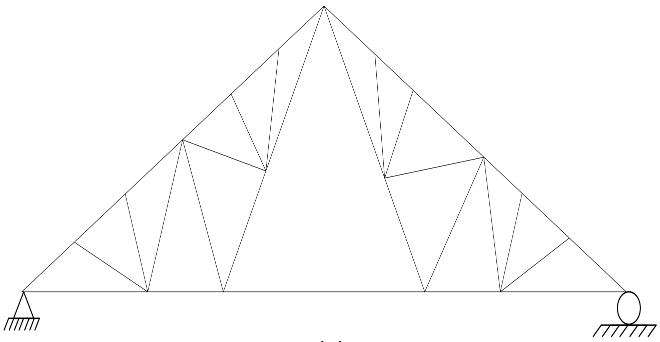

Truss with monitor

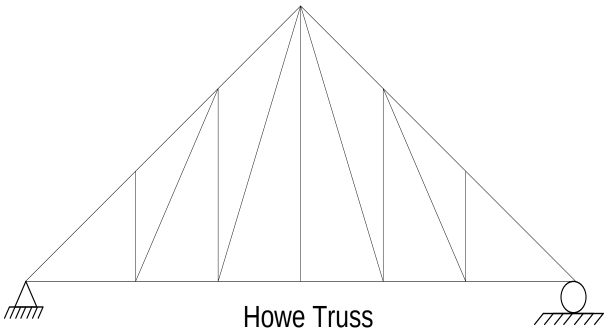

Fink or French


Knee Brace Traverse Bent


Traverse Bent


Traverse Bent


Fan Fink


Howe Truss

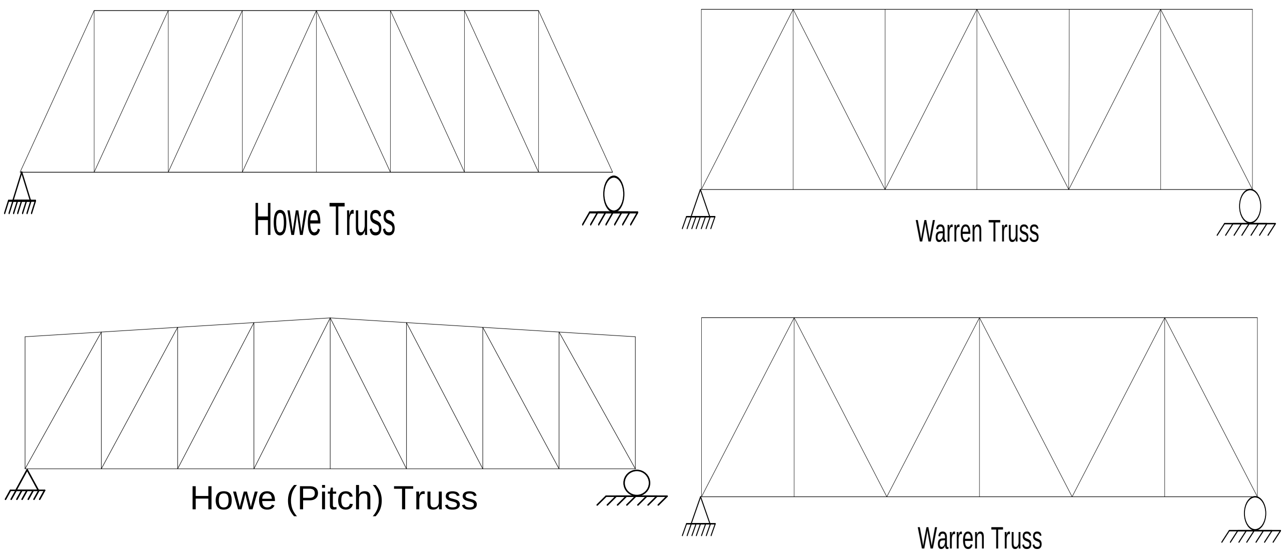


Figure 2.2: Different types of truss

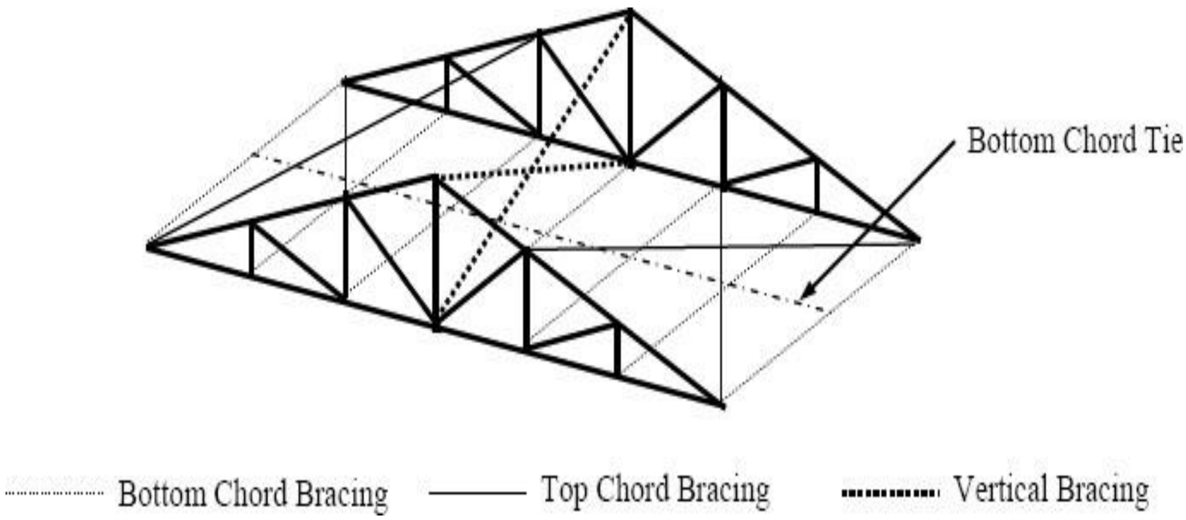


Figure 2.3: Bracing System of truss

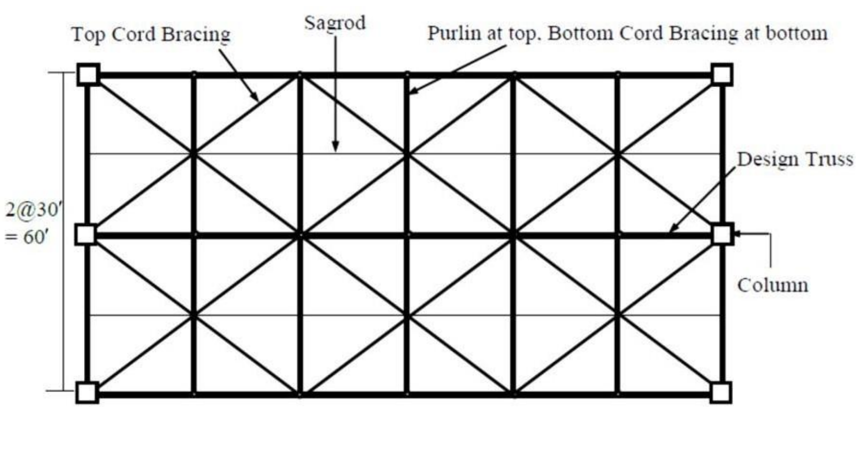


Figure 2.4: Building Plan

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 2020 (Bangladesh National Building Code 2020).

(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)

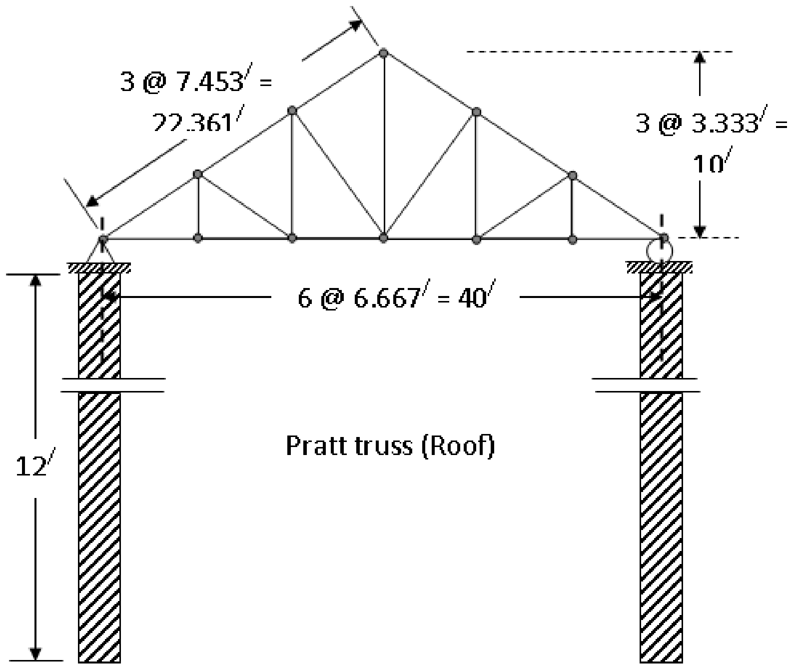


Figure 2.5: Pratt type Roof Truss

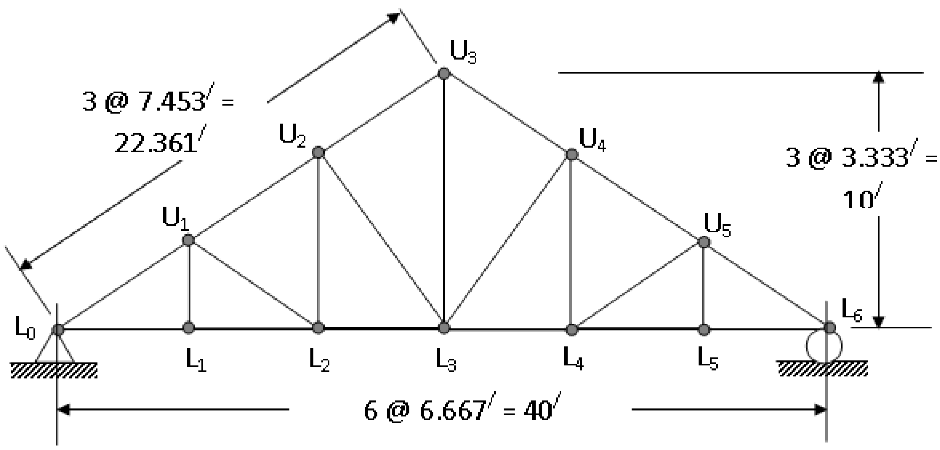


Figure 2.6: Truss notations (member numbering): Pratt truss (Roof)

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 = negligible

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)

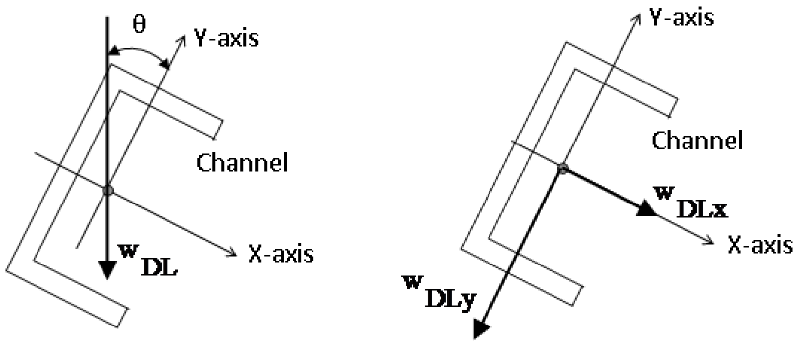


Figure 2.7: Bi-axial loading on the purlin

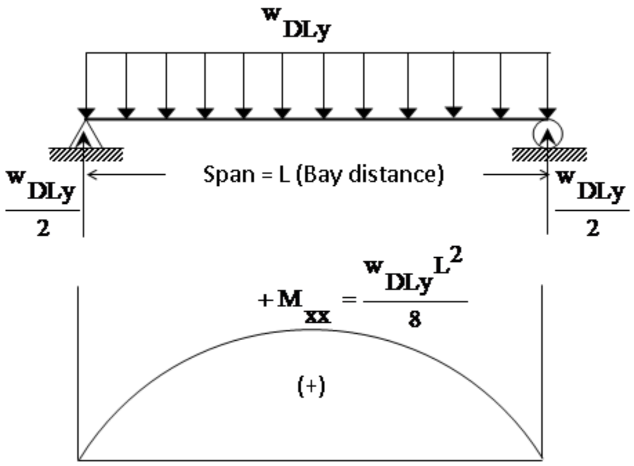


Figure 2.8: Bending moment diagram for loading in Y direction (loading perpendicular to the roof surface, bending parallel to the roof)

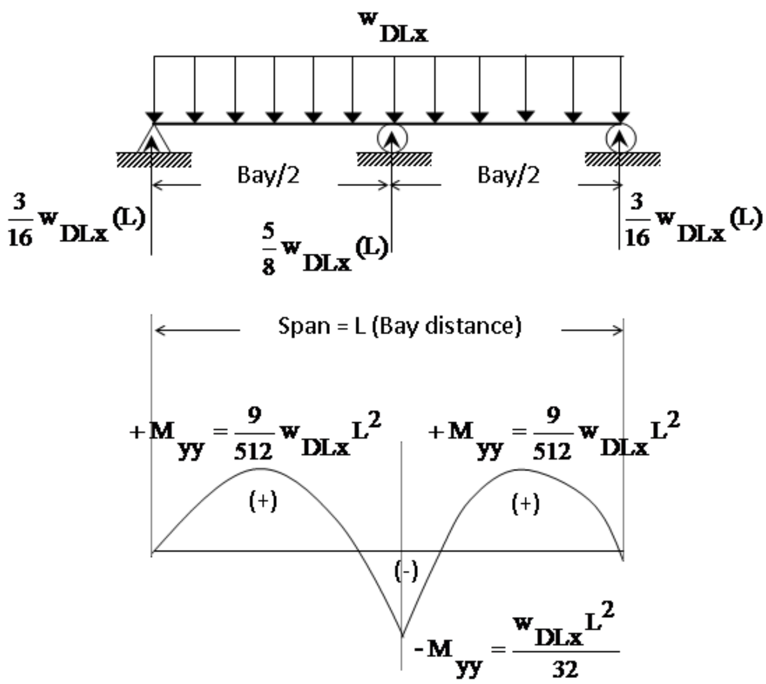


Figure 2.9: Bending moment diagram for loading in X direction (loading parallel to the roof surface, bending perpendicular to the roof)

⇒ 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 sagrod 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	0.196

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

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

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

$$f = \frac{1.8228 \times 12}{1.10} + \frac{0.22785 \times 12}{0.196} = 33.835 \text{ ksi}$$

⇒ Check of bending stress:

Actual bending stress (f = 33.835 ksi) > allowable bending stress (F_b = 23.76 ksi)

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

Table 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: 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	0.196	33.835	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

⇒ 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).

<u>2.6.2 Analysis & Design of Purlin for dead load plus wind load:</u>

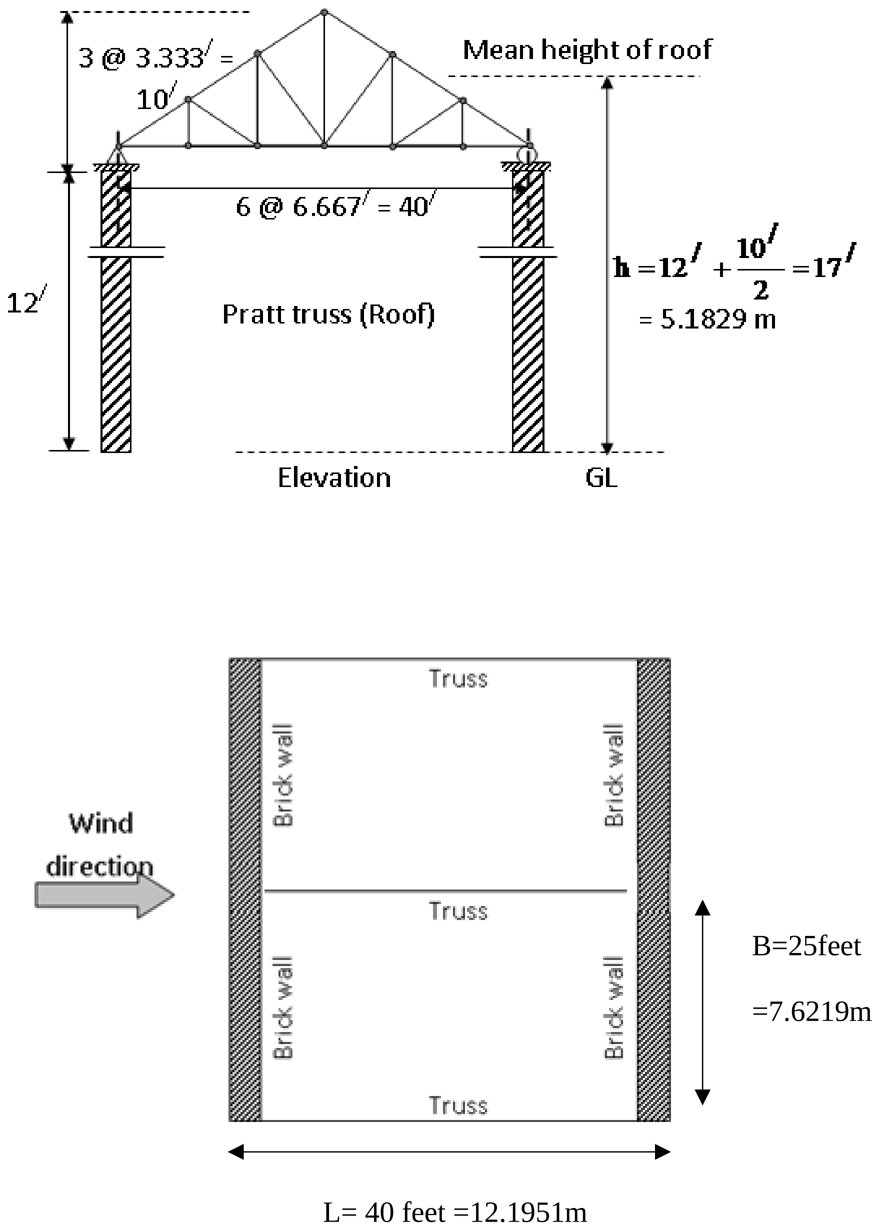


Figure 2.10: Plan and elevation of roof truss

Wind load Calculation:

Different parameters for wind load calculation:

B = Horizontal dimension of the building, in meters measured normal to wind direction = bay distance (truss to truss spacing) =25 feet = 7.5219m

L= span of truss = 40 feet = 12.1951 m

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

z = Height above the ground is meters = 17 feet =5.1829 m

$\theta =$ Angle of the plane of roof from horizontal, degrees = 26.5651° (degree)

Step-1

Truss location: Dhaka

Basic wind speed, V = 65.7m/s (Dhaka)

Importance Factor, I = 1 (Class II)

K_h or K_z = 0.7 (Case 1)

Wind Directionality factor, K_d = 0.85

Surface roughness A (urban Area)

Exposure A

Topographic factor, K_{zt} = 1.0

Step 2:

Velocity Pressure:

$q_z = 0.000613\ K_z\ K_{zt}\ K_d\ V^2\ I$

$= 0.000613\times(0.7)\ \times1\times0.85\times(65.7)^2\times1$ [V in m/s]

= 1.574 KN/m^2 (q_h also)

Step 3:

Gust effect factor G/G_f = 0.85 (for rigid structure)

Internal pressure co-efficient GC_{pi}

(+) sign→ Towards the internal surface

(−) sign→ Outward direction from internal surface

$GC_{pi} = \pm 0.18$ [Enclosed building]

Pressure Co-efficient External C_p

θ= 26.5651°

h/L = 5.1829/12.1951=0.425

h/L	25°	30°	26.5651°
≤ 0.25	-0.2	-0.2	-0.2
	0.3	0.3	0.3
0.425	–	–	-0.248
	–	–	0.23
0.5	-0.3	-0.2	-0.269
	0.2	0.2	0.2

$$\frac{-0.3-(-0.2)}{25-30} = \frac{-0.3 - \times}{25-26.5651}$$ (Interpolation for angle)

$\therefore \times = -0.269$

$$\frac{-0.2-(-0.269)}{0.25-0.5} = \frac{-0.2-x}{0.25-0.425}$$ (Interpolation for h/L ratio)

$\therefore x = -0.248$

$$\frac{0.3-0.2}{0.25-0.5} = \frac{0.3-x}{0.25-0.425}$$ (Interpolation for h/L ratio)

$\therefore x = 0.23$

<u>Windward roof</u>

Normal to ridge, $C_p/GC_{pf} = -0.248$ & 0.23

<u>Leeward roof</u>

Normal to ridge, $C_p/GC_{pf} = -0.6$

Step 4:

Design Wind load P [1 KN/m^2 =20.89 psf]

$P = q_z GC_p - q_i (GC_{pi})$

Windward, P = $[1.574 \times (0.85) \times (-0.248)] - [(1.574) \times (0.18)]$

= $-0.6151\ KN/m^2$

$= -12.84$ psf

Windward, P = $[1.574 \times (0.85) \times (0.23)] - [(1.574) \times (0.18)]$

= $0.024\ KN/m^2$

$= 0.5$ psf

Leeward, P = $[1.574 \times (0.85) \times (-0.6)] - [(1.574) \times (0.18)]$

= $-1.086\ KN/m^2$

$= -22.67$ psf

Again,

Windward, P = $[1.574 \times 0.85 \times (-0.248)] - [1.574 \times (-0.18)]$

$= -0.332 + 0.283$

$= -0.049\ KN/m^2$

$= -1.023$ psf

Windward, P = $[1.574 \times 0.85 \times (0.23)] - [1.574 \times (-0.18)]$

= $(0.31 + 0.283)\ KN/m^2$

$= 0.593\ KN/m^2$

$= 12.382$ psf

Leeward, P = $[1.574 \times 0.85 \times (-0.6)] - [(1.574) \times (-0.18)]$

= $-0.803 + 0.283$

$= -0.519\ KN/m^2$

$= -10.836$ psf

So, consider

Windward, 12.382 psf and -12.84 psf

Leeward, -22.67 psf

Calculation of UDL on Purlin

UDL on purlin on windward side

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

= $[(12.382) \times 7.4535]$ lb/ft

= 92.289 Ib/ft

Again,

= $[(-12.84) \times 7.4535]$ lb/ft

$= -95.703$ Ib/ft

UDL on purlin on leeward side

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

= $(-22.67) \times 7.4535 feet$

= -168.971 lb/ft

Resultant load in Y direction,

$W_y = W_{DLy} + P_z$

$= (23.332 + 92.289)$ lb/ft

$= 115.621$ lb/ft

Again, $W_y = (23.332 - 95.703)$ lb/ft

$= -72.371$ lb/ft

Again, $W_y = (23.332 - 168.971)$ lb/ft

$= -145.638$ lb/ft

$$M_{XX} = \frac{Wy \times L^2}{8}$$

$$= \frac{(-145.638) \times (25)^2}{8}$$

$= -11378.035 \text{ lb} - \text{ft}$

$M_{XX} = -11.378 \text{ kip} - \text{ft}$

$M_{YY} = 0.22785 \text{ kip} - \text{ft}$ (From previous report)

Allowable bending stress

For A36 steel,

$F_b=0.66F_y$

= 0.66× (36) ksi

= 23.76 ksi

Bending stress developed on purlin section

$$f = \pm \frac{Mxx(cy)}{Ixx} \pm \frac{Myy(cx)}{Iyy}$$

$$= \frac{Mxx}{Sxx} + \frac{Myy}{Syy}$$

For channel section, C 4×5.4 (S_{xx}=1.10 in^3 & S_{yy}= 0.202 in^3) [Channel from previous report]

$$f = \frac{11.378\times(12)}{1.10} + \frac{0.22785\times12}{0.202}$$

= 116.024 ksi > 23.76 ksi (not ok)

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

$$f = \frac{11.378\times12}{5.80} + \frac{0.22785\times12}{0.642}$$

=27.79 ksi > 23.76 ksi (not ok)

Select channel C 8×11.5 (S_{xx}=8.14 in^3 & S_{yy}=0.775 in^3)

$$f = \frac{11.378\times12}{8.14} + \frac{0.22785\times12}{0.775}$$

= 20.301 < 23.76 (ok)

Check self- weight of purlin

For C 8×11.5 Channel,

Self -weight is $\frac{11.5}{7.4535}\ \frac{\mathrm{lb/ft}}{\mathrm{ft}}$ = 1.543 psf > 1.5 psf (Not ok)

As upper section of C 8×11.5 Channel has higher self-weight, Use 1.55 self -weight of purlin for 2nd trial.

