INDEX

Heaven's Light Is Our Guide

RAJSHAHI UNIVERSITY OF ENGINEERING & TECHNOLOGY

Department Of Civil Engineering

Expt No. 01

Name of Expt. Introduction of Bridge

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COURSE NO. : CE 3112	**GROUP :**
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Chapter No: 1

Date: 26-12-2020

Chapter Name: Introduction of Bridge.

Different types of bridges and its suitability:

Definition: A bridge is a structure providing passage over an obstacle without closing the way beneath. The required passage may be for a road, a railway, pedestrains, a canal or a pipeline. The obstacle to be crossed may be a river, a road, railway or a valley.
In other words, bridge is a structure for carrying the road traffic or other moving loads over a depression or obstruction such as channel, road or railway.
A bridge is an arrangement made to cross an obstacle in the form of a low ground or a stream or a river without closing the way beneath.

Components of bridge:

The bridge structure comprises of the following parts

1) Superstructure or Decking:

This includes slab, girder, truss, etc. This bears the load passing over it and transmits the forces caused by the same to the substructures.

2) Bearings :
The bearings transmit the load received from the decking on to the substructure and are provided for distribution of the load evenly over the substructure material which may not have sufficient bearing strength to bear the superstructure load directly.

3) Substructure:
This comprises piers and abutments, wing walls or returns and their foundation.

i) Piers and Abutments
These are vertical structures supporting deck / bearing provided for transmitting the load down to the bed/ earth through foundation.

ii) Wing walls and Returns
These are provided as extension of the abutments to retain the earth of approach bank which otherwise has a natural angle of repose.

iii) Foundation
This is provided to transmit the load from the piers or abutments and wings or returns to and evenly distribute the load on to the strata. This is to be

provided sufficiently deep so that it is not affected by the scour caused by the flow in the river and does not get undermined.

Classification:

Bridges may be classified in many ways, as below.

⇒ According to the flexibility of superstructure as fixed span bridges or movable bridges.

i) Fixed span superstructure:

In case of fixed span superstructure, the superstructure remains in a fixed position and most of the bridges are of this category.

ii) Movable span bridges:

The superstructure is lifted or moved with the help of some suitable arrangement.

⇒ According to the position of bridge floor relative to the formation level and the highest flood discharge as deck bridges, through bridges or semi-through bridges.

i) Deck bridges:

Deck type ~~bt~~ bridges refer to those in which the road deck is carried on the top flange or on top of the supporting girders

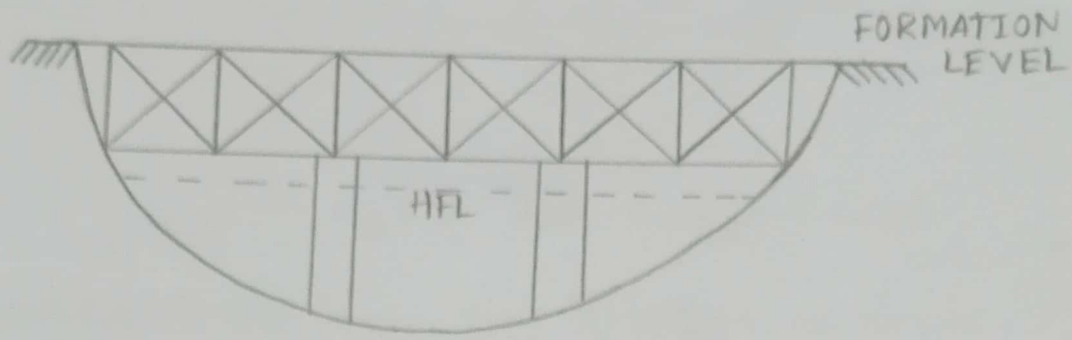

Fig 1.1: Deck Bridge

ii) Through Bridge:

The decking is supported by the bottom flange of the main supporting girders provided on either side.

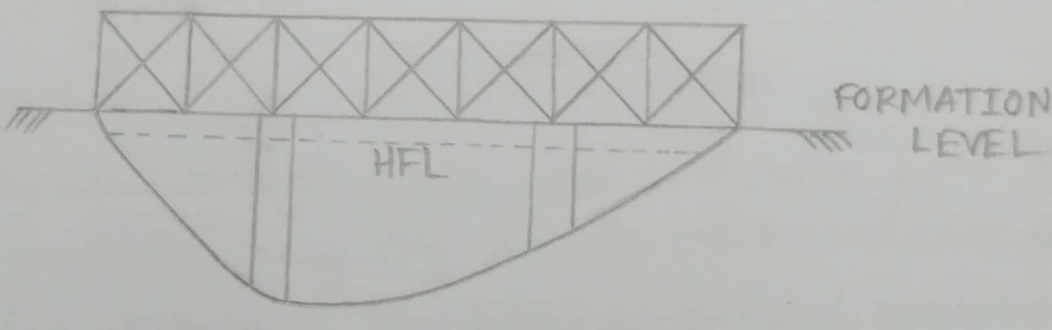

Fig 1.2: Through Bridge

iii) Semi through bridges:

The semi-through bridge has its deck midway and the deck load is transmitted to the girder through the web of the girder. In this also, the main girders are on either side of deck.

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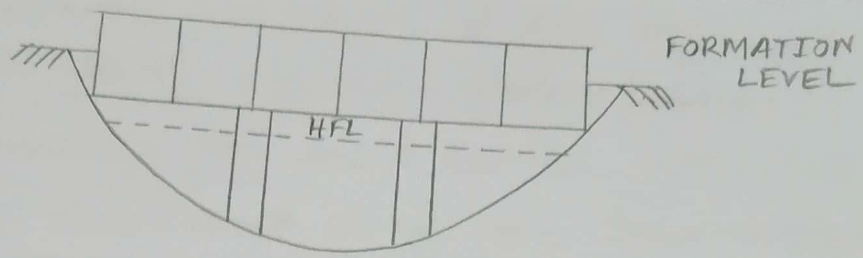

Fig 1.3: Semi-through Bridge

⇒ According to the inter-span relations as simple, continuous or cantilever bridges.