C.G.I sheet roofing = 2 psf

Self-weight of purlins = 1.55 psf

Total dead load = 3.55 psf

UDL on purlin, W_{DL}= 3.55 psf × purlin spacing

= 3.55 psf × 7.453 ft.

= 26.458 lb/ ft

$W_{DLx} = W_{DL} \times \sin\theta$ = 11.832 lb/ ft

$W_{DLy} = W_{DL} \times \cos\theta$ = 23.665 lb/ ft

$$W_y = W_{DLy} + P_z$$

$$= (23.665 - 168.971)\ \mathrm{lb/ft}$$

$$= -145.306\ \mathrm{lb/ft}$$

$$M_{xx} = \frac{(-145.306) \times (25)^2}{8} = -11352.031\ \mathrm{lb - ft}$$

$$M_{XX} = -11.352\ \mathrm{kip - ft}$$

$$M_{yy} = \frac{(11.832) \times (25)^2}{32} = 231.094\ \mathrm{lb - ft}$$

$$M_{YY} = 0.231\ \mathrm{kip - ft}$$

Select channel C 8×11.5 (S_{xx}=8.14 in^3 & S_{yy}=0.775 in^3)

$$f = \frac{11.352 \times 12}{8.14} + \frac{0.231 \times 12}{0.775}$$

= 20.311 < 23.76 (ok)

Check self- weight of purlin (2^{nd} trial)

For C 8×11.5 Channel,

Self -weight is $\frac{11.5}{7.4535}\ \frac{lb/ft}{ft}$= 1.543 psf < 1.55 psf

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)

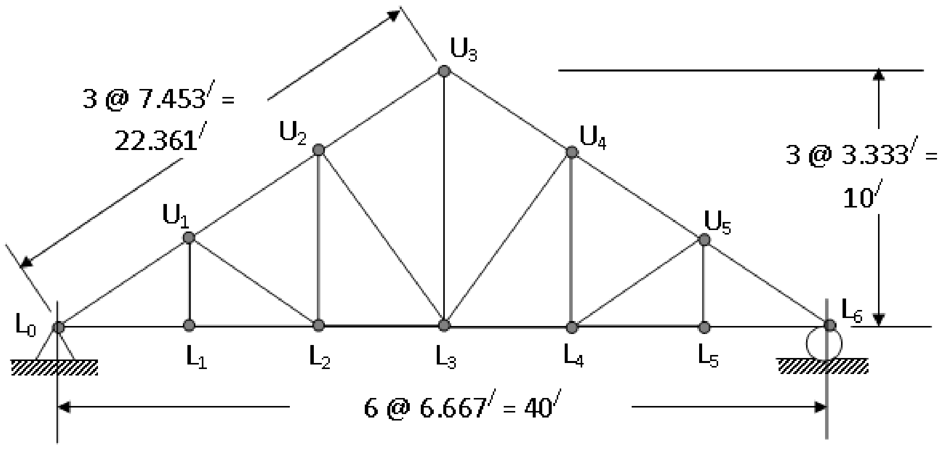


Figure 2.11: Truss notations (member numbering): Pratt truss (Roof)

⇒ **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.

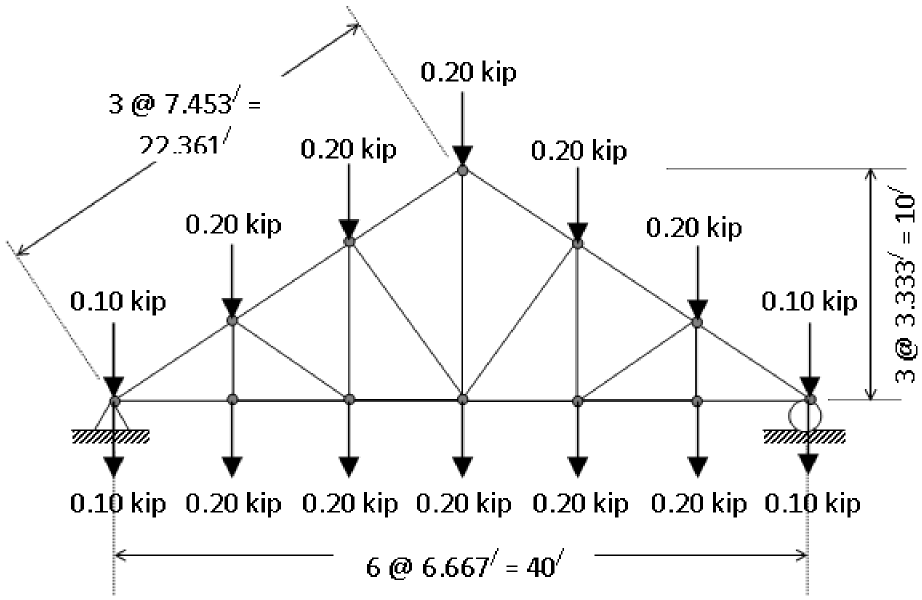


Figure 2.12(a): Dead loads on the truss (self-weight distribution)

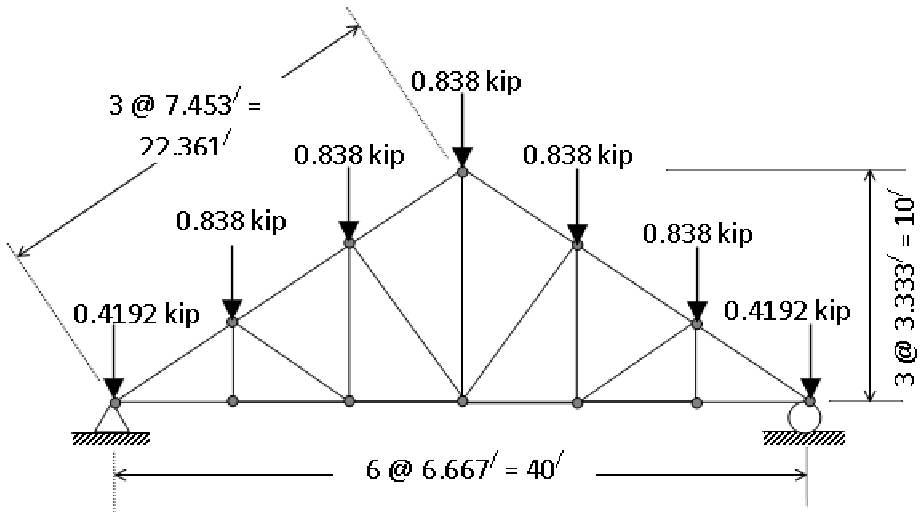


Figure 2.12(b): Dead loads on the truss (sagrod, bracing, purlin self-weight, roof weight distribution)

Summing figure 12(a) & 12(b), gives dead load on the truss (see figure 13)

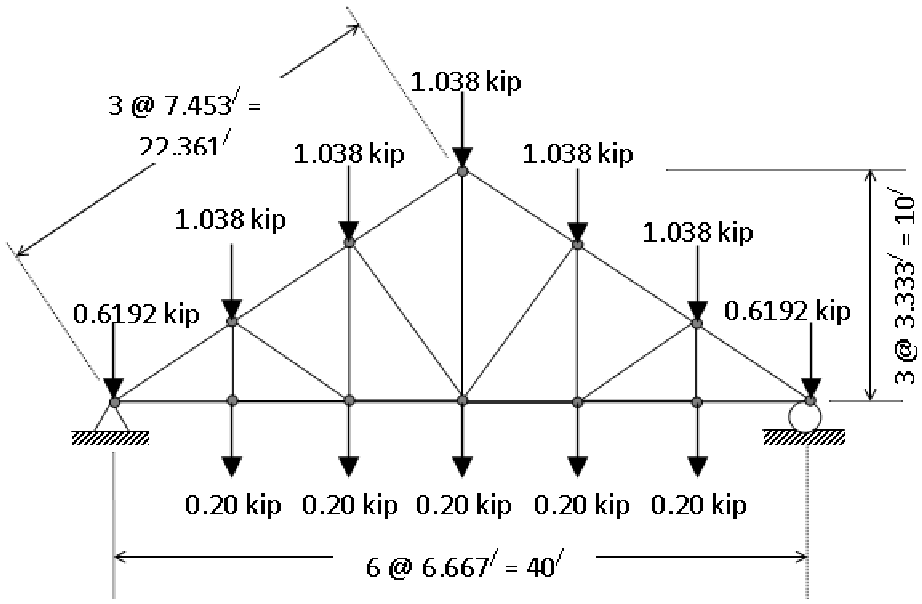


Figure 2.13: Dead loads on the truss

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

Design wind pressure on windward side = -12.84 psf

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

= -12.84 psf × 7.4535 feet × 25 feet

= - 2393 lb = - 2.393 kip

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

= (-12.84 psf) × (7.4535/2 feet) × (25 feet)

= - 1196 lb = - 1.196 kip

Design wind pressure on leeward side = -22.67psf

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

= -22.67 psf × 7.4535 feet × 25 feet

= - 4224 lb = - 4.224 kip

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

= (-22.67 psf) × (7.4535/2 feet) × (25 feet)

= - 2112 lb = - 2.112 kip

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

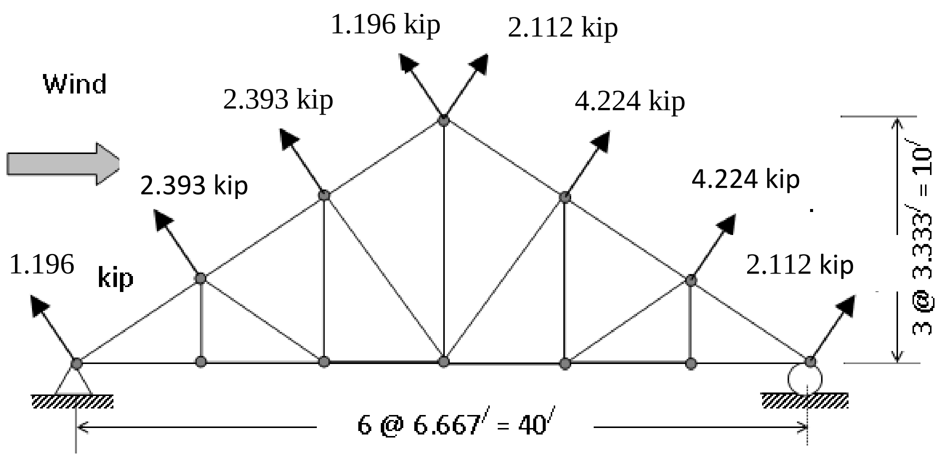


Figure 2.14: Wind loads on the truss for wind blowing from left to right

⇒ **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).

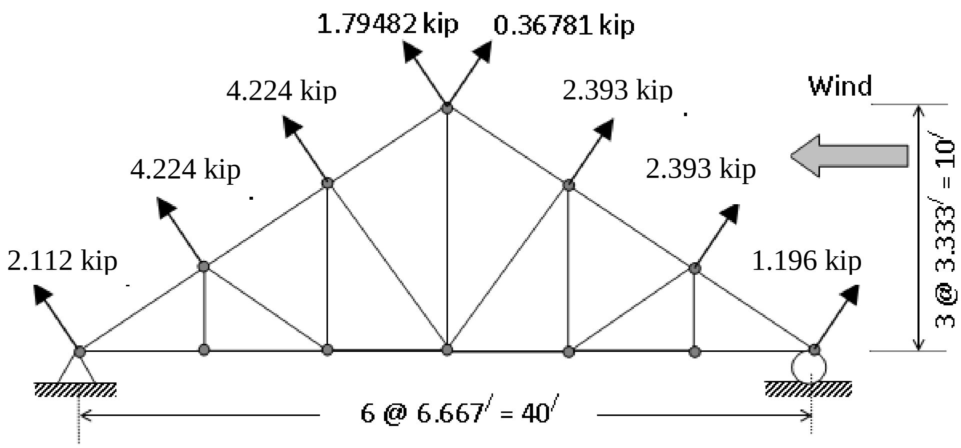


Figure 2.15: Wind loads on the truss for wind blowing from right to left

Note:

Truss loading for	Load direction	
Dead load	Vertically downward	See figure 13
Wind load (left-to-right & right-to-left)	Perpendicular to roof surface	See figure 14 & 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 (Graphical Rapid Analysis of Structural Programme)