1) Simply supported:

Generally width of bridge is divided into number of individual spans. For each span, the load carrying member is simply supported at both ends.

2) Continuous:

In continuous bridges spans are continuous over two or more supports. They are statically indeterminate structures. In continuous bridges the bending moment anywhere in the span is considerably less than that in case of simply supported span. Such reduction of bending moment ultimately results in the economic section for the bridge. In continuous bridges the stresses are reduced due to negative moments developed at pier of supports. Thus continuous span bridges have considerable saving compared to simply supported bridge construction.

Following are the advantages of RCC continuous girder bridges over simply supported girder bridges.

i) As the bearings are placed on the centerline of piers, the reactions at piers are transmitted centrally.

ii) It is found that the continuous girder bridges suffers less vibration and deflection

iii) The continuous girder bridge requires only one bearing at each pier as against two bearing for simply supported girder bridge.

iv) The depth of decking at mid span is reduced and it may prove to be useful for over bridges where headroom is of prime consideration.

v) The expansion joints required will be less.

vi) There is reduction in cost as less quantity of concrete and steel are required.

Following are the disadvantages of RCC continuous girder bridges over simply supported girder bridges.

i) The design is more complicated as it is a statically indeterminate structure.

ii) The detailing and placing of reinforcements are to be carried out with extreme care.

iii) The placing of concrete and removal of formwork are to be executed carefully in proper sequence.

3) Cantilever:

A cantilever bridge is formed of cantilevers projecting from supporting piers. The ends of a cantilever bridge are treated as fixed. A cantilever bridge combines the advantages of a simply supported span and a continuous span. For long spans and deep valleys and at places where it will not be practicable to use centering, cantilever bridges are more suitable. They are suitable in case of uneven settlement of foundation. The construction of a cantilever bridge may either be of simple type or of balanced type.

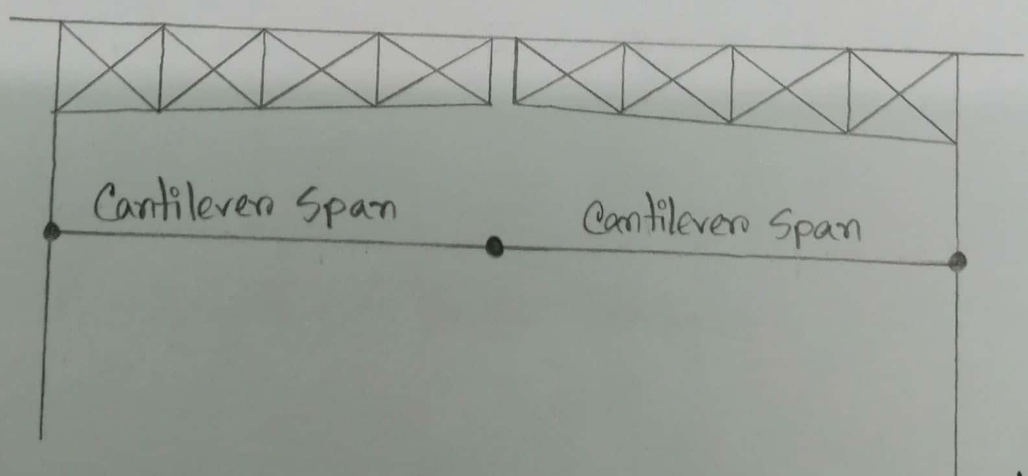

Fig.1.4: Cantilever Bridge with simple construction

In case of cantilever bridge with balanced type of construction, hinges are provided at the points of contraflexure of a continuous span and an intermediate simply supported span is suspended between two hinges.

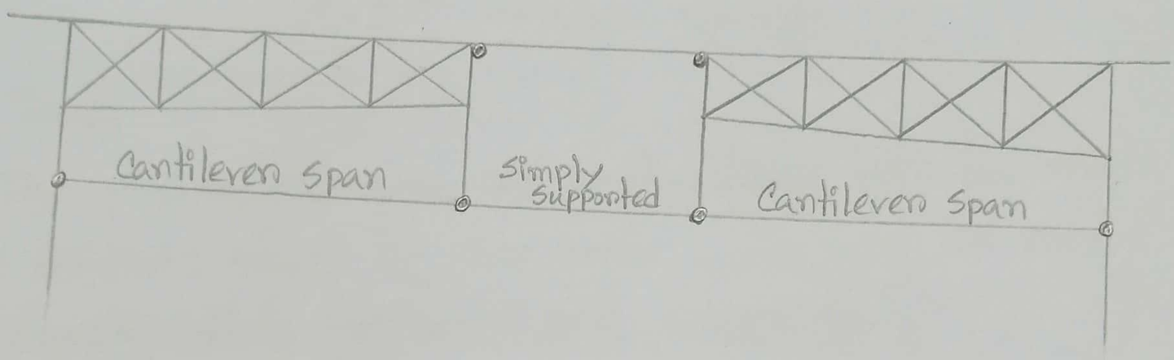

Fig 1.5: Cantilever Bridge with simple construction

⇒ According to the form or type of superstructure as arch, beam, truss, slab, rigid frame or suspension bridges.