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

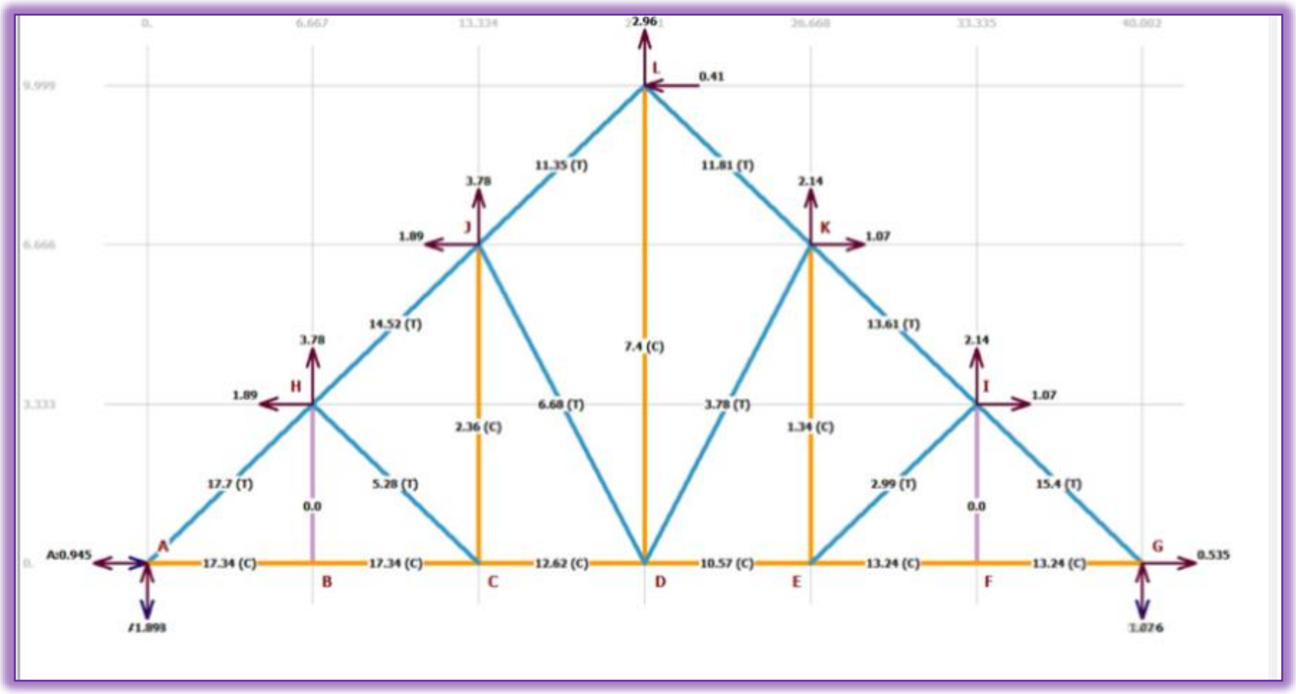


Figure 2.16: Wind loads on the truss for wind blowing from left to right

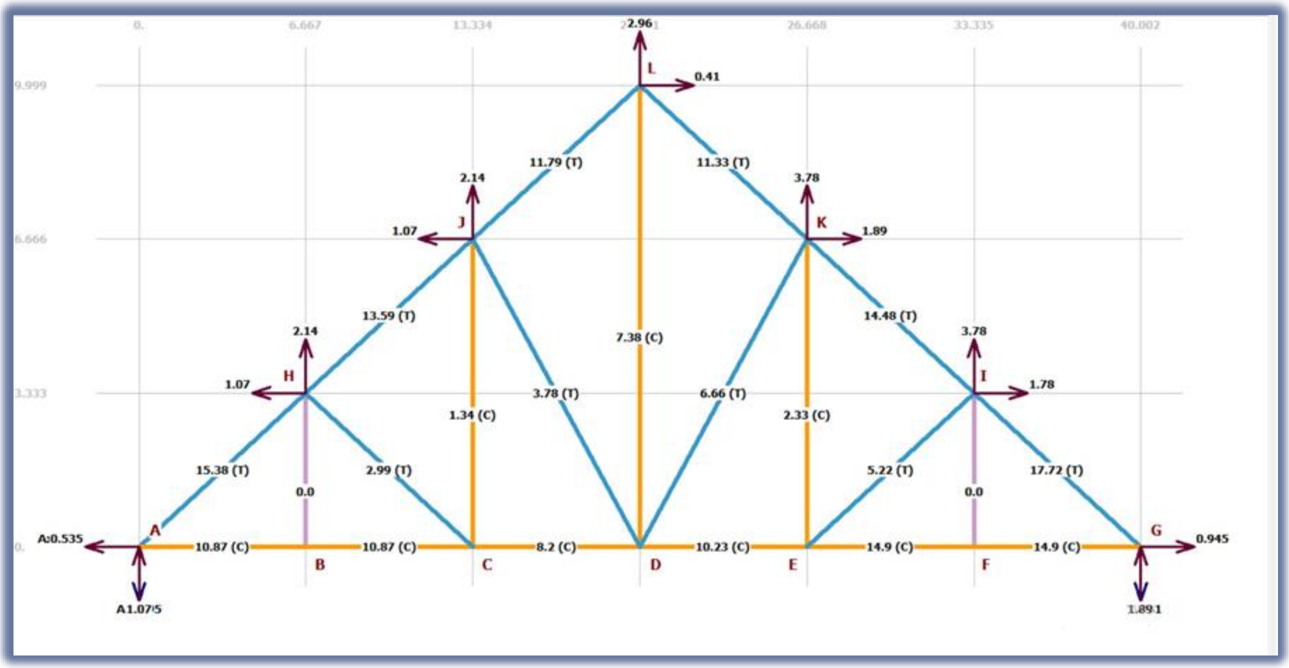


Figure 2.17: Wind loads on the truss for wind blowing from right to left

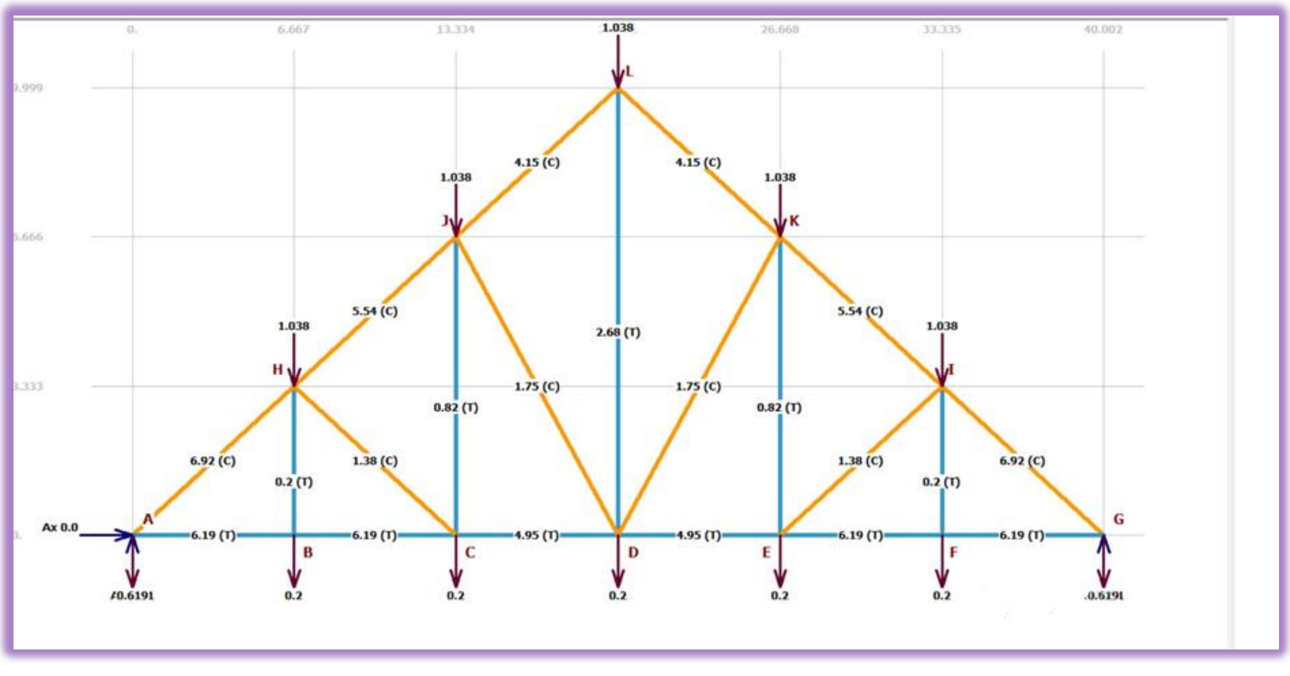


Figure 2.18: Dead loads on the truss

Chart: 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	-6.92	15.40	17.70	-6.92	8.48	10.78	10.78	-6.92
	U1U2	7.4535	-5.54	13.61	14.52	-5.54	8.07	8.98	8.98	-5.54
	U2U3	7.4535	-4.15	11.81	11.35	-4.15	7.66	7.20	7.66	-4.15
	U3U4	7.4535	-4.15	11.35	11.81	-4.15	7.20	7.66	7.66	-4.15
	U4U5	7.4535	-5.54	14.53	13.61	-5.54	8.99	8.07	8.99	-5.54
	U5L6	7.4535	-6.92	17.70	15.40	-6.92	10.78	8.48	10.78	-6.92
Bottom Chord	L0L1	6.6667	6.19	-10.78	-17.34	6.19	-4.59	-11.15	6.19	-11.15
	L1L2	6.6667	6.19	-10.78	-17.34	6.19	-4.59	-11.15	6.19	-11.15
	L2L3	6.6667	4.95	-8.11	-12.62	4.95	-3.16	-7.67	4.95	-7.67
	L3L4	6.6667	4.95	-10.16	-10.57	4.95	-5.21	-5.62	4.95	-5.62
	L4L5	6.6667	6.19	-14.88	-13.24	6.19	-8.69	-7.05	6.19	-8.69
	L5L6	6.6667	6.19	-14.88	-13.24	6.19	-8.69	-7.05	6.19	-8.69
Verticals	U1L1	3.3333	0.20	0	0	0.20	0.20	0.20	0.20	0
	U2L2	6.6667	0.82	-1.34	-2.36	0.82	-0.52	-1.54	0.82	-1.54
	U3L3	10	2.68	-7.40	-7.40	2.68	-4.72	-4.72	2.68	-4.72
	U4L4	6.6667	0.82	-2.36	-1.34	0.82	-1.54	-0.52	0.82	-1.54
	U5L5	3.3333	0.20	0	0	0.20	0.20	0.20	0.20	0
Diagonals/Web members	U1L2	7.4537	-1.38	2.99	5.28	-1.38	1.61	3.90	3.90	-1.38
	U2L3	9.4286	-1.75	3.78	6.68	-1.75	2.03	4.93	4.93	-1.75
	U4L3	9.4286	-1.75	6.68	3.78	-1.75	4.93	2.03	4.93	-1.75
	U5L4	7.4535	-1.38	5.28	2.99	-1.38	3.90	1.61	3.90	-1.38

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 = − 6.92 kip & maximum tensile force = + 10.78 kip.

Select an angle section L $2\times\mathbf{2}\times\frac{1}{4}$; (cross sectional area, A = 0.944 inch^2 & minimum radius of gyration, r_z = 0.387 inch).

Check for Compression: Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times7.4535\times12}{0.387}$ = 138.669

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)

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

$$\Rightarrow F_a = \frac{36\times\left[1-\frac{1}{2}\left(\frac{138.669}{126.099}\right)^2\right]}{\frac{5}{3}+\frac{3}{8}\left(\frac{138.669}{126.099}\right)-\frac{1}{8}\left(\frac{138.669}{126.099}\right)^3} \qquad \text{if } \frac{KL}{r}\le C_c$$

$$\Rightarrow F_a = 7.45 \text{ ksi}$$

Allowable force in compression, $P_a = F_a \times A = 7.45$ ksi × 0.944 inch^2 = 7.0328 kip. (which is greater than design compressive force 15.73 kip). The L $2\times\mathbf{2}\times\frac{1}{4}$; section is OK for compressive force.

<u>Check for Tension:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times7.4535\times12}{0.387}$ = 138.669 (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 × 0.944 inch^2 = 20.3904 kip. (which is greater than design tensile force +10.78 kip). The L $2\times\mathbf{2}\times\frac{\mathbf{1}}{\mathbf{4}}$ section is OK for tensile force.

Top chord	L $2\times2\times\frac{1}{4}$

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 = − 11.15 kip & maximum tensile force = + 6.19 kip.

Select an angle section L $2\frac{1}{2}\times 2\times\frac{1}{4}$ (cross sectional area, A = 1.07 inch^2 & minimum radius of gyration, r_z = 0.423 inch).

<u>Check for Compression:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times6.6667\times12}{0.423}$ = 113.481

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)

$$C_c = \pi\sqrt{\frac{2E}{F_y}} = \pi\sqrt{\frac{2\times29\times10^3}{36}} = 126.099$$

F_a = allowable stress in compression (ksi)

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

$$\Rightarrow F_a = \frac{36\times\left[1-\frac{1}{2}\left(\frac{113.481}{126.099}\right)^2\right]}{\frac{5}{3}+\frac{3}{8}\left(\frac{113.481}{126.099}\right)-\frac{1}{8}\left(\frac{113.481}{126.099}\right)^3} \quad \text{if } \frac{KL}{r} \le C_c$$

$$\Rightarrow F_a = 11.21 \text{ ksi}$$

Allowable force in compression, $P_a = F_a \times A$ = 11.21 ksi × 1.07 inch^2 = 11.9947 kip. (which is greater than design compressive force -11.15 kip). The L $2\frac{1}{2}\times 2\times\frac{1}{4}$ section is OK for compressive force.

<u>Check for Tension:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times6.6667\times12}{0.423}$ = 113.481 (which is less than 300).

F_t = allowable stress in tension (ksi)

$$\Rightarrow F_t = 0.6F_y = 0.6 \times 36 \text{ ksi} = 21.6 \text{ ksi}$$

Allowable force in tension, $P_t = F_t \times A$ = 21.6 ksi × 1.07 inch^2 = 23.112 kip. (which is greater than design tensile force 6.19 kip). The L $2\frac{1}{2}\times 2\times\frac{1}{4}$ section is OK for tensile force.

Bottom chord	L $2\frac{1}{2}\times 2\times\frac{1}{4}$

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 = − 4.72 kip & maximum tensile force = + 2.68 kip.

Select an angle section L $2\frac{1}{2}\times2\times\frac{1}{4}$; (cross sectional area, A = 1.07 inch^2 & minimum radius of gyration, r_z = 0.423 inch).

<u>Check for Compression:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times10\times12}{0.423}$ = 170.213

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)

$$C_c = \pi\sqrt{\frac{2E}{F_y}} = \pi\sqrt{\frac{2\times29\times10^3}{36}} = 126.099$$

F_a = allowable stress in compression (ksi)

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

$$\Rightarrow F_a = \frac{12\pi^2E}{23\left(\frac{KL}{r}\right)^2} = \frac{149000}{(170.213)^2} \qquad \text{if } \frac{KL}{r} \geq C_c$$

$$\Rightarrow F_a = 5.143 \text{ ksi}$$

Allowable force in compression, $P_a = F_a \times A$ = 5.143 ksi × 1.07 inch^2 = 5.503 kip. (which is greater than design compressive force 5.23 kip). The L $2\frac{1}{2}\times2\times\frac{1}{4}$; section is OK for compressive force.

Check for Tension: Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times10\times12}{0.423}$ = 170.213 (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 × 1.07 inch^2 = 23.112 kip. (which is greater than design tensile force 2.68 kip). The L $2\frac{1}{2}\times2\times\frac{1}{4}$ section is OK for tensile force.

Verticals	L $2\frac{1}{2}\times2\times\frac{1}{4}$

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 = − 1.75 kip & maximum tensile force = + 4.93 kip.

Select an angle section L $2\times2\times\frac{1}{8}$; (cross sectional area, A = 0.491 inch^2 & minimum radius of gyration, r_z = 0.391 inch).

Check for Compression: Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times9.4286\times12}{0.391}$ = 173.261

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)

$$C_c = \pi\sqrt{\frac{2E}{F_y}} = \pi\sqrt{\frac{2\times 29\times 10^3}{36}} = 126.099$$

F_a = allowable stress in compression (ksi)

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

$$\Rightarrow F_a = \frac{12\pi^2 E}{23\left(\frac{KL}{r}\right)^2} = \frac{149000}{(173.621)^2} \qquad \text{if } \frac{KL}{r} \geq C_c$$

$$\Rightarrow F_a = 4.943 \text{ ksi}$$

Allowable force in compression, $P_a = F_a \times A$ = 4.943 ksi × 0.491 inch^2 = 2.427 kip. (which is greater than design compressive force 1.75 kip). The L 2×2×$\frac{1}{8}$; section is OK for compressive force.

<u>Check for Tension:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times 9.4286\times 12}{0.391}$ = 173.261 (which is less than 300).

F_t = allowable stress in tension (ksi)

$$\Rightarrow F_t = 0.6F_y = 0.6 \times 36 \text{ ksi} = 21.6 \text{ ksi}$$

Allowable force in tension, $P_t = F_t \times A$ = 21.6 ksi × 0.491 inch^2 = 10.61kip. (which is greater than design tensile force 4.93 kip). The L 2×2×$\frac{1}{8}$ section is OK for tensile force.

Check whether the selected L 2×2×$\frac{1}{8}$ 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 = − 1.38 kip & maximum tensile force = + 3.90 kip.

Select an angle section L $2\times\mathbf{2}\times\frac{1}{8}$; (cross sectional area, A = 0.491 inch^2 & minimum radius of gyration, r_z = 0.391 inch).

<u>Check for Compression:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times7.4537\times12}{0.391}$ = 137.255

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)

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

$$\Rightarrow F_a = \frac{12\pi^2E}{23\left(\frac{KL}{r}\right)^2} = \frac{149000}{(137.255)^2} \qquad \text{if } \frac{KL}{r} \geq C_c$$

$$\Rightarrow F_a = 7.90 \text{ ksi}$$

Allowable force in compression, $P_a = F_a \times A$ = 7.91 ksi × 0.491 inch^2 = 3.884 kip. (which is greater than design compressive force 1.38 kip). The L $2\times\mathbf{2}\times\frac{1}{8}$ section is OK for compressive force.

<u>Check for Tension:</u> Slenderness ratio, $\frac{KL}{r} = \frac{0.6\times7.4537\times12}{0.391}$ = 137.255 (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 × 0.391 inch^2 = 8.45 kip. (which is greater than design tensile force 3.90 kip). The L $2\times\mathbf{2}\times\frac{1}{8}$ section is OK for tensile force.

Diagonals	$L2\times2\times\frac{1}{8}$

Table: Design summary for truss members

Member type	Design section
Top chord	L $2\times\mathbf{2}\times\frac{1}{4}$
Bottom chord	L $2\frac{1}{2}\times2\times\frac{1}{4}$
Verticals	L $2\frac{1}{2}\times\mathbf{2}\times\frac{1}{4}$
Web member / Diagonals	L $2\times\mathbf{2}\times\frac{1}{8}$

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.

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

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

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

$\Rightarrow r_{minimum} > 0.3056183391$ 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.

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

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

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

$\Rightarrow r_{minimum} > 0.350$ inch

From AISC chart for angles we select $L\ 2 \times 2 \times \frac{5}{16}$ 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 connected to the bottom chord struts using standard $\frac{1}{2}$ inch nuts in a manner similar to the sagrods.

Table: Design summary for bracing systems

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

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 =

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

<u>Weld Design of Joint L0:</u>

Here, two members L0U1 (L $2\times 2\times\frac{1}{4}$) & L0L1 L ($2\frac{1}{2}\times 2\times\frac{1}{4}$) 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 = $\frac{3}{8}$ inch.

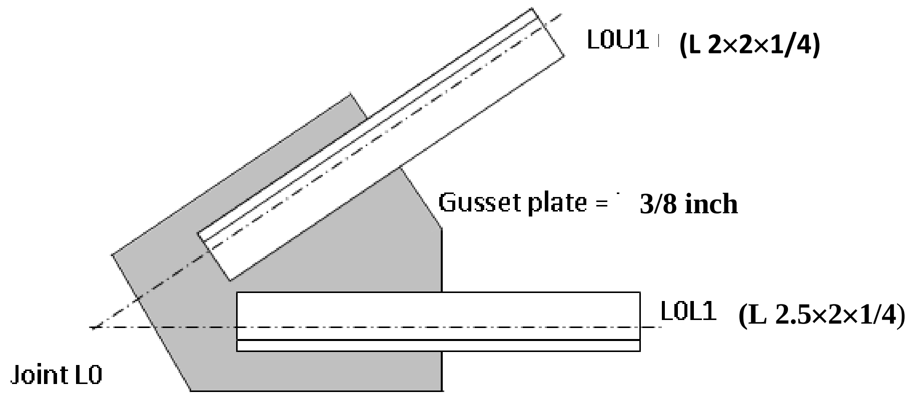


Figure 2.19: Joint Lo of roof truss

<u>Weld for L0L1:</u>

Consider, L0L1 (L $2\frac{1}{2} \times 2 \times \frac{1}{4}$) & gusset plate $\frac{3}{8}$ inch

$t_{max} = \frac{3}{8}$ inch and $t_{min} = \frac{1}{4}$ inch

Maximum thickness of the part being connected, $t_{max} = \frac{3}{8}$ 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, $s_{max} = (\frac{1}{4} - \frac{1}{16})$ inch $= \frac{3}{16}$ inch (from table 2, chapter: welded connections)

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

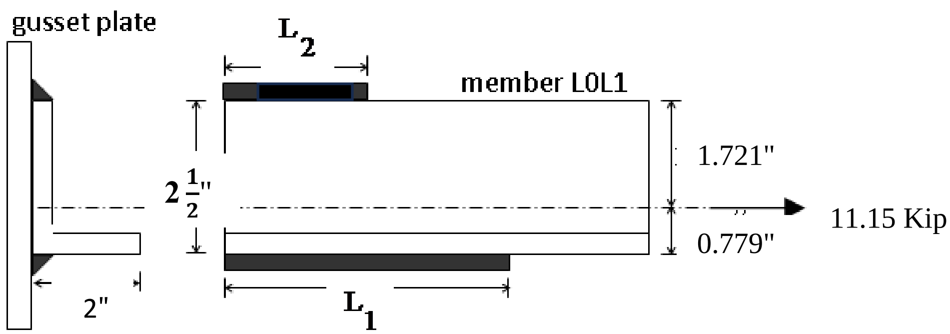


Figure 2.20: Weld design for L_0L_1

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,

$$L_{L_1U_0} = \frac{\left|P_{maximum}\right|_{tensile\ or\ compressive}}{F_v \times t_e} = \frac{11.15\ \text{kip}}{18\ \text{ksi} \times \frac{3}{16} \cos 45^0\ \text{inch}}$$

$$\Rightarrow L_{L_1U_0} = 4.672 \text{ inch}$$

$$\Rightarrow L_1 + L_2 = L_{L_1U_0} = 4.672 \text{ inch}$$

Taking moment about L_2,

$$\Rightarrow (L_1 \times t_e \times F_v) \times (2\tfrac{1}{2}'') = (11.15 \text{ kip}) \times (1.721'')$$

$$\Rightarrow (L_1 \times \frac{3}{16} \cos 45^0 \text{ inch} \times 18 \text{ ksi}) \times (2\tfrac{1}{2}'') = (11.15 \text{ kip}) \times (1.721'')$$

$$\Rightarrow L_1 = 3.216 \text{ inch} \approx 3.50 \text{ inch}$$

Taking moment about L_1,

$$\Rightarrow (L_2 \times t_e \times F_v) \times (2\tfrac{1}{2}'') = (11.15 \text{ kip}) \times (0.779'')$$

$$\Rightarrow (L_2 \times \frac{3}{16} \cos 45^0 \text{ inch} \times 18 \text{ ksi}) \times (2\tfrac{1}{2}'') = (11.15 \text{ kip}) \times (0.779'')$$

$$\Rightarrow L_2 = 1.456 \text{ inch} \approx 1.50 \text{ inch}$$

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

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

Alternatively, $L_1 + L_2 = 4.672$ inch & $\frac{L_1}{L_2} = \frac{1.721 \text{ inch}}{0.779 \text{ inch}}$; from which, $L_1 = 3.23$ inch & L_2 = 1.46 inch.

Weld for L0U1:

Consider, L0U1 ($L\ 2 \times 2 \times \frac{1}{4}$) & gusset plate ($\frac{3}{8}$ inch)

$t_{max} = \frac{3}{8}$ inch and $t_{min} = \frac{1}{4}$ inch

Maximum thickness of the part being connected, $t_{max} = \frac{3}{8}$ 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, $s_{max} = \left(\frac{1}{4} - \frac{1}{16}\right) \text{inch} = \frac{3}{16}$ inch (from table 2, chapter: welded connections)

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

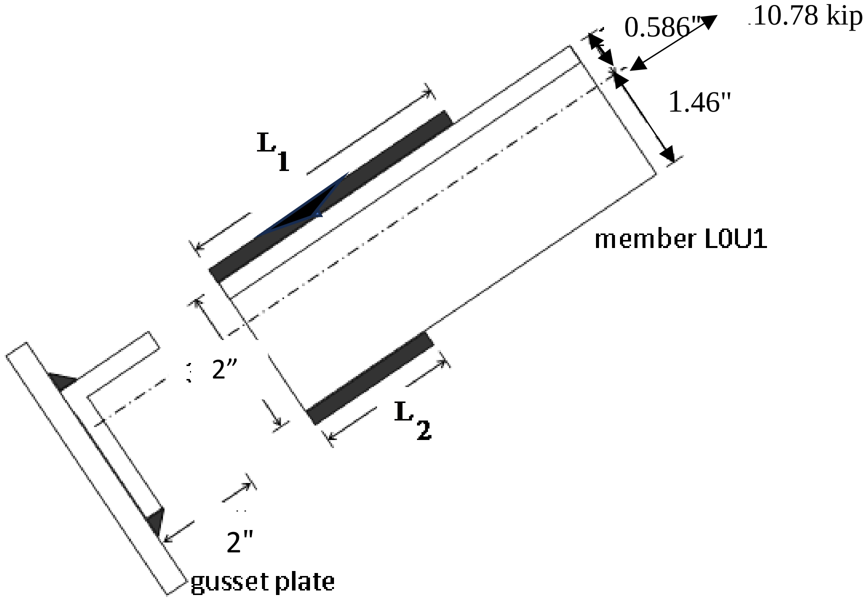


Figure 2.21: Weld design for L_0U_1

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,

$$L_{L_0U_1} = \frac{\left|P_{maximum}\right|_{tensile\ or\ compressive}}{F_v \times t_e} = \frac{10.78\ \text{kip}}{18\ \text{ksi} \times \frac{3}{16} \cos 45^0\ \text{inch}}$$

$$\Rightarrow L_{L_0U_1} = 4.52\ \text{inch}$$

$$\Rightarrow L_1 + L_2 = L_{L_0U_1} = 4.52\ \text{inch}$$

Taking moment about L_2,

$\Rightarrow (L_1 \times t_e \times F_v) \times (2'') = (10.78 \text{ kip}) \times (1.46'')$

$\Rightarrow (L_1 \times \frac{3}{16} \cos 45^0 \text{ inch} \times 18 \text{ ksi}) \times (2'') = (10.78 \text{ kip}) \times (1.46'')$

$\Rightarrow L_1 = 3.297 \text{ inch} \approx 3.50 \text{ inch}$

Taking moment about L_1,

$\Rightarrow (L_2 \times t_e \times F_v) \times (2'') = (10.78 \text{ kip}) \times (0.586'')$

$\Rightarrow (L_2 \times \frac{3}{16} \cos 45^0 \text{ inch} \times 18 \text{ ksi}) \times (2'') = (10.78 \text{ kip}) \times (0.586'')$

$\Rightarrow L_2 = 1.324 \text{ inch} \approx 1.50 \text{ inch}$

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

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

Alternatively, $L_1 + L_2 = 4.52$ inch & $\frac{L_1}{L_2} = \frac{1.46 \text{ inch}}{0.586 \text{ inch}}$; from which, $L_1 = 3.226$ inch & $L_2 =$ 1.295 inch.

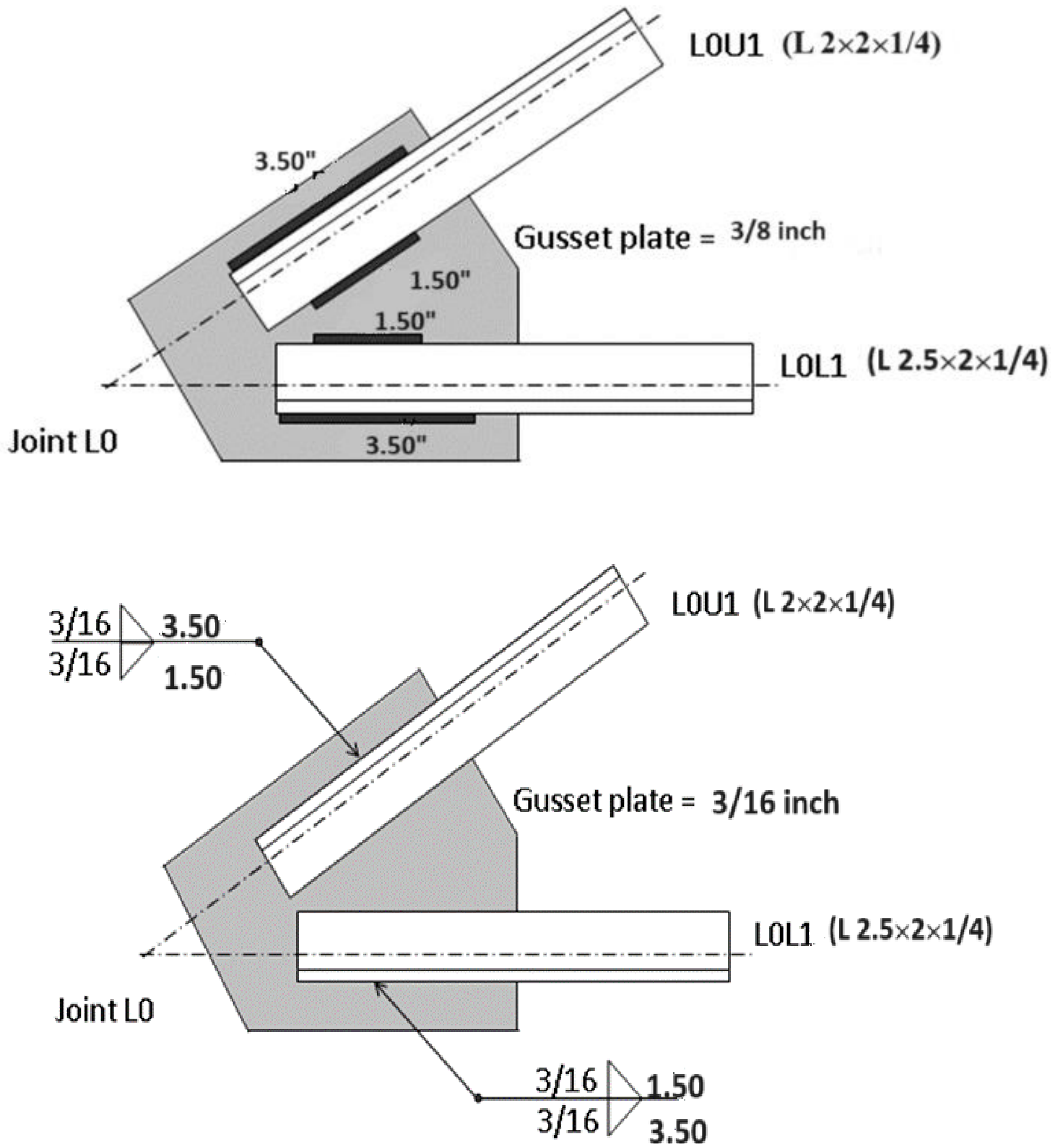


Figure 2.22: Weld design of joint L_0

<u>Weld Design of Joint U1:</u>

Here, two members L0U1 (L $2 \times 2 \times \frac{1}{4}$), U1L1 (L $2\frac{1}{2} \times 2 \times \frac{1}{4}$) & U1L2 (L $2 \times 2 \times \frac{1}{8}$) 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 = $\frac{6}{16}$ inch.

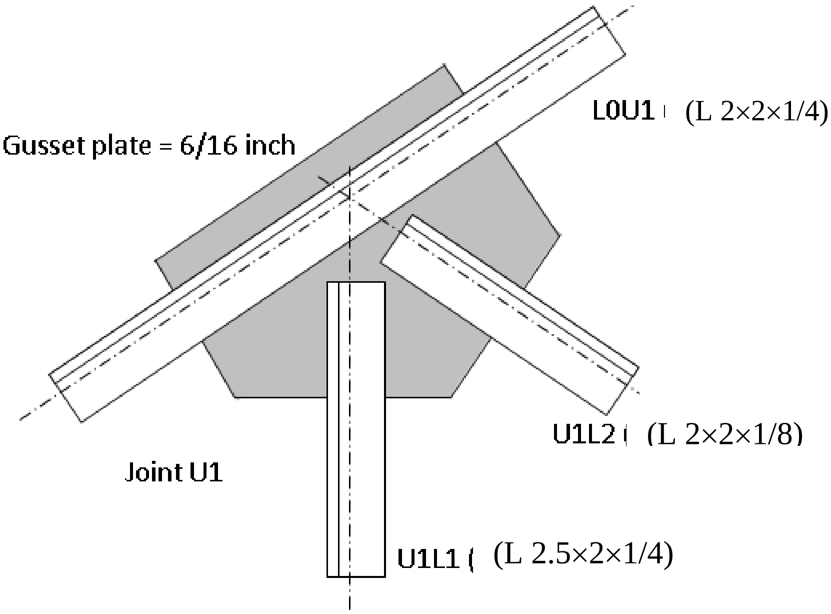


Figure 2.23: Weld design of joint U_1

Weld for L0U1U2:

Consider, L0U1 (L $2 \times 2 \times \frac{1}{4}$) & 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 $\left\lvert F_{L_0U_1} - F_{U_1U_2} \right\rvert$
DL	− 6.92	− 5.54	$\lvert -6.92-(-5.54)\rvert$ = 1.38
DL + W (L→R)	+ 8.48	+ 8.07	$\lvert +8.48-(+8.07)\rvert$ = 0.41
DL + W (R→L)	+ 10.78	+ 8.98	$\lvert +10.78-(+8.98\rvert$ = 1.80

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.

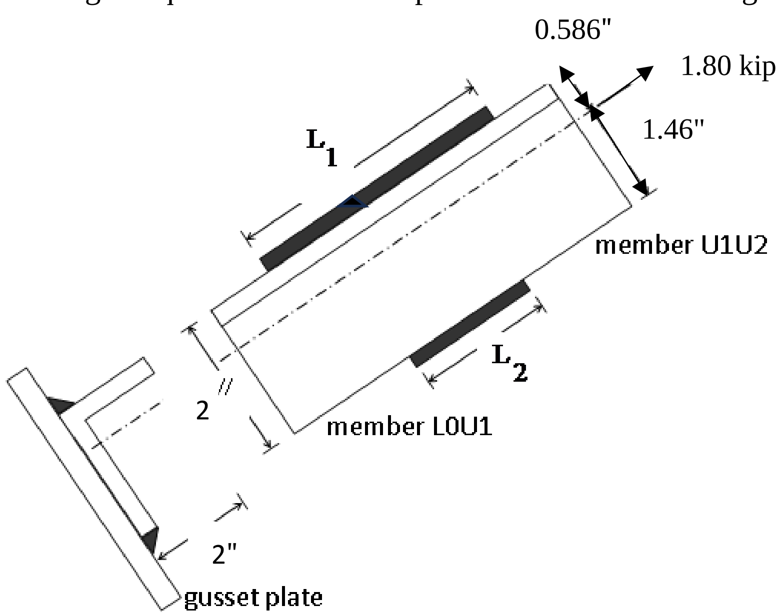


Figure 2.24: Weld design of member L_0U_1 and U_1U_2

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, $(t_e) = s \times \cos 45^0 = \frac{3}{16} \times \cos 45^0$ inch.

Weld length required for member L0U1U2,

$$L_{L_0U_1U_2} = \frac{|P_{maximum}|_{tensile\ or\ compressive}}{F_v \times t_e} = \frac{1.80\ \text{kip}}{18\ \text{ksi} \times \frac{3}{16} \cos 45^0\ \text{inch}}$$

$$\Rightarrow L_{L_0U_1U_2} = 0.7542\ \text{inch}$$

$$\Rightarrow L_1 + L_2 = L_{L_0U_1U_2} = 0.7542\ \text{inch}$$

Taking moment about L_2,

$$\Rightarrow (L_1 \times t_e \times F_v) \times (2^{//}) = (1.80\ \text{kip}) \times (1.46^{//})$$

$$\Rightarrow (L_1 \times \frac{3}{16} \cos 45^0\ \text{inch} \times 18\ \text{ksi}) \times (2^{//}) = (1.80\ \text{kip}) \times (1.46^{//})$$

$$\Rightarrow L_1 = 0.551\ \text{inch} \approx 1.0\ \text{inch}$$

Taking moment about L_1,

$$\Rightarrow (L_2 \times t_e \times F_v) \times (2^{//}) = (1.80\ \text{kip}) \times (0.589^{//})$$

$$\Rightarrow (L_2 \times \frac{3}{16} \cos 45^0\ \text{inch} \times 18\ \text{ksi}) \times (2^{//}) = (1.80\ \text{kip}) \times (0.589^{//})$$

$$\Rightarrow L_2 = 0.22\ \text{inch} \approx 0.5\ \text{inch}$$

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

$L_1 > L_{minimum}$; OK $L_2 < L_{minimum}$ so L_2 =0.75 inch

Alternatively, $L_1 + L_2 = 0.7542$ inch & $\frac{L_1}{L_2} = \frac{1.46\ \text{inch}}{0.586\text{inch}}$; from which, $L_1 = 0.538$ inch & L_2 = 0.216 inch. So L_1 and L_2 will be the minimum size of weld length.

Weld for L1U1:

Consider, L1U1 ($L \quad 2\frac{1}{2} \times 2 \times \frac{1}{4}$) & 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, $s_{max} = (\frac{1}{4} - \frac{1}{16})$inch = $\frac{3}{16}$ inch (from table 2, chapter: welded connections)

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

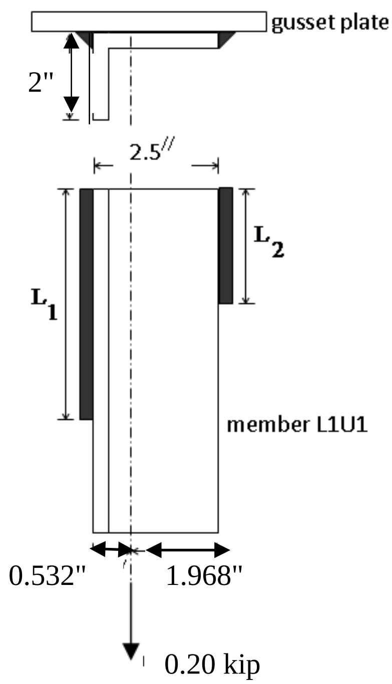


Figure 2.25: Weld design of member L_1U_1

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,

$$L_{L_1U_1} = \frac{\left|P_{maximum}\right|_{tensile\ or\ compressive}}{F_v \times t_e} = \frac{0.20\ kip}{18\ ksi \times \frac{3}{16} \cos 45^0\ inch}$$

$\Rightarrow L_{L_1U_1}$ = 0.084 inch

$\Rightarrow L_1 + L_2 = L_{L_1U_1}$ = 0.084 inch

Taking moment about L_2,

$\Rightarrow (L_1 \times t_e \times F_v) \times (2.5^{//}) = (0.20\ kip) \times (1.968^{//})$

$\Rightarrow (L_1 \times \frac{3}{16}\ \cos 45^0\ inch \times 18\ ksi) \times (2.5^{//}) = (0.20\ kip) \times (1.968^{//})$

$\Rightarrow L_1$ = 0.066 inch ≈ 0.10 inch

Taking moment about L_1,

$\Rightarrow (L_2 \times t_e \times F_v) \times (2.5^{//}) = (0.20\ kip) \times (0.532^{//})$

$\Rightarrow (L_2 \times \frac{3}{16}\ \cos 45^0\ inch \times 18\ ksi) \times (2.5^{//}) = (0.20\ kip) \times (0.532^{//})$

$\Rightarrow L_2$ = 0.018 inch ≈ 0.05 inch

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

Both L_1 & $L_2 < L_{minimum}$; So, L_1 & L_2 will be the minimum size of weld length.

Alternatively, $L_1 + L_2$ = 0.084 inch & $\frac{L_1}{L_2} = \frac{1.968\ inch}{0.532\ inch}$; from which, L_1 = 0.0179 inch & L_2 = 0.066 inch. The weld length of L_1 & L_2 will be 0.75 inch.

Weld for U1L2:

Consider, U1L2 (L $2 \times 2 \times \frac{1}{8}$) & gusset plate ($\frac{6}{16}$ inch)

$t_{max} = \frac{6}{16}$ inch and $t_{min} = \frac{1}{8}$ 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, $t_{min} = \frac{1}{8}$ inch. So, Maximum fillet weld size, $s_{max} = \frac{1}{8}$ inch (from table 2, chapter: welded connections)

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

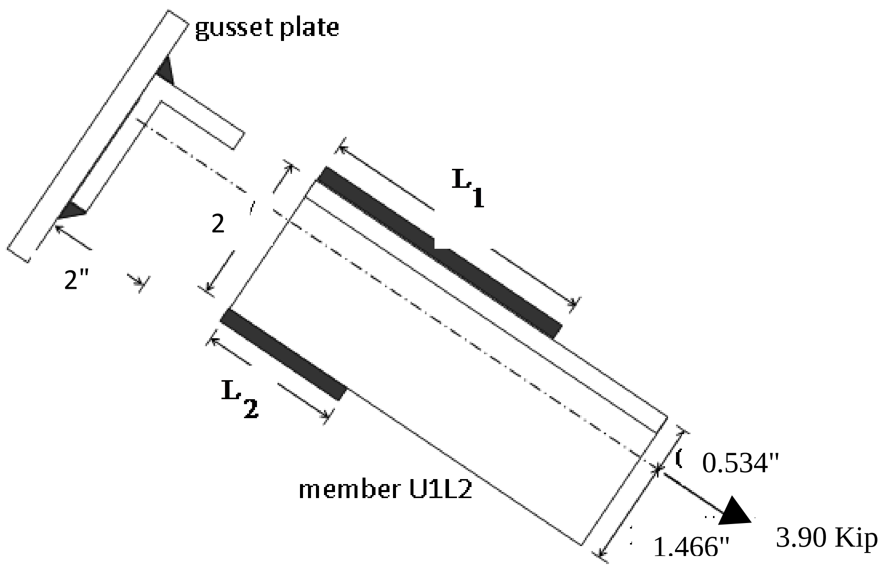


Figure 2.26: Weld design of member U_1L_2

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, $(t_e) = s \times \cos 45^0 = \frac{3}{16} \times \cos 45^0$ inch.

Weld length required for member L1U2,

$$L_{L_1U_1} = \frac{\left|P_{maximum}\right|_{tensile\ or\ compressive}}{F_v \times t_e} = \frac{3.90\ \text{kip}}{18\ \text{ksi} \times \frac{3}{16}\cos 45^0\ \text{inch}}$$

$\Rightarrow L_{L_1U_1} = 1.634$ inch

$\Rightarrow L_1 + L_2 = L_{L_1U_1} = 1.634$ inch

Taking moment about L_2,

$\Rightarrow (L_1 \times t_e \times F_v) \times (2^{//}) = (3.90\ \text{kip}) \times (1.466^{//})$

$\Rightarrow (L_1 \times \frac{3}{16} \cos 45^0\ \text{inch} \times 18\ \text{ksi}) \times (2^{//}) = (3.90\ \text{kip}) \times (1.466^{//})$

$\Rightarrow L_1 = 1.197$ inch ≈ 1.5 inch

Taking moment about L_1,

$\Rightarrow (L_2 \times t_e \times F_v) \times (2^{//}) = (3.90\ \text{kip}) \times (0.538^{//})$

$\Rightarrow (L_2 \times \frac{3}{16} \cos 45^0\ \text{inch} \times 18\ \text{ksi}) \times (2^{//}) = (3.90\ \text{kip}) \times (0.538^{//})$

$\Rightarrow L_2 = 0.4396$ inch ≈ 0.5 inch

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

Both $L_1 > L_{minimum}$; OK $L_2 < L_{minimum}$; Not Ok that's why $L_2 = 0.75$ inch

Alternatively, $L_1 + L_2 = 1.634$ inch & $\frac{L_1}{L_2} = \frac{1.466\ \text{inch}}{0.534\text{inch}}$; from which, $L_1 = 0.436$ inch & $L_2 = 1.197$ inch.

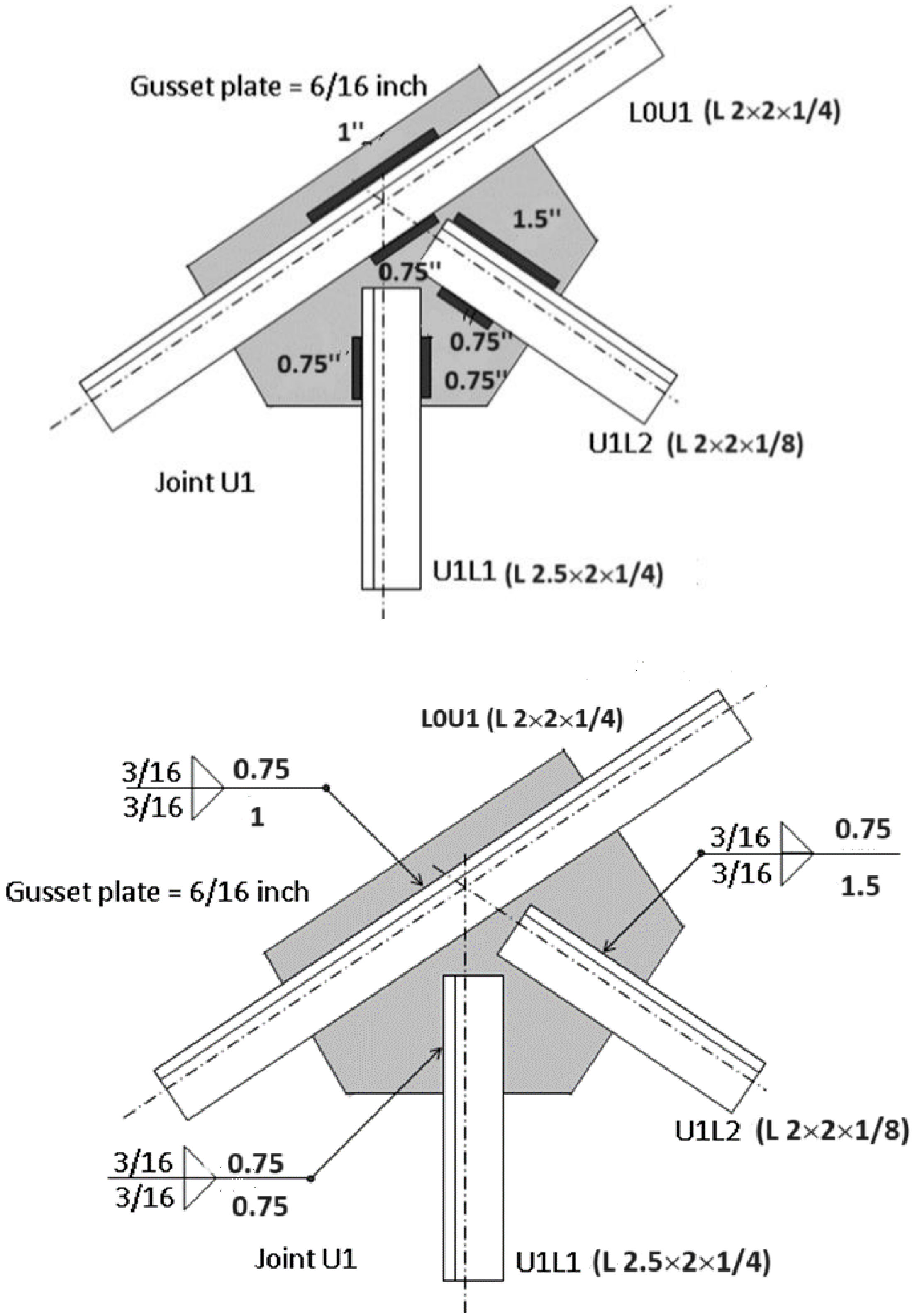


Figure 2.27: Weld design of joint U_1

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	3.714 ↑	7.957 ↓ 2.460 ←	9.803 ↓ 2.460 →	3.714 (C)	4.243 (T) 2.46 (S)	6.083 (T) 2.460 (S)
L_6	3.714 ↑	9.803 ↓	7.957 ↓	3.714 (C)	6.083 (T)	4.243 (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 = 6.083/(10*20) = 0.030 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 = 3.714

Ap = 3.714/1.05

Ap = 3.54 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.