1) Slab

2) Beam

3)

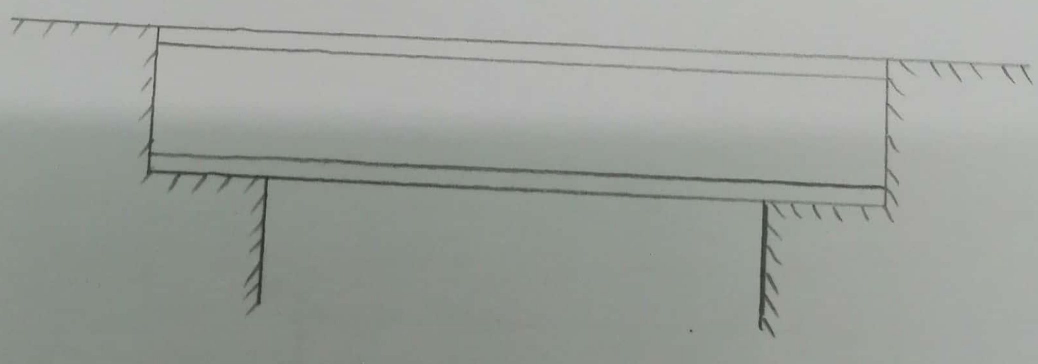

Fig. 1.6: Beam Bridge

3) Girder

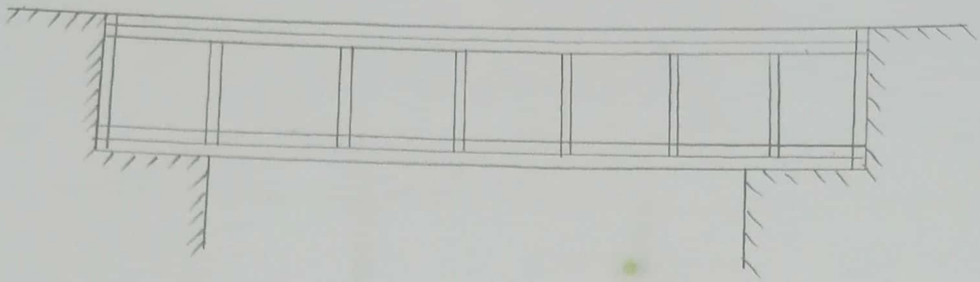

Fig 1.7: Girder Bridge

4) Truss

The girder/beam as well as the truss can be made up of timber, steel or concrete, or can be made up of combination of steel and concrete.

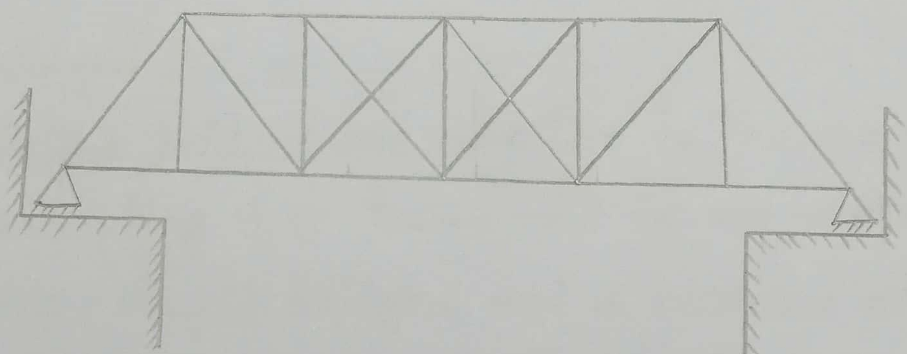

Fig 1.8: Truss Girder Bridge

5) Arch

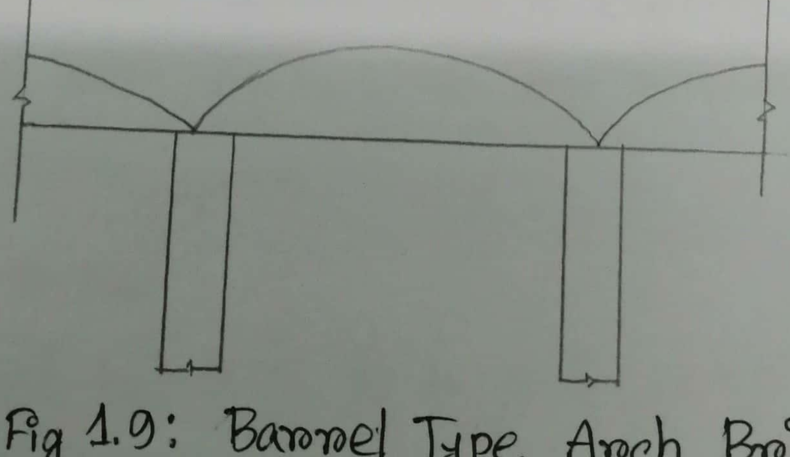

Fig 1.9: Barrel Type Arch Bridge

6) Suspension

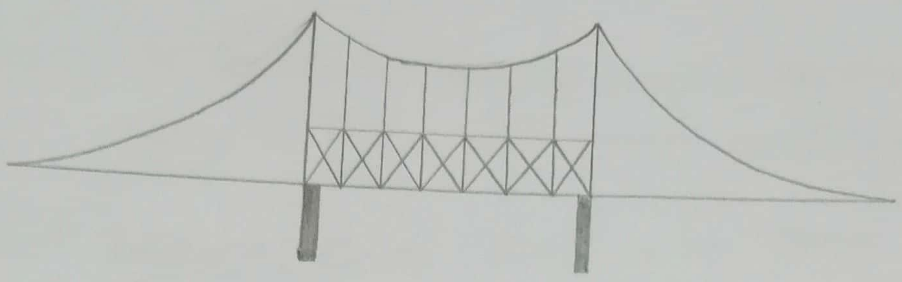

Fig 1.10: Suspension Bridge

suspension bridges are made up of high tensile steel cables strung in form of catenaries to which the deck is attached by steel suspenders, which are mainly made up of steel rods / members / cables.

7) Cable stayed

Cable-stayed bridges are similar to the suspension bridges excepting that there will be no suspenders in the cable-stayed bridges and a number of these can be of masonry, concrete or steel.

Various economical span ranges for these types generally adopted are:

Arch: For small spans of 3 to 15m in masonry, steel arch up to 519m and concrete arches up to 305m spans.

Slabs: Up to 9m

Girders and beams: 10 to 16m (exception up to 250m in continuous construction)

Trusses : 30 to 375m simply supported and up to 550m with cantilevered combination.

Suspension Bridges: Over 500m up to 1400m

Cable stayed: 300 to 600m

⇒ According to the materials of construction used for superstructure as cement concrete, prestressed concrete, steel, masonry, iron, timber or composite bridges.

The masonry bridges are used for short spans and according to availability of material and skilled labour. Timber bridges are used for ~~spans~~ short spans, light loads and for use as temporary and unimportant bridges. With the invention and development of concrete, bridges are being built entirely with concrete, either reinforced or prestressed or a combination of both for superstructure. The common examples of composite construction are:

i) Concrete beams reinforced with steel bars

ii) Precast prestressed concrete girders with cast-

-in-situ RCC slab.

iii) Rolled steel joists topped by a cast-in-situ RCC slab.

Following are the ~~dis~~advantages of composite bridges

i) It leads to reduction in deflection and vibrations

ii) It leads to speed in construction

iii) It proves to be economical

iv) It results in better quality control

v) The cost of formwork is reduced

vi) The cost of foundations for abutments is reduced.

vii) The cost of transportation is minimized

viii) The overall depth of beam for a composite construction is reduced and it leads to savings in lengths of approaches.

Table shows the maximum spans up to which a particular type of bridge can be recommended.

Sr. No.	Type	Maximum span
1.	RCC arch bridge	200m
2	RCC bow-string girder bridge	45m
3	RCC cantilever bridges with balanced type	30m
4.	RCC continuous bridge	45m
5	RCC deck type girder bridge	20m
6	RCC filled spandrel fixed arch bridge	35m
7	RCC open spandrel rib type bridge	50m
8	RCC portal frame bridge	15m
9	Prestressed concrete continuous bridge	110m
10	Prestressed concrete arch bridge	150m
11	Prestressed concrete girder bridge simply supported	55m
12	Steel arch bridge	500m
13	Steel bow-string girder bridge	240 m
14	Steel cable suspension bridge	1200 m
15	Steel plate girder	30m
16	Steel rolled beam bridge	10m
17	Steel truss bridge	180 m

⇒ According to the method of clearance for navigation as bascule, lift, swing or transporter bridges.

i) Movable - bascule bridges :
The main girders are lifted together with deck about the hinge provided on one end of the span. It may be single or double depending on span.

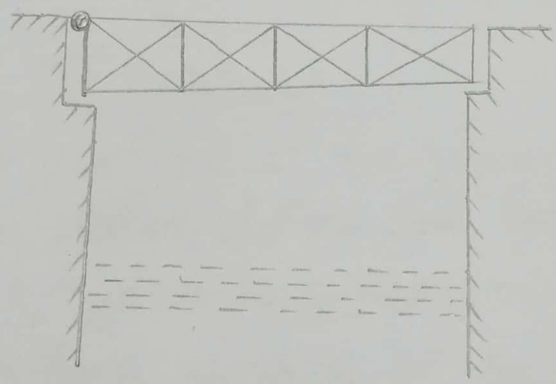

Fig 1.11 : Bascule Bridge

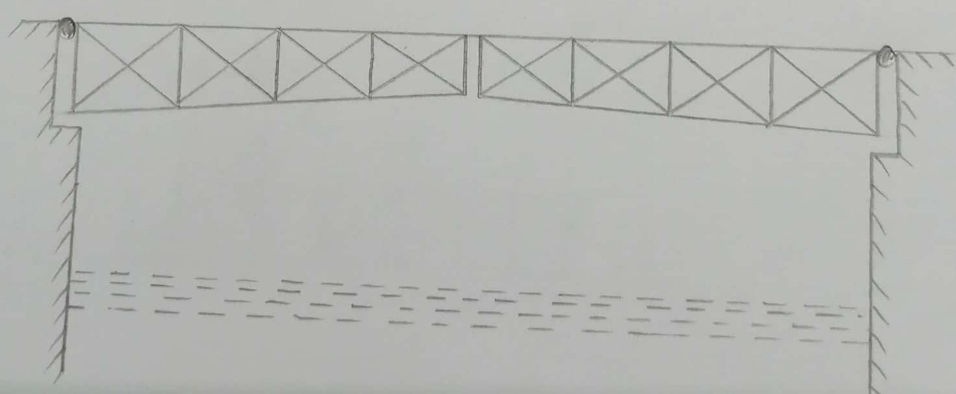

Fig 1.12 : Double Buscule Bridge.

ii) Movable – swing bridges:
The girders and deck can be swung about its middle over the middle pier, clearing the span on either side for passage ship.

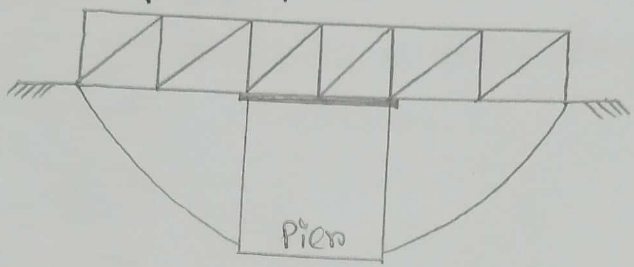

Fig 1.13: Swing Bridge

iii) Movable – lift Bridges:
Gantries are provided at the piers at either end of the span and the entire girder and the floor system is lifted up by a hydraulic arrangement to the extent required for free passage of the ship.

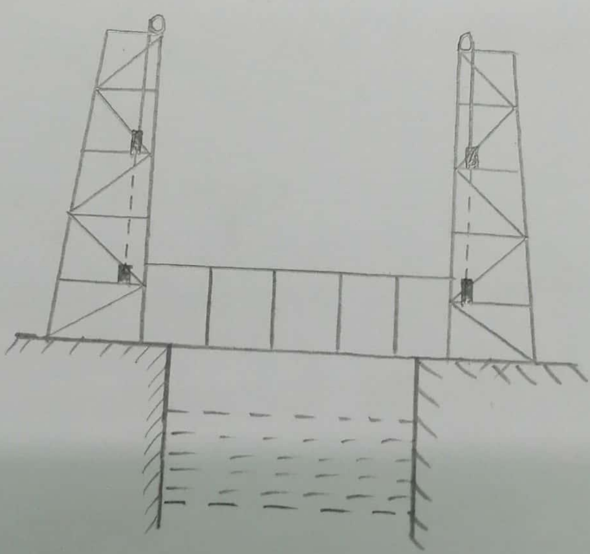

Fig 1.14: Lift Bridge

iv) Transporter:

In case of transporter bridges, a moving cage is suspended from an overhead truss with the help of cable or wire ropes. The overhead truss rests on two towers and it contains rails for cage to roll.