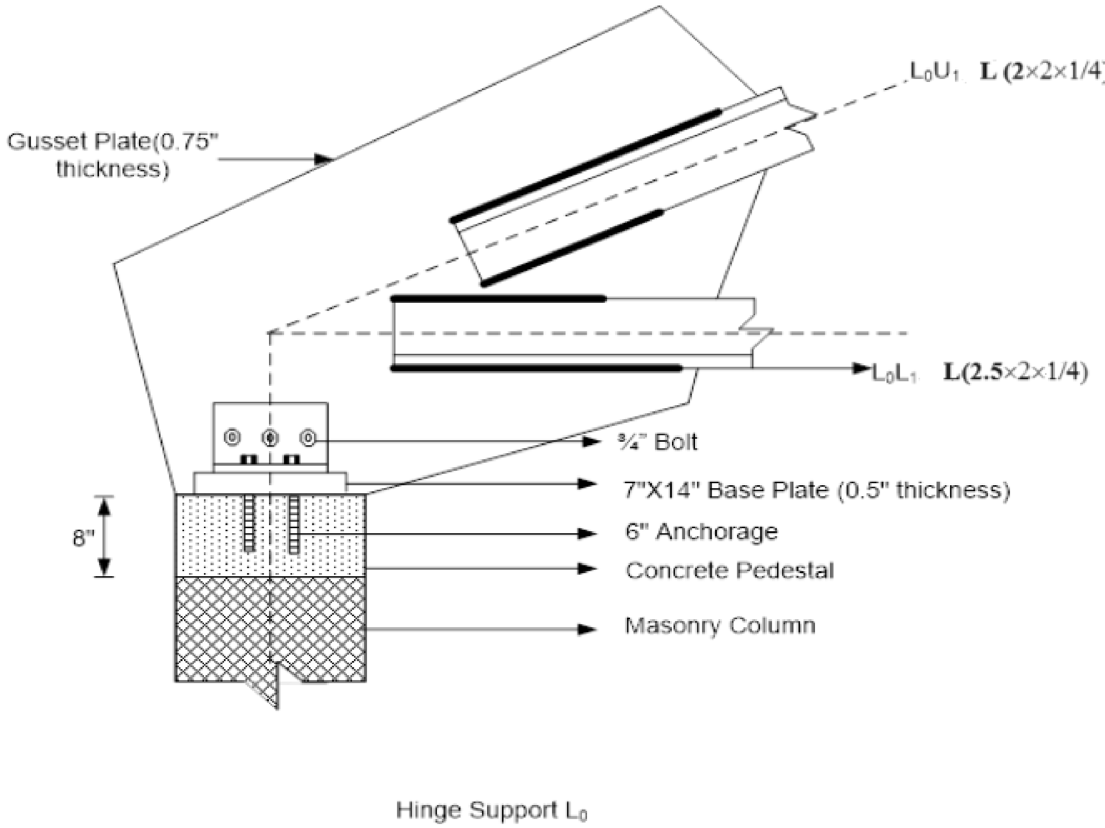


Figure 2.28: Design of anchorage at hinge support L_0

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) =6.083/ (4*20) = 0.076 in^2

Required area (based on shear force) = 2.460/ (4*12) = 0.051 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

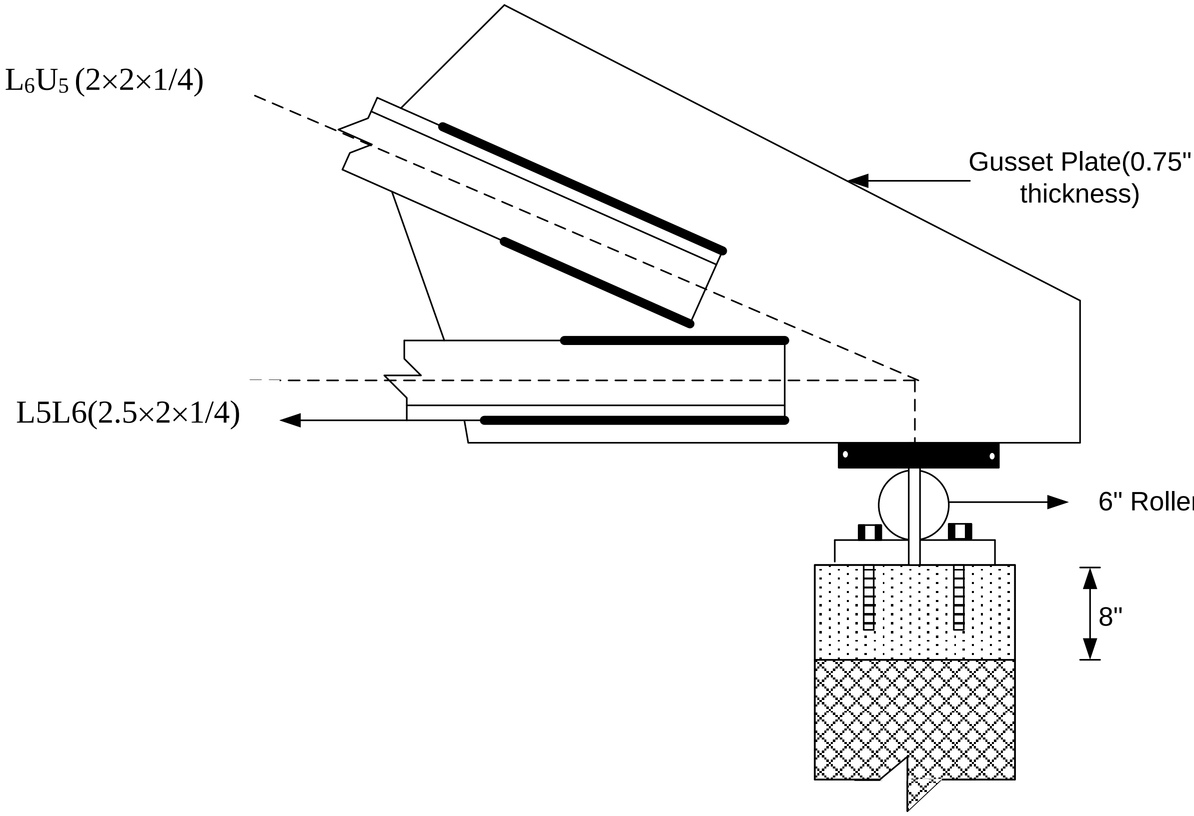
Allowable bond force per unit length = 35√fc' = 35√3000 lb/in = 1.92 k/in

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 (=6.083 kips) by shear.

Required area = 6.083/12 = 0.51 in^2, i.e., provide 3-3/4" diameter bolts in double shear.



Roller Support L_6

Figure 2.29: 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.

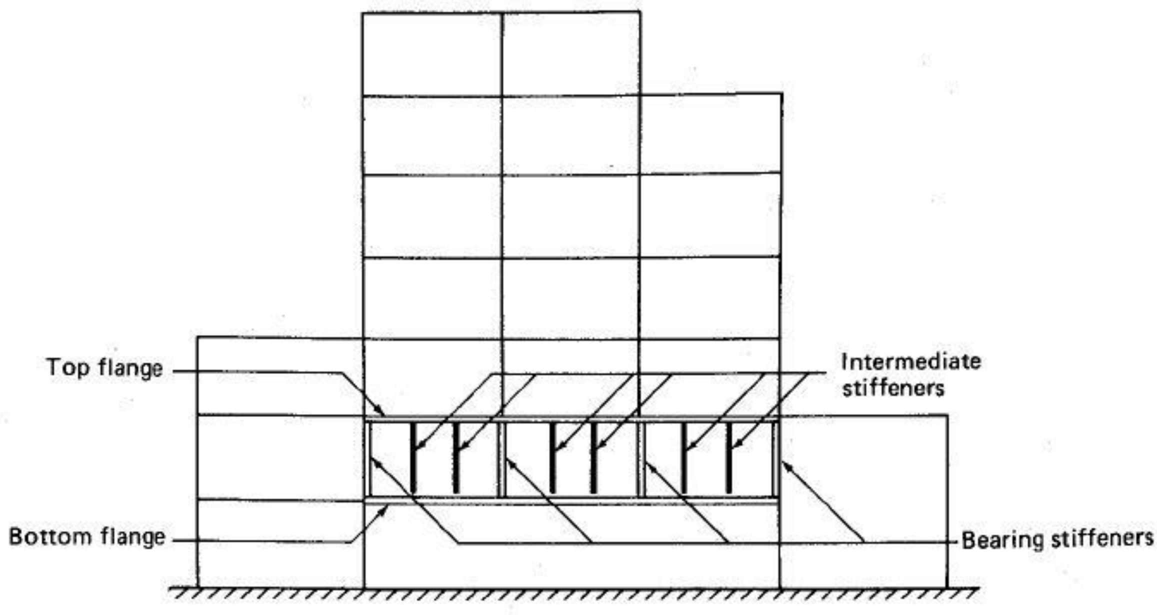


Figure 3.1: Plate girder in a multistory building

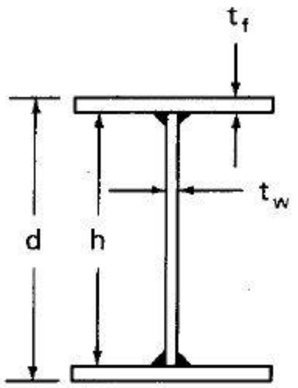


Fig.3.2(a): Welded plate girder

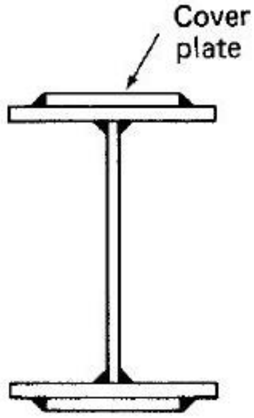


Fig. 3.2(b): Plate girder with cover plate

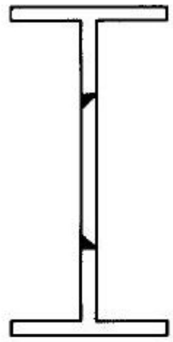


Fig. 3.3(c): Built-up girder with T sections

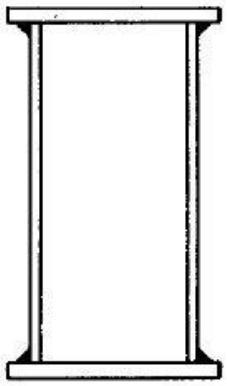


Fig. 3.2(d): Welded box girder

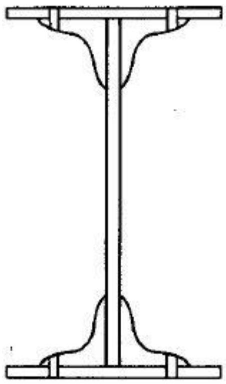


Fig. 3.2(e): Riveted or bolted plate girder

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.

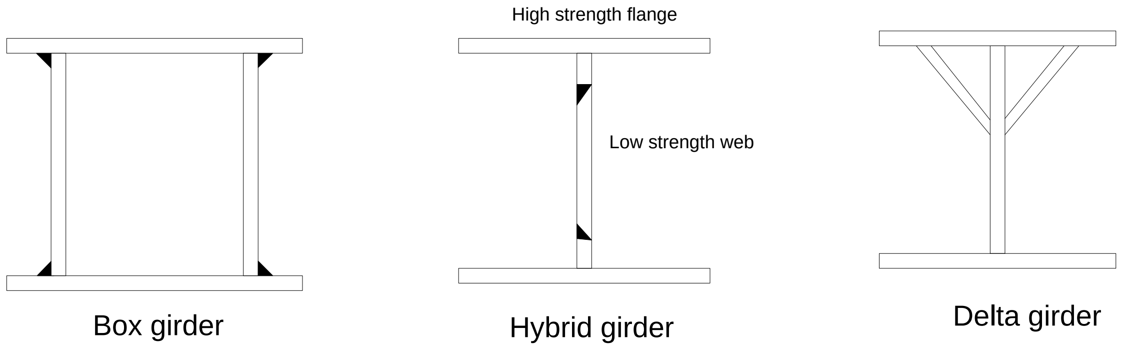
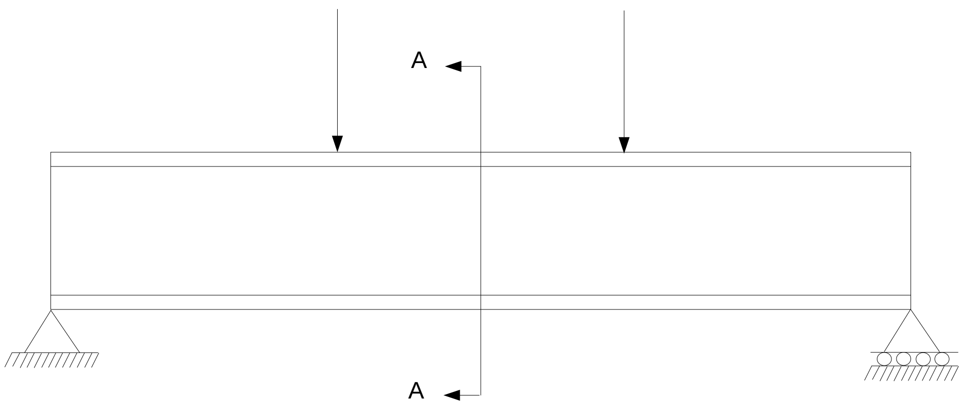


Figure 3.3: Types of plate girder

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

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

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

Intermediate Stiffener: 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.

Bearing Stiffener: 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.

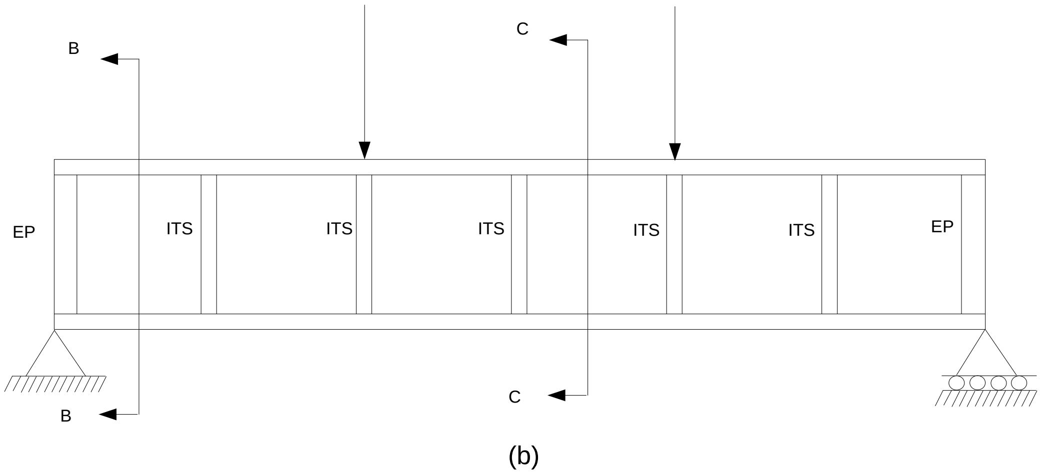


Figure 3.4(b): Plate girder with intermediate and bearing stiffeners

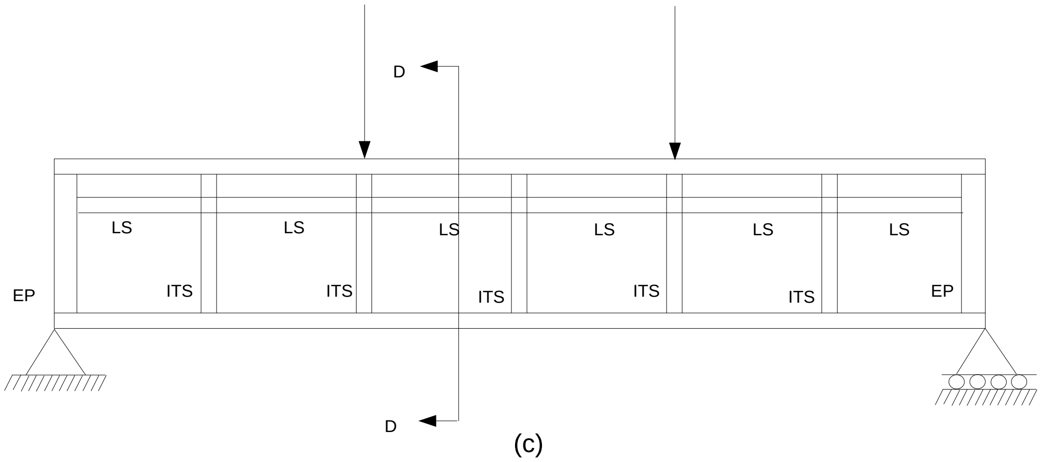


Figure 3.4(c): Plate girder with intermediate, bearing and lateral stiffeners

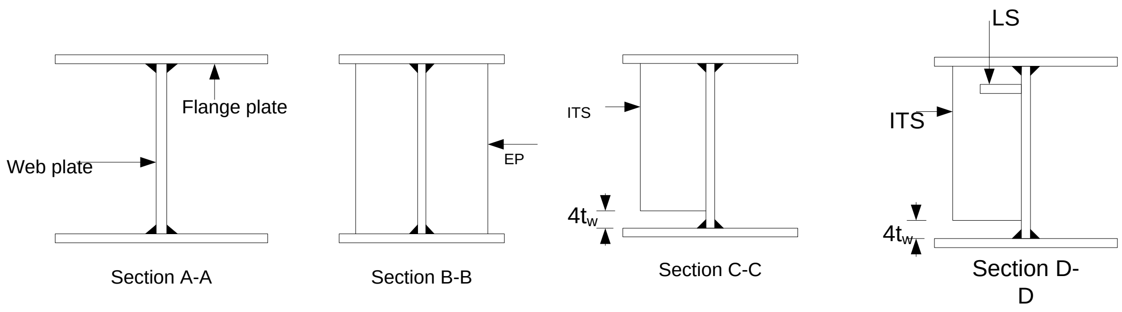


Figure 3.4(d): Forms of plate girder

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. 30(a). It will then be able to carry additional loads. A stiffened plate girder has considerable post-buckling strength.

The Pratt truss of Fig. 33(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 33(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.

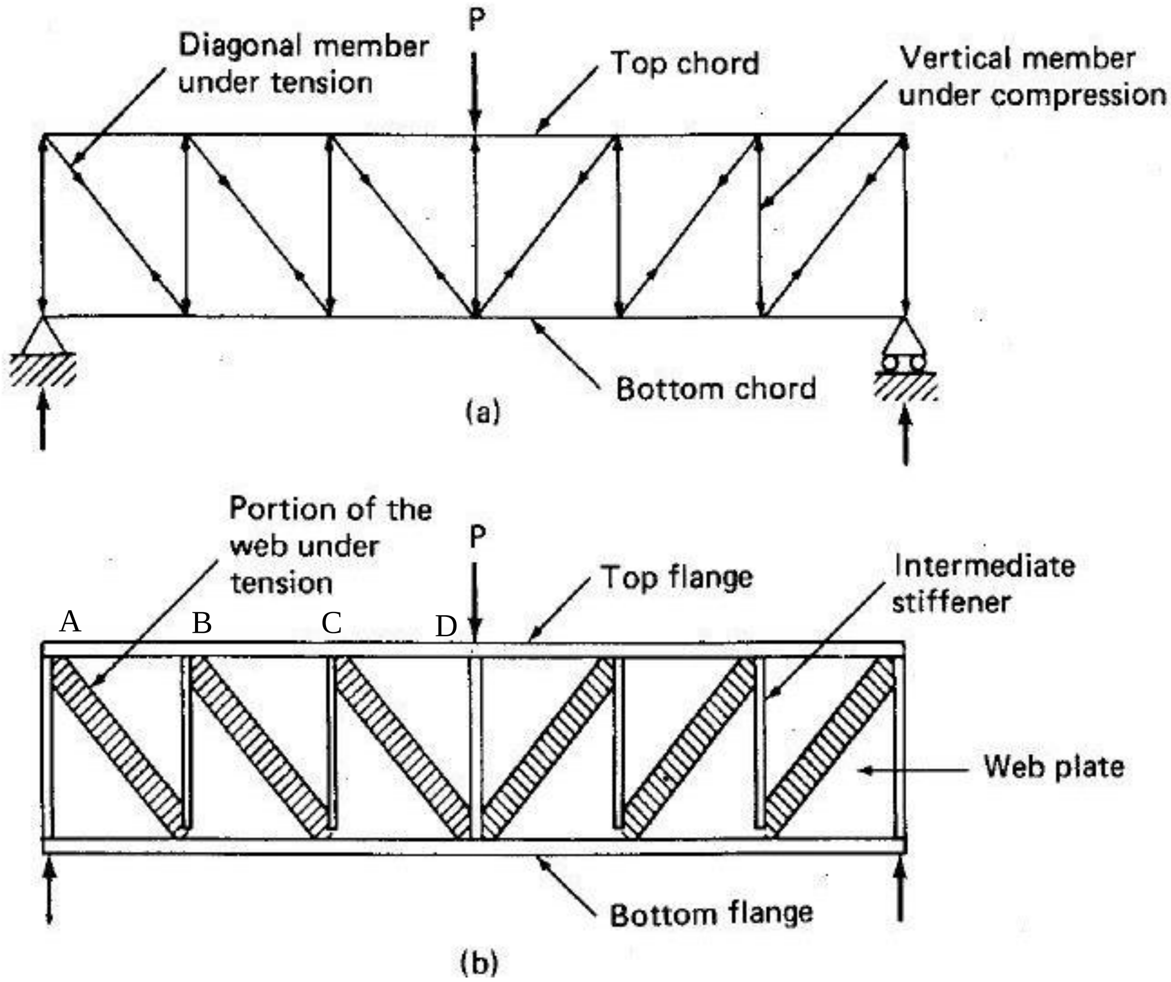


Figure 3.5: Analogy between a truss and a stiffened plate girder

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

$$3.76\sqrt{\frac{E}{F_y}} < \frac{h}{t_w} \leq 5.70\sqrt{\frac{E}{F_y}} \qquad \text{.....................(Eq-3.1)}$$

and the web is slender if

$$\frac{h}{t_w} > 5.70\sqrt{\frac{E}{F_y}} \qquad \text{.....................(Eq-3.2)}$$

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

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

Where, a is the clear distance between stiffeners.