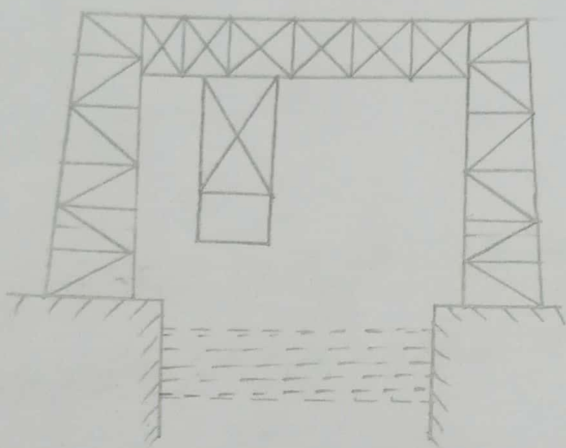

Fig 1.15 : Transporter Bridge

⇒ According to the expected utility period of service as temporary, military or permanent bridges.

i) Temporary bridges:

The temporary bridges are defined as structures, which are constructed to cross a river or a stream in place of permanent works. The useful life of a temporary bridge is estimated as about 10 years.

ii) Military (Ponton, bailey):

Ponton bridges are constructed on roads on which traffic is minor and seasonal and the river itself is subjected to floods during only short periods, not exceeding three months of the year when the traffic on the load can safely be suspended floating type bridges, pontoons are provided.

Bailey refers to bridge superstructures normally made up of assemblage units, which can be carried in units, assembled and launched, in a short duration over a gap.

iii) Permanent bridges:

The permanent bridges are defined as structures, which are constructed to cross a river or a stream permanently.

⇒ According to the function as road, railway, road-cum-railway or pipeline bridges.

⇒ According to the method of connections adopted in ~~schoo~~ steel bridges as riveted, welded or pin-connected bridges.

Majority of steel bridges are of riveted type, welded type is recently used.

i) Pin-connected

ii) Riveted

Riveted connections are proved to be more rigid and responsible for secondary stresses.

iii) Welded.

⇒ According to the length of span as culvert, minor bridges, major bridges or long span bridges.

i) Culverts (less than 6m)

Culverts is a bridge having a ~~span~~ gross length of six meters or less between the faces of abutments or extreme vent way boundaries and measured at right angles thereto.

ii) Minor bridge (6 to 30m)

iii) Major bridge (above 30m)

iv) Long span bridge (above 120m)

⇒ According to the degree of redundancy as determinate or indeterminate bridges.

i) Determinate

ii) Indeterminate

⇒ According to the level of crossing of highways and railways as over bridges or under bridges

i) Over bridge

ii) Under bridge

Selection of type of bridge:

In the selection of the proper type of concrete bridge for any particular case, cost is usually the determining factor. Occasionally, however, the problem is complicated by special requirements, such as appearance, restricted headroom, difficult foundations, limited time of construction, or ~~different~~ difficulties in formwork caused either by the required height of supports or by the fact that is necessary to maintain traffic under the bridge during construction.

For bridges having one span, the following types of structures may be used:

i) Simply supported deck or through girders.
ii) Right angle rigid frames
iii) Right angle frames with concealed cantilivers with or without counterweights.
iv) Simply supported girders with concealed cantilevers, with or without counterweights.
v) Two short concealed spans, one at each side of the opening, each provided with a cantilever extending into opening and supporting a short center span.

For a bridge with several spans, the following arrangements should be considered:

i) A number of simply supported girder spans.
ii) A combination of girders provided with cantilevers and short spans supported by these cantilevers.
iii) Continuous girders supported by independent piers

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RAJSHAHI UNIVERSITY OF ENGINEERING & TECHNOLOGY

Department Of Civil Engineering

Expt No. 02

Name of Expt. Design of a Plate Girder Bridge (Stringer Design).

SUBJECT: Structural Analysis & Design Sessional-I	SUBMITED BY :
COURSE NO. : CE 3112	NAME: Most. Afrin Sultana
DATE OF EXPT. : 02-01-2021	GROUP :
DATE OF SUB. : 06-03-2021	ROLL NO: 1700082
	SESSION : 2017-18

Chapter No: 02 Date: 02-01-2021

Chapter Name: Design of a Plate Girder Bridge.
(Stringer Design)

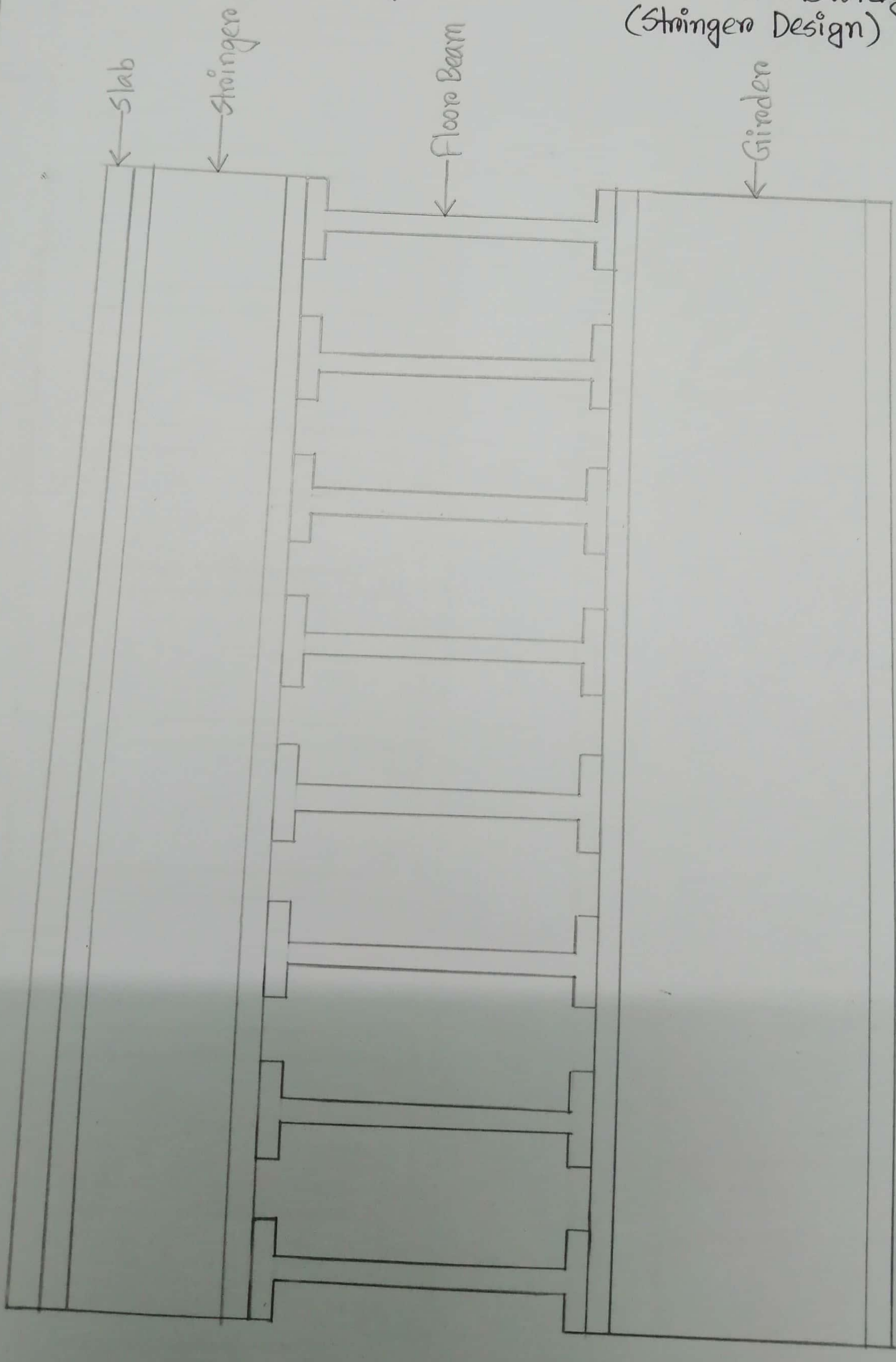

Fig- 2.1 : Elevation

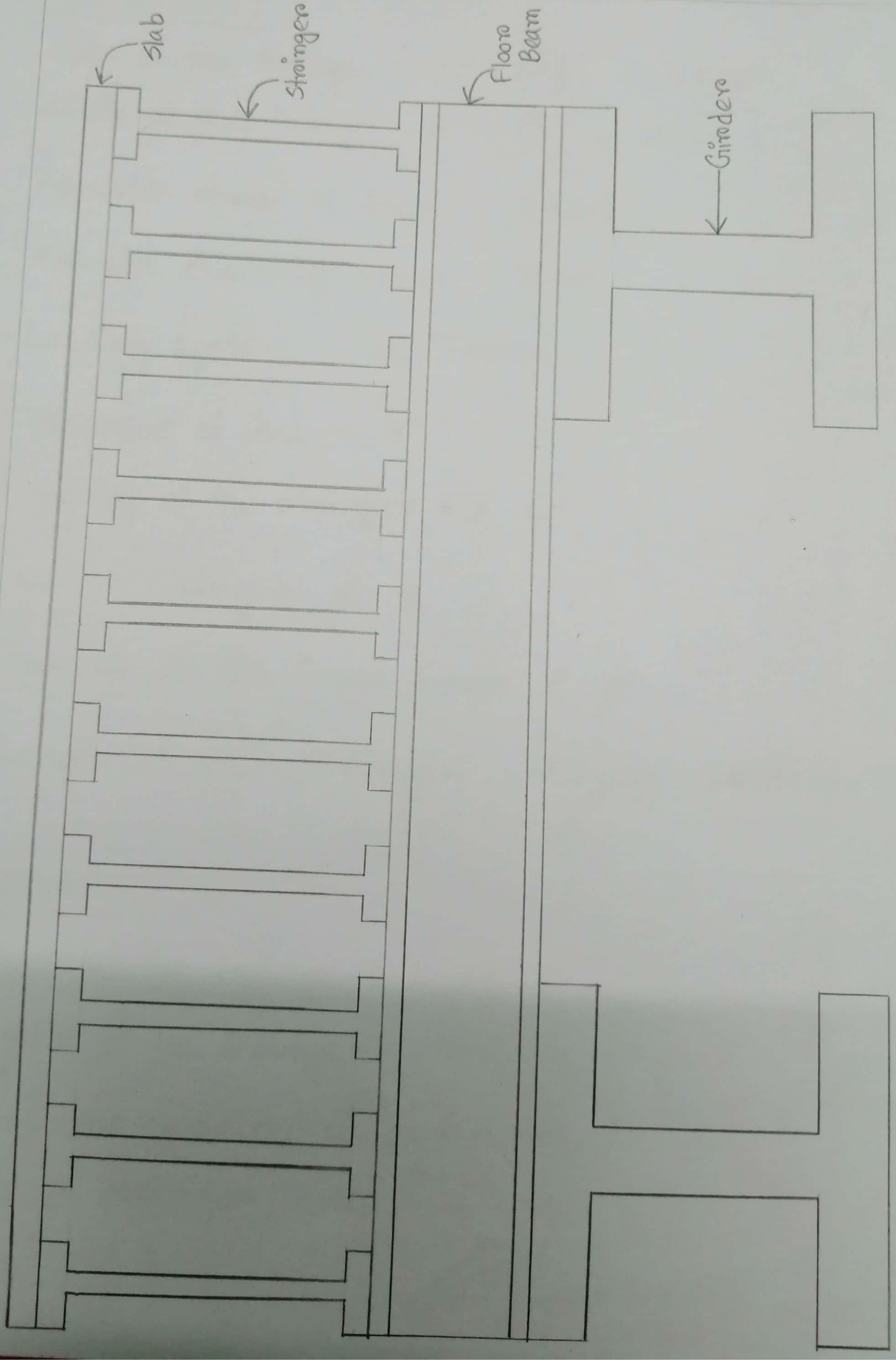

Fig-2.2 : Cross-section

Specification :