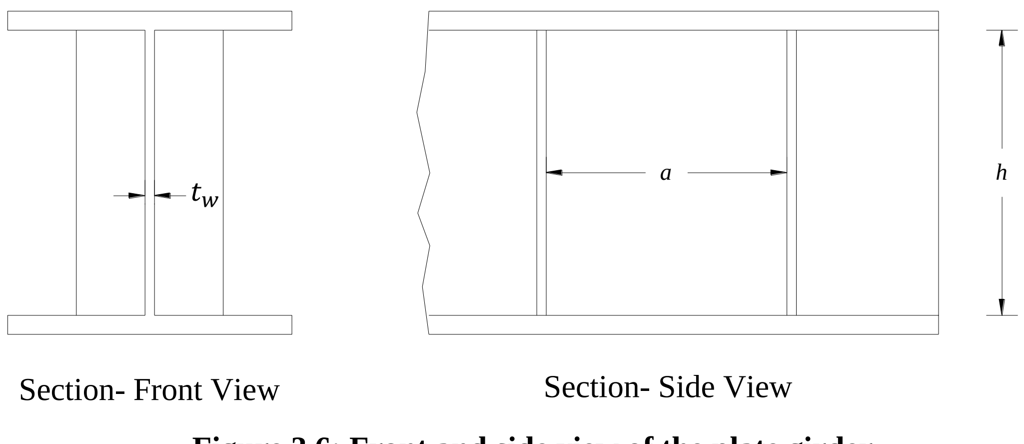


Figure 3.6: Front and side view of the plate girder

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

$M_n = F_y S_{xt}$ ....(Eq-3.5)

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

Compression Flange Yielding

The compression flange nominal strength is given by,

$$M_n = R_{pg} F_{cr} S_{xc} \qquad \text{…………………(Eq-3.6)}$$

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

$$R_{pg} = 1 - \frac{a_w}{1{,}200 + 300a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \le 1.0 \qquad \text{…………………(Eq 3.7)}$$

where

$$a_w = \frac{h_c\, t_w}{b_{fc} t_{fc}} \le 10 \qquad \text{…………………(Eq 3.8)}$$

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,

$$\lambda = \frac{b_f}{2t_f} \qquad \text{…………………(Eq 3.9)}$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} \qquad \text{…………………(Eq 3.10)}$$

$$\lambda_r = 0.95\sqrt{\frac{k_c E}{F_L}} \qquad \text{…………………(Eq 3.11)}$$

$$k_c = \frac{4}{\sqrt{h/t_w}} \text{ but } (0.35 \le k_c \le 0.76) \qquad \text{…………………(Eq 3.12)}$$

$F_L = 0.7F_y$ for girders with slender webs. …………………(Eq 3.13)

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

$$F_{cr} = F_y - 0.3F_y\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \qquad \text{…………………(Eq 3.14)}$$

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

$$F_{cr} = \frac{0.9Ek_c}{(\frac{b_f}{2t_f})^2} \quad \text{…………………(Eq-3.15)}$$

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

$$L_p = 1.1r_t\sqrt{\frac{E}{F_y}} \quad \text{…………………(Eq3.16)}$$

$$L_r = \pi r_t\sqrt{\frac{E}{0.7F_y}} \quad \text{…………………(Eq3.17)}$$

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

$$F_{cr} = C_bF_y - 0.3F_y\left(\frac{L_b-L_p}{L_r-L_p}\right) \leq F_y \quad \text{…………………(Eq-3.18)}$$

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

$$F_{cr} = \frac{C_b\pi^2E}{(\frac{L_b}{r_t})^2} \leq F_y \quad \text{…………………(Eq-3.19)}$$

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,

$$k_v = 5 + \frac{5}{(a/h)^2} \quad \text{…………………(Eq 3.20)}$$

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

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

$= 5$ in unstiffened webs with $\frac{h}{t_w} < 260$ …………………(Eq-3.21)

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

If $\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E}{F_y}}$, $\quad C_v = 1.0$ …………………(Eq-3.22)

If $1.10\sqrt{\frac{k_v E}{F_y}} < \frac{h}{t_w} \leq 1.37\sqrt{\frac{k_v E}{F_y}}$, $C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w}$ …………………(Eq-3.23)

If $\frac{h}{t_w} > 1.37\sqrt{\frac{k_v E}{F_y}}$, $\quad C_v = \frac{1.51 k_v E}{(h/t_w)^2 F_y}$ …………………(Eq-3.24)

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

$$\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v E}{F_y}}$$ …………………(Eq-3.25)

the strength is based on shear yielding, and $V_n = 0.6 F_y A_w$ …………………(Eq-3.26)

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

$$\frac{h}{t_w} > 1.10\sqrt{\frac{k_v E}{F_y}},$$ …………………(Eq-3.27)

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

$$V_n = 0.6 F_y A_w C_v + 0.6 F_y A_w \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}$$ …………………(Eq-3.28)

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

$$V_n = 0.6 F_y A_w C_v$$ …………………(Eq-3.29)

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

$$I_{st} \geq bt_w^3 j \qquad \text{.....................(Eq-3.30)}$$

Where,

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

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.

$$\left(\frac{b}{\mathrm{t}}\right)_{st} \leq 0.56\sqrt{\frac{E}{F_{yst}}} \qquad \text{.....................(Eq-3.31)}$$

Where,

$\left(\frac{b}{\mathrm{t}}\right)_{st}$ =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.

$$I_{st} \geq I_{st1} + (I_{st2} - I_{st1})\frac{V_r - V_{c1}}{V_{c2} - V_{c1}} \qquad \text{.....................(Eq-3.32)}$$

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

$$= \frac{h^4 p_{st}^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5} \qquad \text{.....................(Eq-3.33)}$$

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

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/Ω_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,

$R_n = 1.8F_y A_{pb}$ …………………(Eq-3.34)

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:

For $\frac{KL}{r} \leq 25$ …………………(Eq-3.35)

$$P_n = F_y A_g$$

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:

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

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 $\frac{\mathrm{h}}{t_w}$ 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

$$\boxed{A_f = \frac{M_{nreq}}{hF_y} - \frac{A_w}{6}} \qquad \text{…………………(Eq-3.36)}$$

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:

$$\left(\frac{b}{t}\right)_{st} \leq 0.56\sqrt{\frac{E}{F_{yst}}} \qquad \text{…………………(Eq-3.37)}$$

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

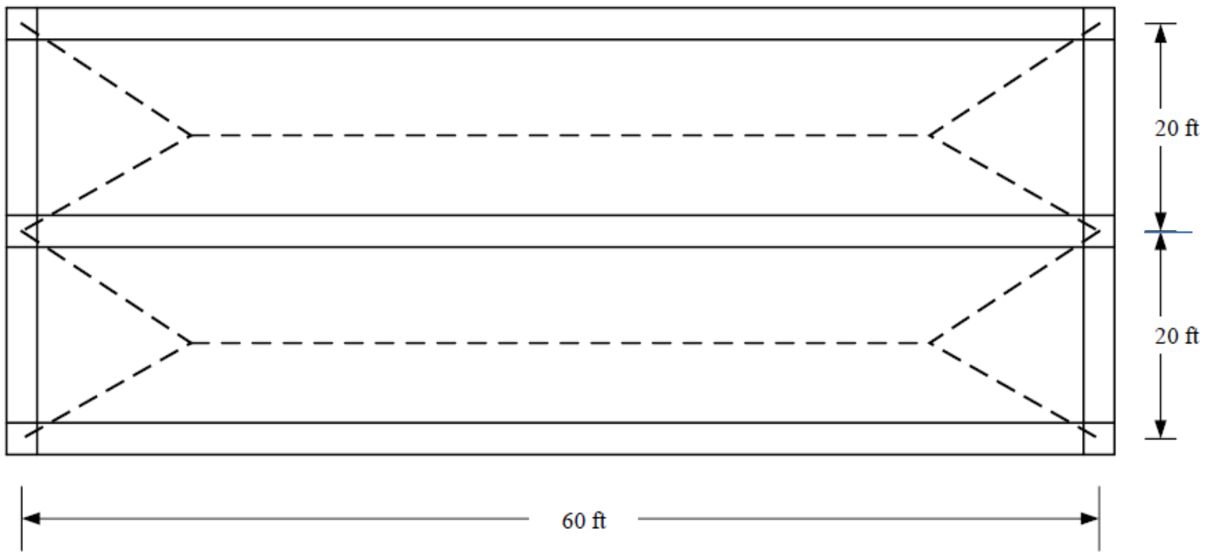


Fig 3.7: Beam Layout

In a simply supported plate girder,
Span length, L = 60 ft
maximum permissible depth= 65 in
Service Loads:
Uniformly distributed loads, $w_D = 1.70$ kip/ft & $w_L = 1.25$ kip/ft
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 ft^2

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

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

∴ Service live load, w_L= 75 (1000/ 60) (1 /1000) = 1.25 k/ft

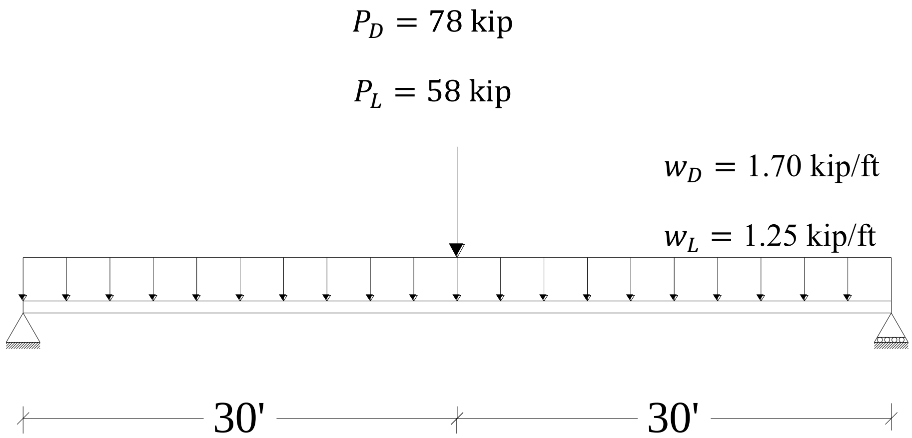


Figure 3.8: Plate girder to be designed under service loads

LRFD Solution:

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

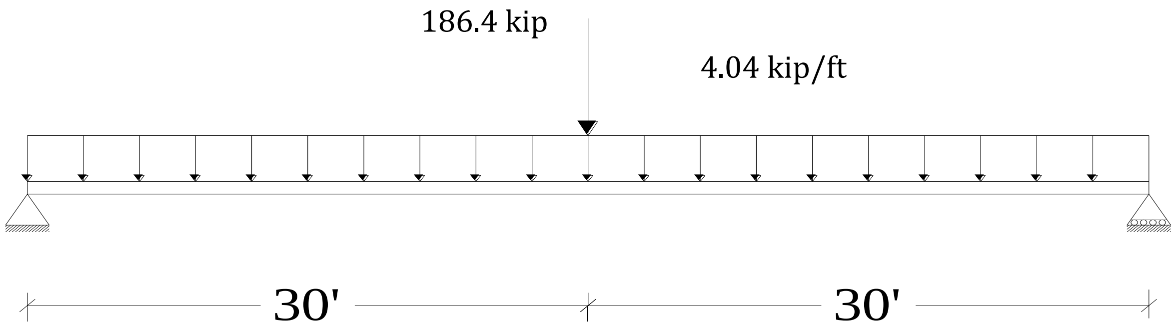


Figure 3.9: Factored Loads (excluding girder self-weight)

Selection of Web Plate:

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.

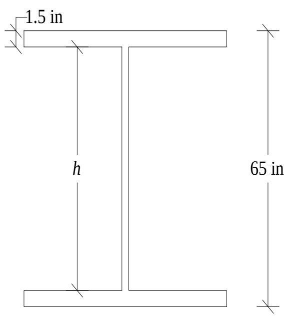


Figure 3.10: Cross-section of the plate girder

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

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

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

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

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

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

Try a $^5/_{16} \times$ 62 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.2, 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, $A_f = \frac{M_{nreq}}{hF_y} - \frac{A_w}{6}$

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

$$= \frac{(4614\times12)/0.90}{62(36)} - \frac{62(5/16)}{6} = 24.33 \text{ in}^2$$

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

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

Try a $1\,{}^{1}/_{2} \times 18$ 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

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

Figure 10.17 shows the trial section, and Figure 10.18 shows the shear and bending moment diagrams for the factored loads, which include the girder weight of 250 lb/ft.

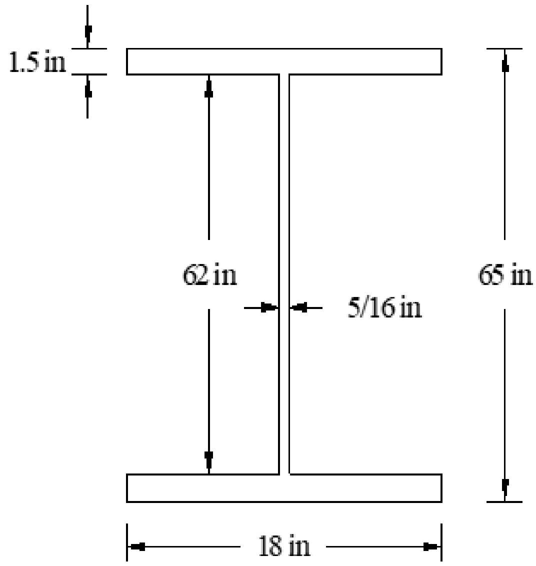


Figure 3.11: Cross-section of the plate

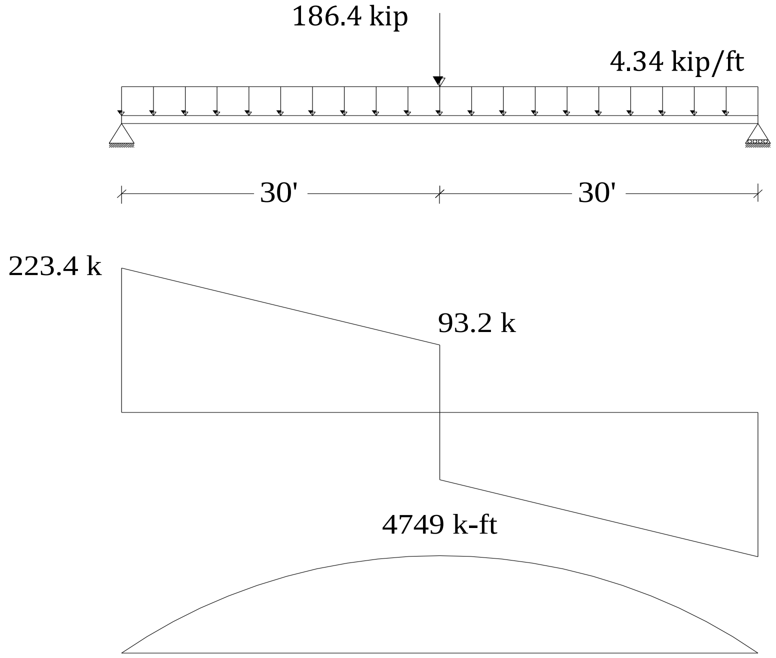


Figure 3.12: SFD and BMD for the recomputed factored loads

Check the flexural strength of the trial section:

From Figure 38, the moment of inertia about the axis of bending is

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

and the elastic section modulus is

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

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-3.7

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

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

$$= 0.9814$$

From AISC Equation 3.6, the nominal flexural strength for the compression flange is

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

and the design strength is

$\varphi_b M_n = 0.90(5494) = 4945$ ft-kips > 4749 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:

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

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

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

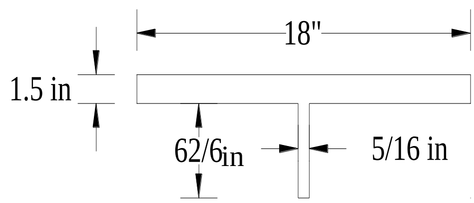


Figure 3.13: Zone corresponding to lateral torsional buckling

Check the unbraced length.

$L_b = 30$ ft

$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} = 1.1(4.91)\sqrt{\frac{29{,}000}{36}} = 153.2$

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

Since $L_p < L_b < L_r$, the girder is subject to inelastic lateral torsional buckling. From Equation-3.18,

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

For the computation of C_b, refer to Figure 36, 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,

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

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

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

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

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-3.20,

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

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

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

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

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

So, from Equation-3.24, $C_v = \frac{1.51 k_v E}{(h/t_w)^2 F_y} = 0.61$

As no tension field action occurs at the end panel, from equation-3.29

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

$\varphi V_n = 0.9\ (255.285) = 229.76$ 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

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

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

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

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

(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 38.

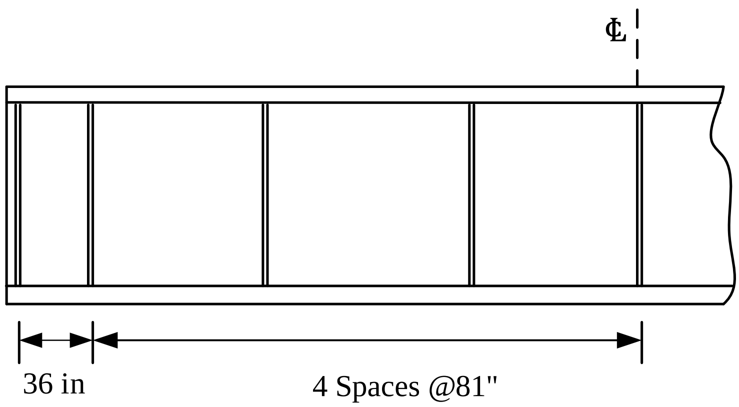


Figure 3.14 : Spacing of Intermediate stiffeners

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 Equation-3.31, 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 Equation-3.32 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 Equation-3.30, 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

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

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

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

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 10.18, the larger required strength in the two adjacent panels is

$V_r = V_u = 223.4$ kips.

From AISC Equation 3.32,

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

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

Try two $3/8 \times 4$ 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 10.9):

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

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

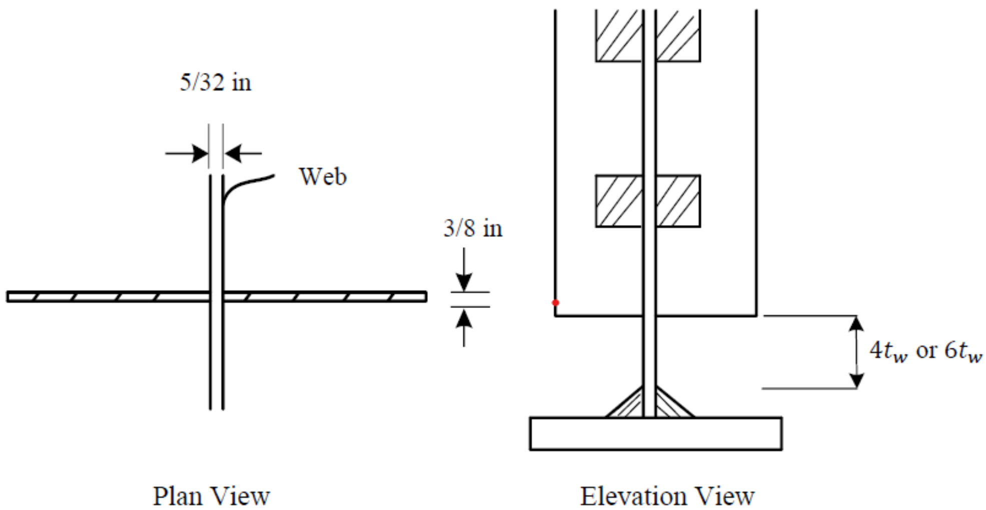


Figure 3.15: Plan and Elevation View of Intermediate Stiffeners

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-3.31

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

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

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.

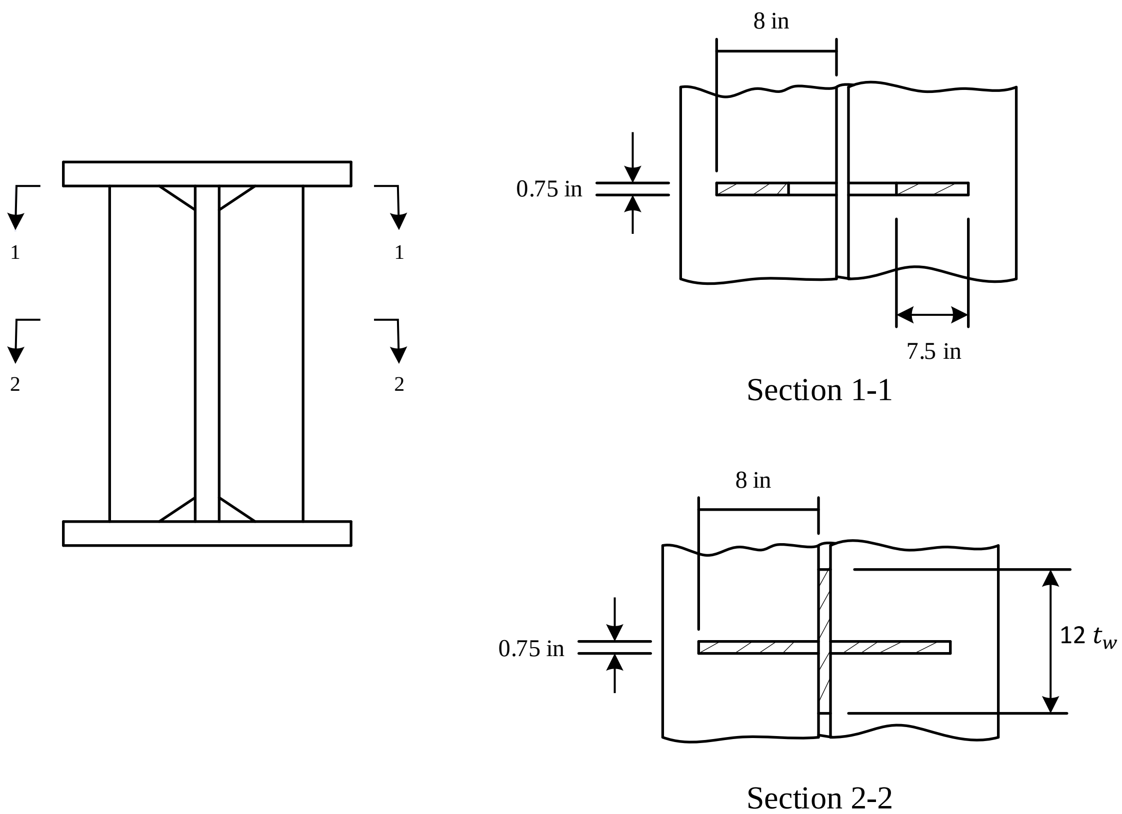


Figure 3.16: Bearing Stiffener at the Support

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 40, 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 41, but base the computations on a total length of web of 3.75 inches, which gives

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

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

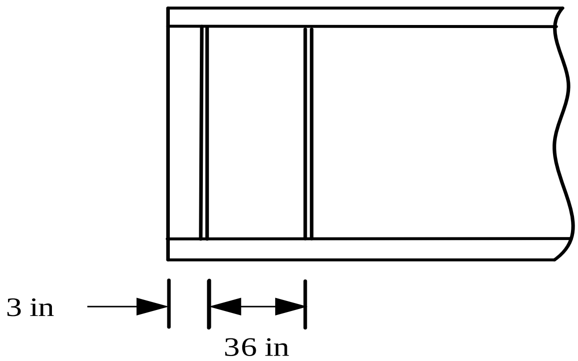


Figure 3.17: Side view of the plate girder

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

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

$P_n = F_yA_g = 36(13.17) = 474.1$ kips

$\varphi P_n = 0.90(474.1) = 427$ kips > 223.4 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 $3/4 \times 8 \times 5' - 2''$

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 35)

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

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

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

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

$= 4\ (\frac{3}{16}) = 0.75$ 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}) …………………(Eq-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 × 3 sixteenths × 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 …………………(Eq-3.35)

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

The base metal shear rupture strength per unit length is

$\varphi R_n = 0.45 F_u t$ ....(Eq-3.36)

$= 0.45(58)(\frac{5}{16}) = 8.156$ 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

$$V_\mathrm{u} = 223.4 - 4.34x = 143.3 \text{ kips}$$

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

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. $\times$ $1\,{}^{1}/_{2}$ in. fillet welds for the flange to web welds, spaced as shown in Figure 42.

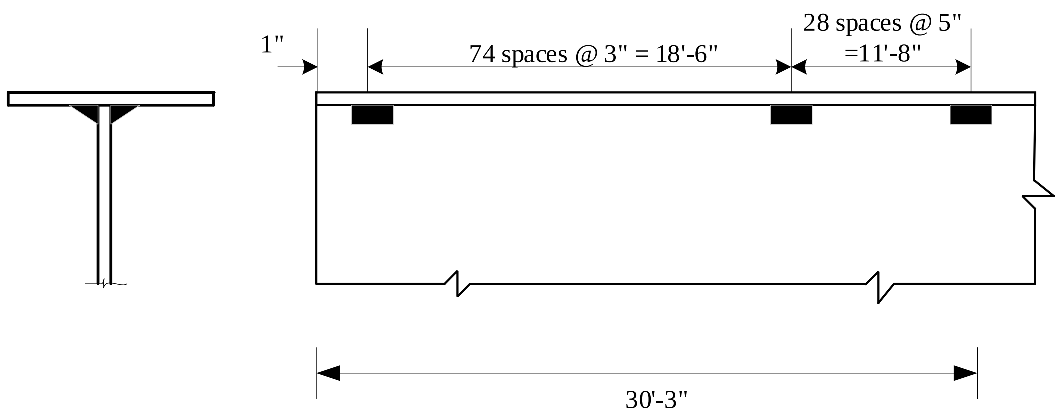


Figure 3.18: Weld between Flange and web

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

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

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

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

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

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

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

$$5.400 \times 2 = 10.80 \text{ kips/in.}$$

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

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

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

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

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

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

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

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

$$4.5 - 1.5 = 3 \text{ in.} < 5 \text{ in. (OK)}$$

Use 3/16-in. × $1\,{}^{1}/_{2}$-in. fillet welds for intermediate stiffeners, spaced as shown in Figure 43.

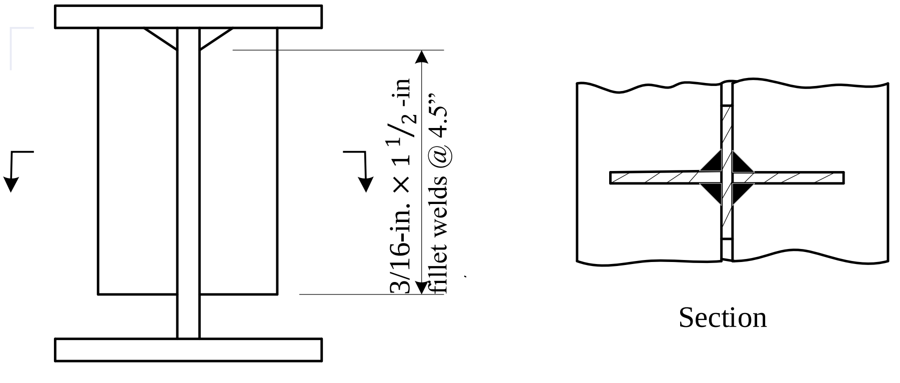


Figure 3.19: Weld between web and intermediate stiffener

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.

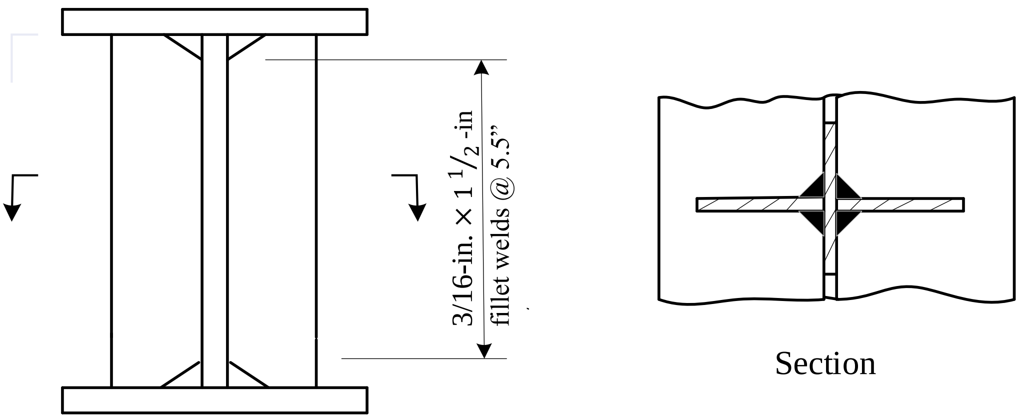


Figure 3.20: Weld between web and bearing stiffener

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-

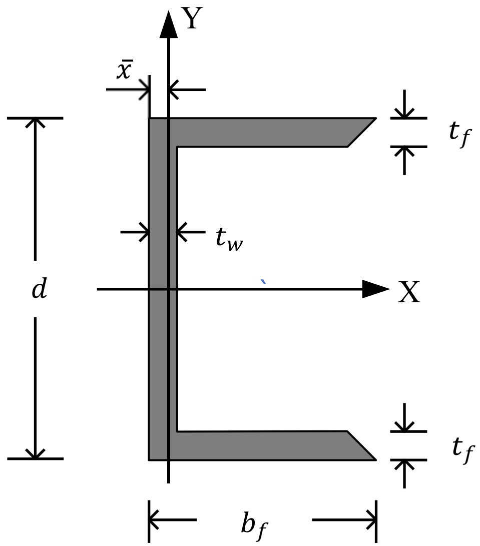
AISC, Plate Sizes

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), 9^{th} edition, Page 59.

American Standard Channel Section (C- Section) Properties



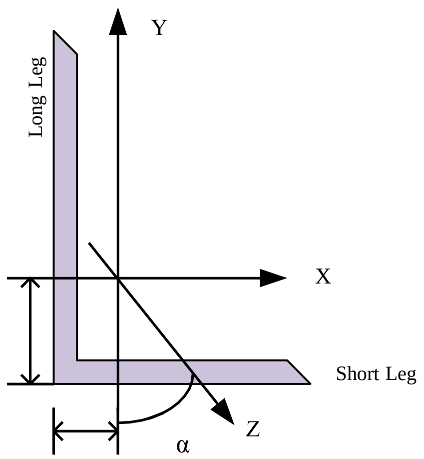
Designation	Nominal weight per foot $lb.$	Area A (in^2)	Depth d (in)	Flange width b_f (in)	Web thickness t_w (in)	Flange thickness t_f (in)	$\bar{x}$ (in)	Axis X-X			Axis Y-Y		
								I_x (in^4)	S_x (in^3)	r_x (in)	I_y (in^4)	S_y (in^3)	r_y (in)
C15×50	50.0	14.7	15.0	3.72	0.716	0.650	0.799	404	53.8	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

Open Structures: Trussed Tower

Tower Cross Section	C_f
Square	$4.0\ \epsilon^2 - 5.9\ \epsilon + 4.0$
Triangle	$3.4\ \epsilon^2 - 4.7\ \epsilon + 3.4$

Notes:

1. For all wind directions considered, the area A_f consistent with the specified force coefficients shall be the solid area of a tower face projected on the plane of that face for the tower segment under consideration.
2. The specified force coefficients are for towers with structural angles or similar flat-sided members.
3. For towers containing rounded members, it is acceptable to multiply the specified force coefficients by the following factor when determining wind forces on such members: $0.51\ \epsilon^2 + 0.57 \leq 1.0$
4. Wind forces shall be applied in the directions resulting in maximum member forces and reactions. For towers with square cross-sections, wind forces shall be multiplied by the following factor when the wind is directed along a tower diagonal:

   $1 + 0.75\ \epsilon \leq 1.2$
5. Wind forces on tower appurtenances such as ladders, conduits, lights, elevators, etc., shall be calculated using appropriate force coefficients for these elements.
6. Notation:

   ϵ : ratio of solid area to gross area of one tower face for the segment under consideration.

Figure 6.2.23 Force coefficient, C_f for other structures - Method 2 (All heights)

Table 6.2.9: Importance Factor, I (Wind Loads)

Occupancy Category[1] or Importance Class	Non-Cyclone Prone Regions and Cyclone Prone Regions with V = 38-44 m/s	Cyclone Prone Regions with V > 44 m/s
I	0.87	0.77
II	1.0	1.00
III	1.15	1.15
IV	1.15	1.15

[1] The building and structure classification categories are listed in Table 6.1.1

Table 6.2.10: Terrain Exposure Constants

Exposure	α	z_g (m)	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ (m)	$\bar{\epsilon}$	z_{min} (m)*
A	7.0	365.76	1/7	0.84	1/4.0	0.45	0.30	97.54	1/3.0	9.14
B	9.5	274.32	1/9.5	1.00	1/6.5	0.65	0.20	152.4	1/5.0	4.57
C	11.5	213.36	1/11.5	1.07	1/9.0	0.80	0.15	198.12	1/8.0	2.13

*z_{min}= Minimum height used to ensure that the equivalent height z is greater of $0.6h$ or z_{min}.

For buildings with $h \le z_{min}$, $\bar{z}$ shall be taken as z_{min}.

Table 6.2.11: Velocity Pressure Exposure Coefficients, K_h and K_z

Height above ground level, z	Exposure (Note 1)			
	A		B	C
(m)	Case 1	Case 2	Case 1 & 2	Case 1 & 2
0-4.6	0.70	0.57	0.85	1.03
6.1	0.70	0.62	0.90	1.08
7.6	0.70	0.66	0.94	1.12
9.1	0.70	0.70	0.98	1.16
12.2	0.76	0.76	1.04	1.22
15.2	0.81	0.81	1.09	1.27
18	0.85	0.85	1.13	1.31
21.3	0.89	0.89	1.17	1.34
24.4	0.93	0.93	1.21	1.38
27.41	0.96	0.96	1.24	1.40
30.5	0.99	0.99	1.26	1.43
36.6	1.04	1.04	1.31	1.48
42.7	1.09	1.09	1.36	1.52
48.8	1.13	1.13	1.39	1.55
54.9	1.17	1.17	1.43	1.58
61.0	1.20	1.20	1.46	1.61
76.2	1.28	1.28	1.53	1.68
91.4	1.35	1.35	1.59	1.73
106.7	1.41	1.41	1.64	1.78
121.9	1.47	1.47	1.69	1.82
137.2	1.52	1.52	1.73	1.86
152.4	1.56	1.56	1.77	1.89

Notes:

1. Case 1:

   (a) All components and cladding.

   (b) Main wind force resisting system in low-rise buildings designed using Figure 6.2.10.

   Case 2:

   (a) All main wind force resisting systems in buildings except those in low-rise buildings designed using Figure 6.2.10.

   (b) All main wind force resisting systems in other structures.

2. The velocity pressure exposure coefficient K_z may be determined from the following formula:

   For $4.57 \text{ m} \leq z \leq z_g$: $K_z = 2.01\ (z/z_g)^{2/\alpha}$

   For $z < 4.57$ m: $K_z = 2.01\ (4.57/z_g)^{2/\alpha}$

   Note: z shall not be taken less than 9.1 m for Case 1 in exposure A.

3. α and z_g are tabulated in Table 6.2.10.

4. Linear interpolation for intermediate values of height z is acceptable.

5. Exposure categories are defined in Sec 2.4.6.3.

Table 6.2.12: Wind Directionality Factor, K_d

Structure Type	Directionality Factor K_d *	Structure Type	Directionality Factor K_d *
Buildings		Solid Signs	0.85
Main Wind Force Resisting System	0.85	Open Signs and Lattice Framework	0.85
Components and Cladding	0.85	Trussed Towers	
Arched Roofs	0.85	Triangular, square, rectangular	0.85
Chimneys, Tanks, and Similar Structures		All other cross section	0.95
Square	0.90		
Hexagonal	0.95		
Round	0.95		

* Directionality Factor K_d has been calibrated with combinations of loads specified in Sec 2.7. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.7.2 and 2.7.3.

Exposure B: Exposure B shall apply for all cases where Exposures A or C do not apply.

Exposure C: Exposure C shall apply where the ground surface roughness, as defined by Surface Roughness C, prevails in the upwind direction for a distance greater than 1,524 m or 20 times the building height, whichever is greater. Exposure C shall extend into downwind areas of Surface Roughness A or B for a distance of 200 m or 20 times the height of the building, whichever is greater.

For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces shall be used.

Exception: An intermediate exposure between the preceding categories is permitted in a transition zone provided that it is determined by a rational analysis method defined in the recognized literature.

2.4.6.4 Exposure category for main wind-force resisting system

Buildings and Other Structures: For each wind direction considered, wind loads for the design of the MWFRS determined from Figure 6.2.6 shall be based on the exposure categories defined in Sec 2.4.6.3.

Low-Rise Buildings: Wind loads for the design of the MWFRSs for low-rise buildings shall be determined using a velocity pressure q_h based on the exposure resulting in the highest wind loads for any wind direction at the site where external pressure coefficients GC_{pf} given in Figure 6.2.10 are used.

2.4.6.5 Exposure category for components and cladding

Components and cladding design pressures for all buildings and other structures shall be based on the exposure resulting in the highest wind loads for any direction at the site.

2.4.6.6 Velocity pressure exposure coefficient

Based on the exposure category determined in Sec 2.4.6.3, a velocity pressure exposure coefficient K_z or K_h, as applicable, shall be determined from Table 6.2.11. For a site located in a transition zone between exposure categories that is near to a change in ground surface roughness, intermediate values of K_z or K_h between those shown in Table 6.2.11, are permitted, provided that they are determined by a rational analysis method defined in the recognized literature.

2.4.9.5 Velocity pressure

Velocity pressure, q_z evaluated at height z shall be calculated by the following equation:

$$q_z = 0.000613 K_z K_{zt} K_d V^2 I \; ; \quad (\mathrm{kN/m^2}),\ V \text{in m/s} \qquad (6.2.17)$$

Where K_d is the wind directionality factor, K_z is the velocity pressure exposure coefficient defined in Sec 2.4.6.6, K_{zt} is the topographic factor defined in Sec 2.4.7.2, and q_z is the velocity pressure calculated using Eq. 6.2.17 at mean roof height h. The numerical coefficient 0.000613 shall be used except where sufficient climatic data are available to justify the selection of a different value of this factor for a design application.

2.4.10 Pressure And Force Coefficients

2.4.10.1 Internal pressure coefficients

Internal Pressure Coefficient. Internal pressure coefficients, GC_{pi} shall be determined from Figure 6.2.5 based on building enclosure classifications determined from Sec 2.4.9.

Reduction Factor for Large Volume Buildings, R_i: For a partially enclosed building containing a single, unpartitioned large volume, the internal pressure coefficient, GC_{pi} shall be multiplied by the following reduction factor, R_i:

$$R_i = 1.0 \quad \text{or,} \quad R_i = 0.5\left(1 + \frac{1}{\sqrt{1 + \frac{V_i}{6951 A_{og}}}}\right) \leq 1.0 \qquad (6.2.18)$$

Where, A_{og} = total area of openings in the building envelope (walls and roof, in $\mathrm{m^2}$)

V_i = unpartitioned internal volume, in $\mathrm{m^3}$

2.4.10.2 External pressure coefficients

Main Wind-Force Resisting Systems: External pressure coefficients for MWFRSs C_p are given in Figures 6.2.6 to 6.2.8. Combined gust effect factor and external pressure coefficients, GC_{pf} are given in Figure 6.2.10 for low-rise buildings. The pressure coefficient values and gust effect factor in Figure 6.2.10 shall not be separated.

Components and Cladding: Combined gust effect factor and external pressure coefficients for components and cladding GC_p are given in Figures 6.2.11 to 6.2.17. The pressure coefficient values and gust-effect factor shall not be separated.

2.4.10.3 Force coefficients

Force coefficients C_f are given in Figures 6.2.20 to 6.2.23.

2.4.10.4 Roof overhangs

Main Wind-Force Resisting System: Roof overhangs shall be designed for a positive pressure on the bottom surface of windward roof overhangs corresponding to $C_p = 0.8$ in combination with the pressures determined from using Figures 6.2.6 and 6.2.10.

Components and Cladding: For all buildings, roof overhangs shall be designed for pressures determined from pressure coefficients given in Figure 6.2.11.

2.4.10.5 Parapets

Main Wind-Force Resisting System: The pressure coefficients for the effect of parapets on the MWFRS loads are given in Sec 2.4.12.2.

Components and Cladding: The pressure coefficients for the design of parapet component and cladding elements are taken from the wall and roof pressure coefficients as specified in Sec 2.4.12.3.

2.4.11 Design Wind Loads on Enclosed and Partially Enclosed Buildings

2.4.11.1 General

Sign Convention: Positive pressure acts toward the surface and negative pressure acts away from the surface.

Critical Load Condition: Values of external and internal pressures shall be combined algebraically to determine the most critical load.

Tributary Areas Greater than 65 m^2: Component and cladding elements with tributary areas greater than 65 m^2 shall be permitted to be designed using the provisions for MWFRSs.

2.4.11.2 Main wind-force resisting systems

Rigid Buildings of All Heights: Design wind pressures for the MWFRS of buildings of all heights shall be determined by the following equation:

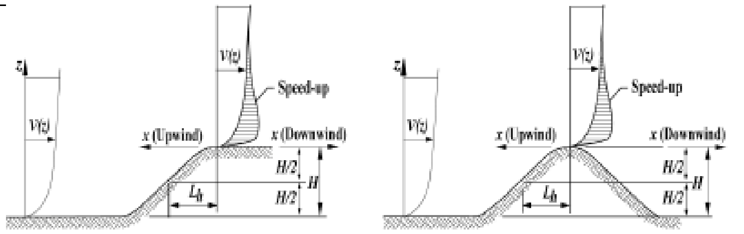
$$p = qGC_p - q_i(GC_{pi}) \quad (\mathrm{kN/m^2}) \qquad (6.2.19)$$

Where,

$q = q_z$ for windward walls evaluated at height z above the ground

$q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h

$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs of enclosed buildings and for negative internal pressure evaluation in partially enclosed buildings.



ESCARPMENT

2-D RIDGE OR 3-D AXISYMMETRICAL HILL

Topographic Multipliers for Exposure B										
H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2-D Ridge	2-D Escarp.	3-D Axisym. Hill		2-D Escarp.	All Other Cases		2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute $2H$ for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:

   H: Height of hill or escarpment relative to the upwind terrain, in meters.

   L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in meters.

   K_1: Factor to account for shape of topographic feature and maximum speed-up effect.

   K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.

   K_3: Factor to account for reduction in speed-up with height above local terrain.

   x: Distance (upwind or downwind) from the crest to the building site, in meters.

   z: Height above local ground level, in meters.

   W: Horizontal attenuation factor.

   γ: Height attenuation factor

   Equation:

   $K_{zt} = (1 + K_1\ K_2\ K_3)^2$; K_1 determined from Table below; $K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$; $K_3 = e^{-\gamma z/L_h}$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	$K_1/(H/L_h)$			γ	μ	
	Exposure A	Exposure B	Exposure C		Upwind of crest	Downwind of Crest
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. Hill	0.95	1.05	1.15	4	1.5	1.5

Figure 6.2.4 Topographic factor, K_{zt} - Method 2

Enclosed, Partially Enclosed, and Open Buildings: Walls & Roofs

Enclosure Classification	GC_{pi}
Open Building	0.00
Partially Enclosed Building	+0.55 -0.55
Enclosed Building	+0.18 -0.18

Notes:

1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of GC_{pi} shall be used with q_z or q_h as specified in Sec 2.4.11.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:

   (i) a positive value of GC_{pi} applied to all internal surfaces

   (ii) a negative value of GC_{pi} applied to all internal surfaces.