Span of the Bridge = $50' + \left(\frac{82}{3}\right)' = 77.33'$

Width of the Bridge = $20' + \left(\frac{82}{4}\right)' = 40.5'$

Allowable stress of Steel = 24 ksi

Allowable shearing stress = 10 ksi

Loading system = H_{20} Truck

Thickness of Slab = 8"

Spacing of the Stringers $\ngtr$ 6'

No. of Stringers $\nless$ 5

Spacing of the Floor Beam $\nless$ 10'

Length of each Stringer = Laying over 3 Floor Beam.

Design :

Span of the Bridge, L = 77.33'

Width of the Bridge, W = 40.5'

Assume spacing of stringer, $S_s = 5'$

No. of stringer, $n_s = \frac{W}{S_s} + 1$

$= \frac{40.5}{5} + 1 = 9.1 \approx 10$

Actual spacing of stringer, $S_{acs} = \frac{W}{n_s - 1}$

$$= \frac{40.5}{10-1}$$

$$= 4.5'$$

Assume spacing of floor beam, $S_f = 12'$

No. of floor beam, $n_f = \frac{L}{S_f} + 1$

$$= \frac{77.33}{12} + 1$$

$$= 7.44 \approx 8$$

Actual spacing of floor beam, $S_{acf} = \frac{L}{n_s - 1}$

$$= \frac{77.33}{8-1}$$

$$= 11.05'$$

Design of slab (8"):

It is kept intentionally blank.

Design of Stringers:

a) Dead Load Calculation:

(1) Load from slab $= \frac{\text{Slab thickness}}{12} \times 0.15 \times S_{acs}$

$= \frac{8}{12} \times 0.15 \times 4.5$

$= 0.45$ klf

(2) Assume self weight of stringer $= 0.032$ klf

Dead Load, $\omega_D = (1) + (2)$

$= 0.45 + 0.032$

$= 0.482$ klf

Each stringer continues through 3 floor beams

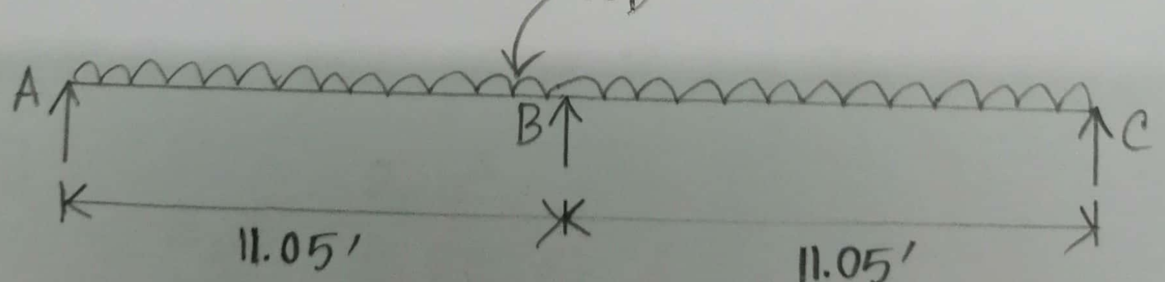

Fig – 2.3: Dead Load calculation

b) Reactions Calculation :