Figure 6.2.5 Internal pressure coefficient, GC_{pi} main wind force resisting system component and cladding - Method 2 (All Heights)

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction		Windward								Leeward		
		Angle, θ (degrees)								Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal To ridge for θ ≥10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horizontal distance from Windward edge	C_p
Normal To ridge for θ <10° and Parallel to ridge for all θ	≤0.5	0 to $h/2$	-0.9, -0.18
		$h/2$ to h	-0.9, -0.18
		h to $2h$	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to $h/2$	-1.3**, -0.18
		> $h/2$	-0.7, -0.18

* Value is provided for interpolation purposes

** Value can be reduced linearly with area over which it is applicable as follows

Area (m²)	Reduction Factor
≤ 9.3	1.0
23.2	0.9
≥ 92.9	0.8

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of L/B, h/L and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Sec 2.4.8.
6. Refer to Figure 6.2.7 for domes and Figure 6.2.8 for arched roofs.
7. Notation:
   - B: Horizontal dimension of building, in meter, measured normal to wind direction.
   - L: Horizontal dimension of building, in meter, measured parallel to wind direction.
   - h: Mean roof height in meters, except that eave height shall be used for e 10 degrees.
   - z: Height above ground, in meters.
   - G: Gust effect factor.
   - q_z, q_h: Velocity pressure, in N/m², evaluated at respective height.
   - θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80°, use C_p = 0.8

Figure 6.2.6 (Contd.) External pressure coefficients, C_p main wind force resisting system - Method 2 (All Heights)

Enclosed, Partially Enclosed Buildings: Low-rise Walls & Roofs

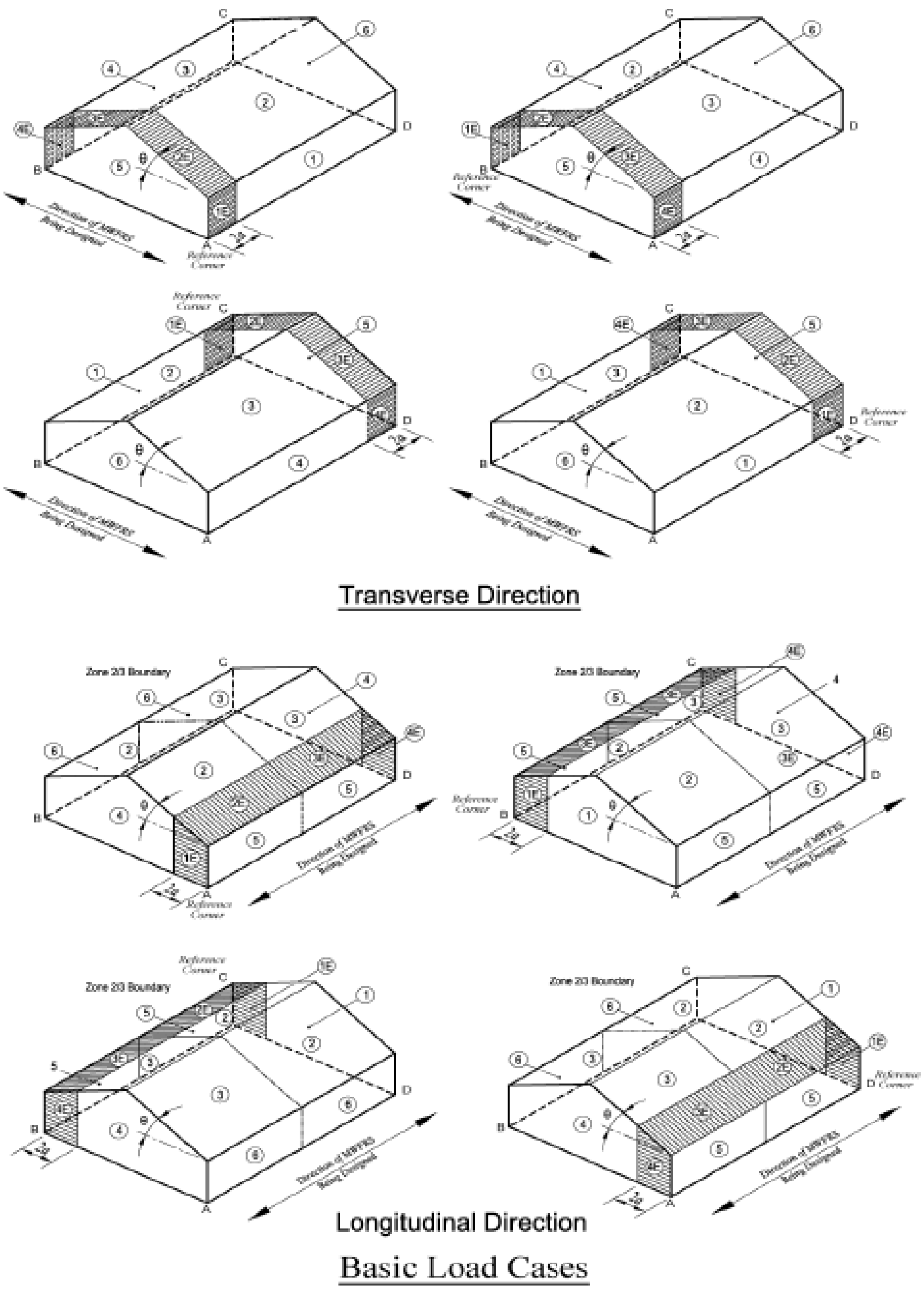


Figure 6.2.10 External pressure coefficients, GC_{pf} for main wind force resisting system- Method 2 ($h \leq 18.3$ m)

Table 6.1.1: Occupancy Category of Buildings and other Structures for Flood, Surge, Wind and Earthquake Loads.

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I, III and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with day care facilities with a capacity greater than 150 • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Healthcare facilities with a capacity of 50 or more resident patients, but not having surgery or emergency Treatment facilities • Jails and detention facilities Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to: • Power generating stations[a] • Water treatment facilities • Sewage treatment facilities • Telecommunication centers Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.	III

Figure 6.2.24 Seismic zoning map of Bangladesh

Table 6.2.14: Description of Seismic Zones

Seismic Zone	Location	Seismic Intensity	Seismic Zone Coefficient, Z
1	Southwestern part including Barisal, Khulna, Jessore, Rajshahi	Low	0.12
2	Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans	Moderate	0.20
3	Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur	Severe	0.28
4	Northeastern part including Sylhet, Mymensingh, Kurigram	Very Severe	0.36

Table 6.2.15: Seismic Zone Coefficient Z for Some Important Towns of Bangladesh

Town	Z	Town	Z	Town	Z	Town	Z
Bagerhat	0.12	Gaibandha	0.28	Magura	0.12	Patuakhali	0.12
Bandarban	0.28	Gazipur	0.20	Manikganj	0.20	Pirojpur	0.12
Barguna	0.12	Gopalganj	0.12	Maulvibazar	0.36	Rajbari	0.20
Barisal	0.12	Habiganj	0.36	Meherpur	0.12	Rajshahi	0.12
Bhola	0.12	Jaipurhat	0.20	Mongla	0.12	Rangamati	0.28
Bogra	0.28	Jamalpur	0.36	Munshiganj	0.20	Rangpur	0.28
Brahmanbaria	0.28	Jessore	0.12	Mymensingh	0.36	Satkhira	0.12
Chandpur	0.20	Jhalokati	0.12	Narail	0.12	Shariatpur	0.20
Chapainababganj	0.12	Jhenaidah	0.12	Narayanganj	0.20	Sherpur	0.36
Chittagong	0.28	Khagrachari	0.28	Narsingdi	0.28	Sirajganj	0.28
Chuadanga	0.12	Khulna	0.12	Natore	0.20	Srimangal	0.36
Comilla	0.20	Kishoreganj	0.36	Naogaon	0.20	Sunamganj	0.36
Cox's Bazar	0.28	Kurigram	0.36	Netrakona	0.36	Sylhet	0.36
Dhaka	0.20	Kushtia	0.20	Nilphamari	0.12	Tangail	0.28
Dinajpur	0.20	Lakshmipur	0.20	Noakhali	0.20	Thakurgaon	0.20
Faridpur	0.20	Lalmanirhat	0.28	Pabna	0.20		
Feni	0.20	Madaripur	0.20	Panchagarh	0.20		

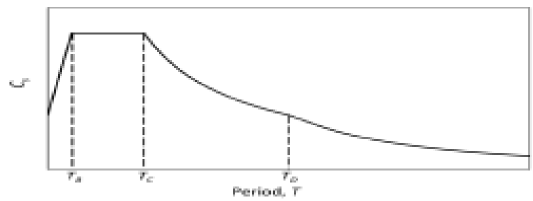


Figure 6.2.25 Typical shape of the elastic response spectrum coefficient Cs

Table 6.2.16: Site Dependent Soil Factor and Other Parameters Defining Elastic Response Spectrum

Soil type	S	T_B(s)	T_C (s)	T_D (s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

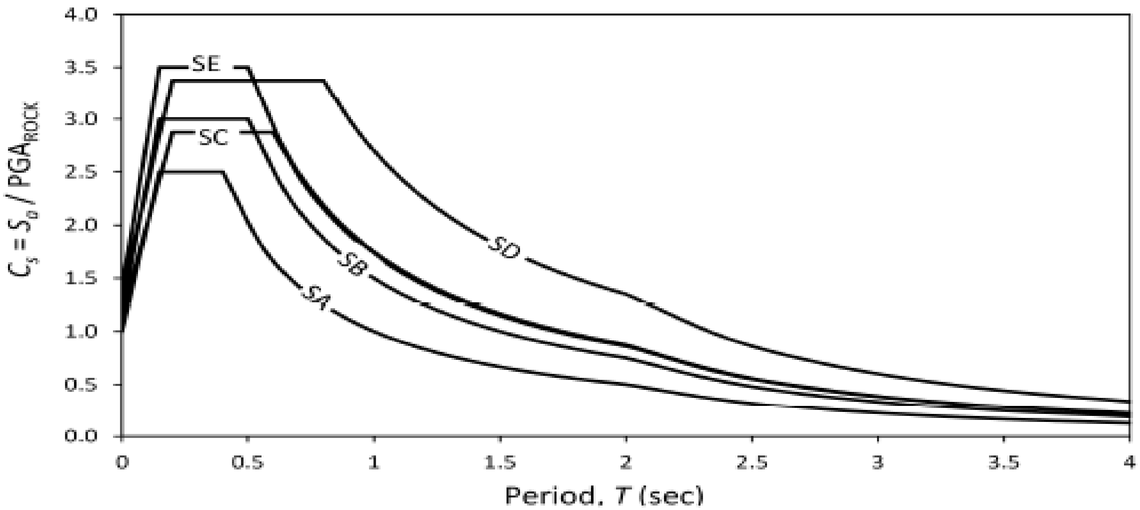


Figure 6.2.26 Normalized design acceleration response spectrum for different site classes.

2.5.5 Building Categories

2.5.5.1 Importance factor

Buildings are classified in four occupancy categories in Chapter 1 (Table 6.1.1), depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse. Depending on occupancy category, buildings may be designed for higher seismic forces using importance factor greater than one. Table 6.2.17 defines different occupancy categories and corresponding importance factor.

Table 6.2.17: Importance Factors for Buildings and Structures for Earthquake design

Occupancy Category	Importance factor I
I, II	1.00
III	1.25
IV	1.50

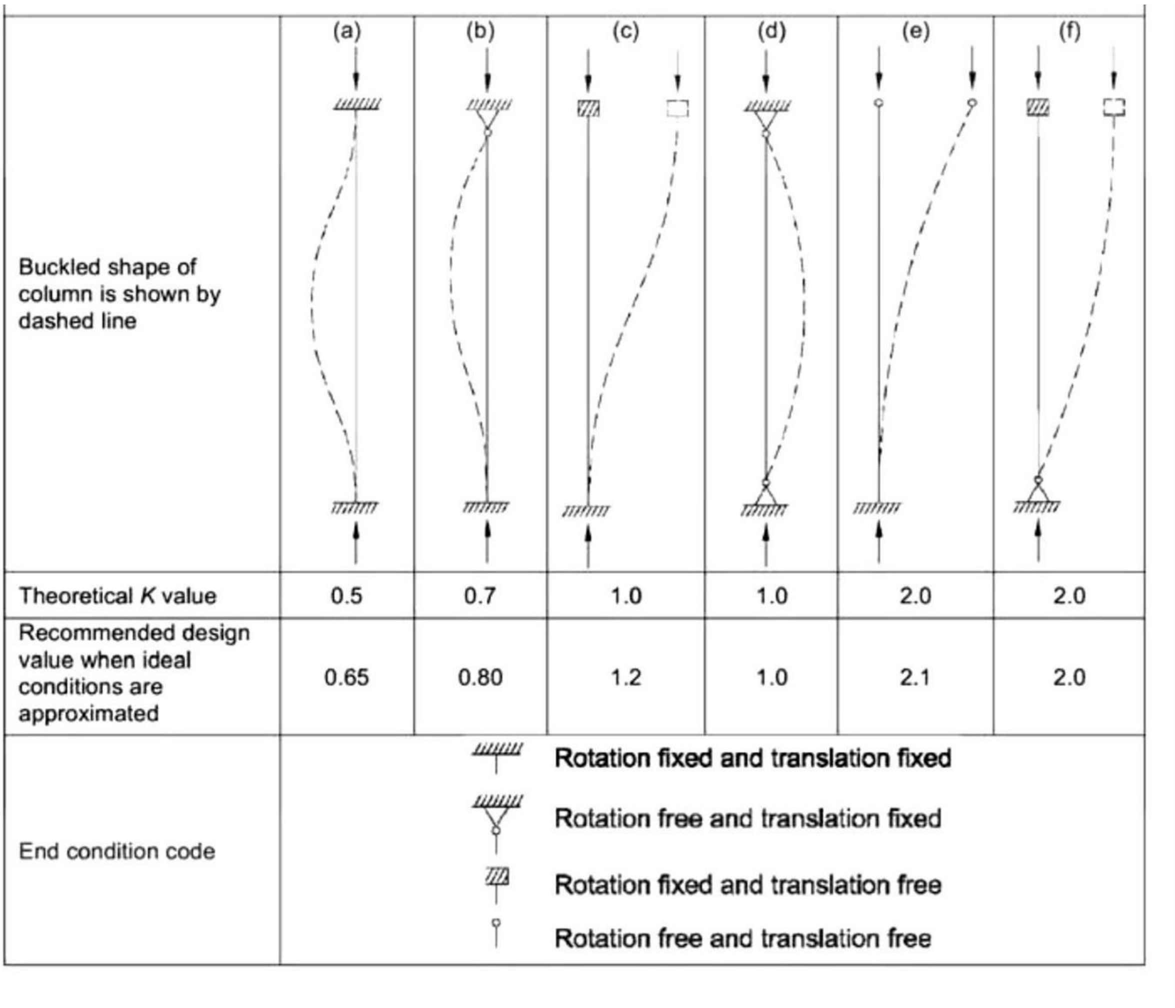
Minimum Size of Fillet Weld
AISC, Table-J2.4

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



	(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

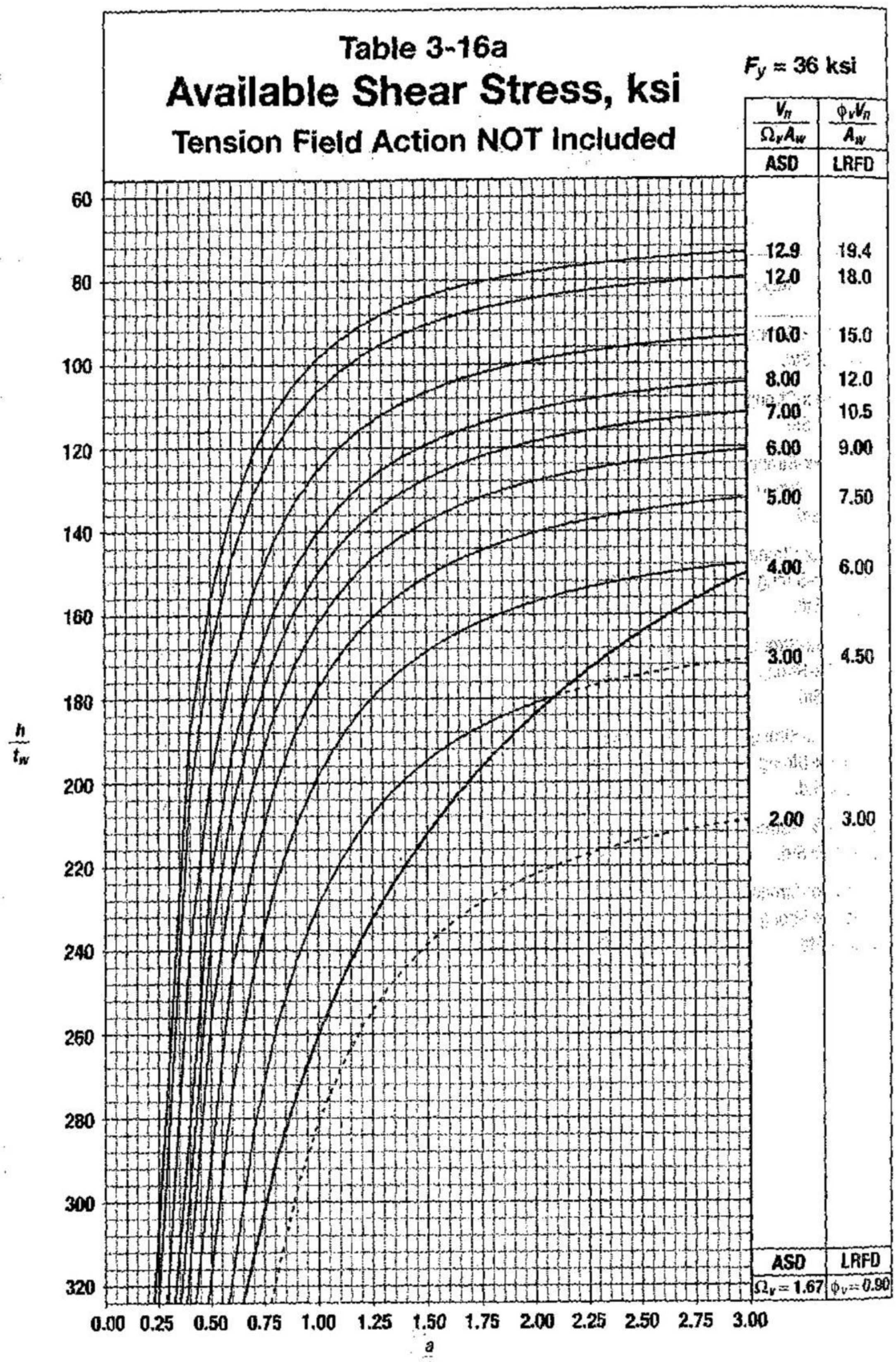


Table 3-16b
Available Shear Stress, ksi
Tension Field Action Included

$F_y = 36$ ksi

$\frac{V_n}{\Omega_v A_w}$	$\frac{\phi_v V_n}{A_w}$
ASD	LRFD

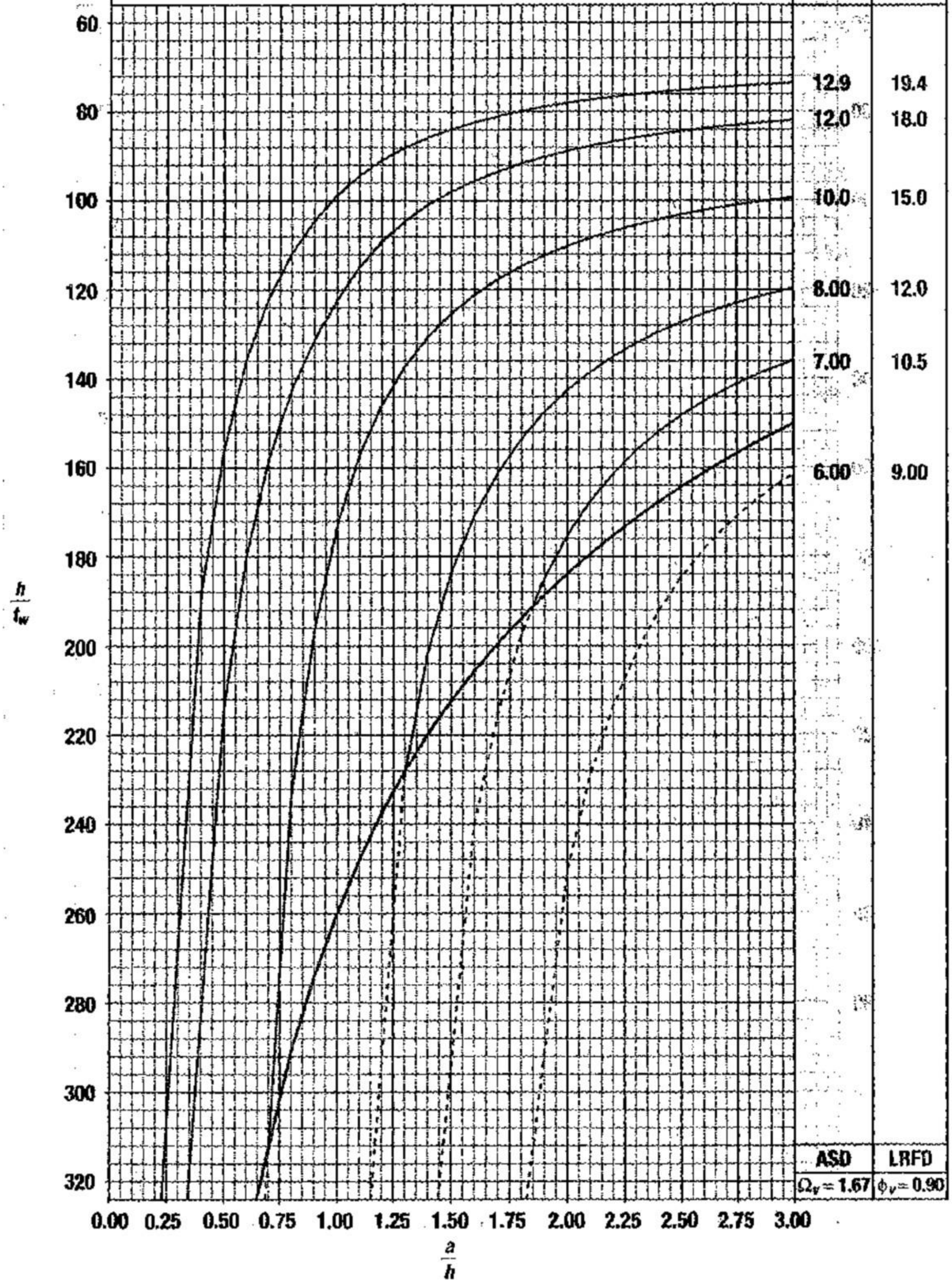


Table 3-17a
Available Shear Stress, ksi
Tension Field Action NOT Included

$F_y = 50$ ksi