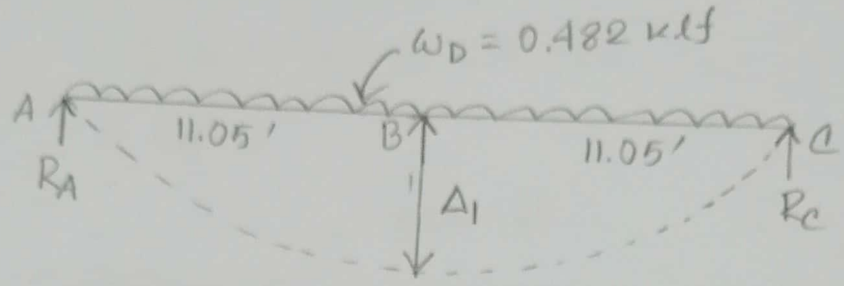

Fig - 2.4: Reaction calculation

Deflection at mid point B for the uniformly distributed load,

$$\Delta_1 = \frac{5 \omega_D L^4}{384 EI} \qquad [L = 2 \times S_{acf}]$$

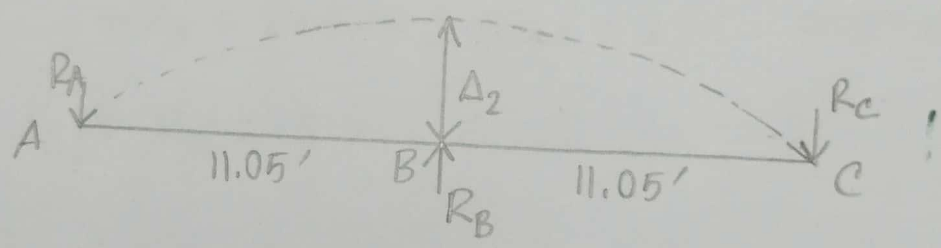

Fig - 2.5: Reaction calculation

Deflection at mid point B for the concentrated load,

$$\Delta_2 = \frac{R_B L^3}{48 EI}$$

A support exists at point B,

$$\Delta_1 = \Delta_2$$

$$\Rightarrow \frac{5 \omega_D L^4}{384 EI} = \frac{R_B L^3}{48 EI}$$

$$\Rightarrow R_B = \frac{5\omega_D L}{8}$$

$$= \frac{5}{8} \times 0.482 \times 2 \times 11.05$$

$$= 6.66 \text{ K}$$

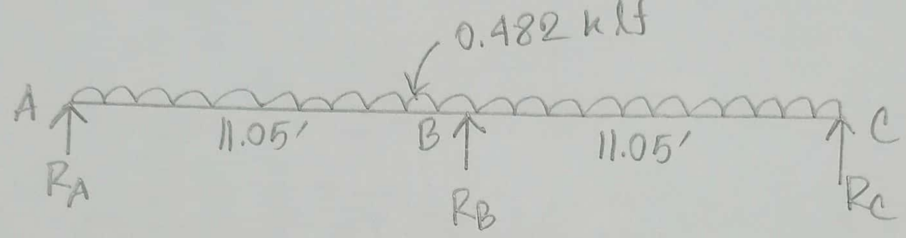

Fig - 2.6: Reaction calculation

$$\Sigma M_c = 0 \ ; \quad R_A \times 2S_{acf} + R_B \times S_{acf} - \omega_D \frac{(2S_{acf})^2}{2} = 0$$

$$\Rightarrow R_A \times 2 \times 11.05 + 6.66 \times 11.05 - 0.482 \times \frac{(2 \times 11.05)^2}{2} = 0$$

$$\Rightarrow R_A = 2 \text{ K}$$

$$\Sigma F_y = 0 \ ; \quad R_A + R_B + R_c = \omega_D \times 2S_{acf}$$

$$\Rightarrow 2 + 6.66 + R_c = 0.482 \times 2 \times 11.05$$

$$\Rightarrow R_c = 2 \text{ K}$$

c) Shear and Moment Calculation:

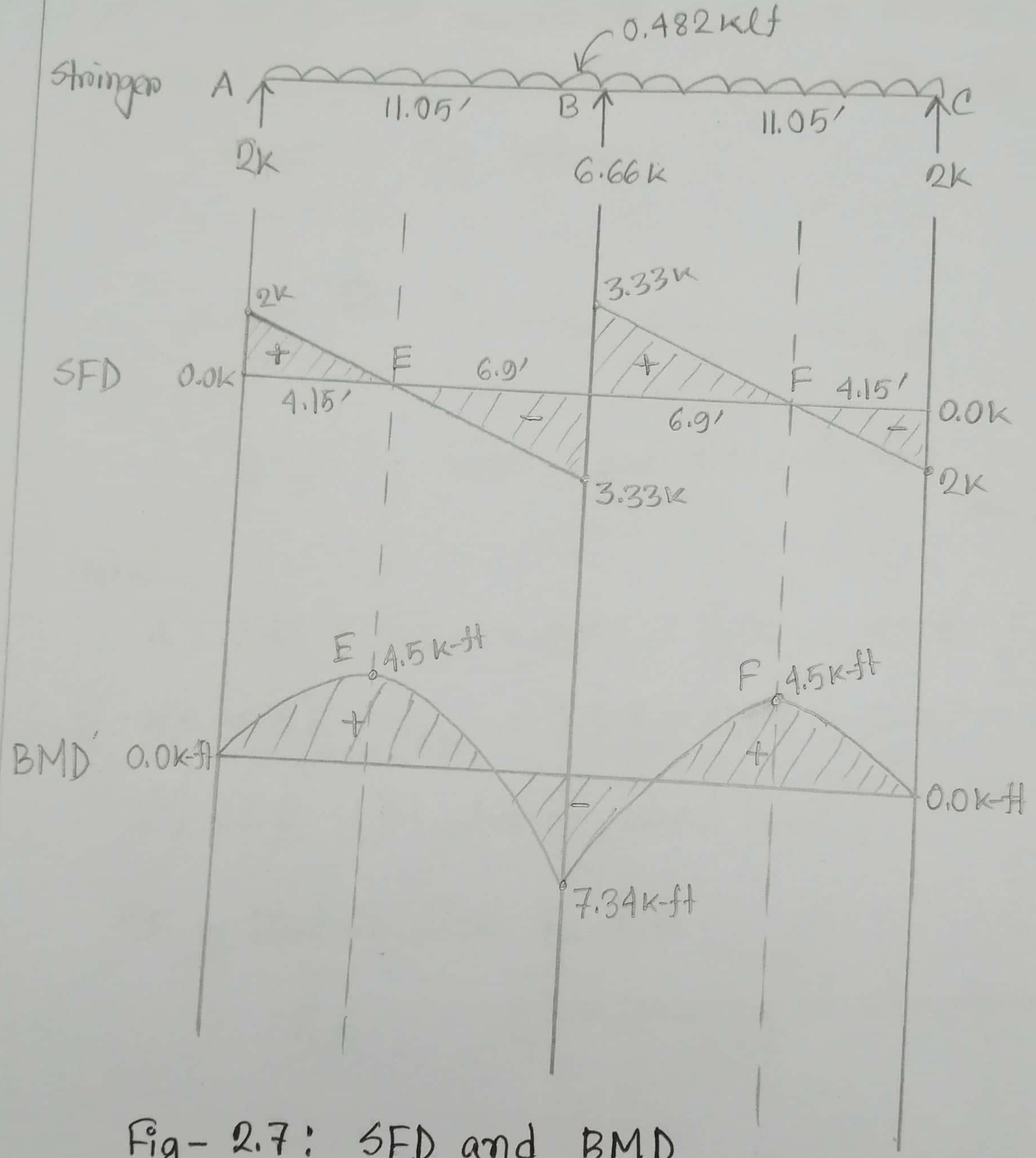

Fig- 2.7: SFD and BMD

Shear at point A, $V_A = 2k$

Shear at point B, $V_B = 3.33\ k$

Maximum (+)ve Moment at point E or F, M_E or $M_F = 4.15$ kft

Maximum (-)ve Moment at point B, $M_B = 7.34$ k-ft

d) Live Load Calculation :

It is found that maximum negative moment for a wheel is produced when the position of that is as shown in fig. below –

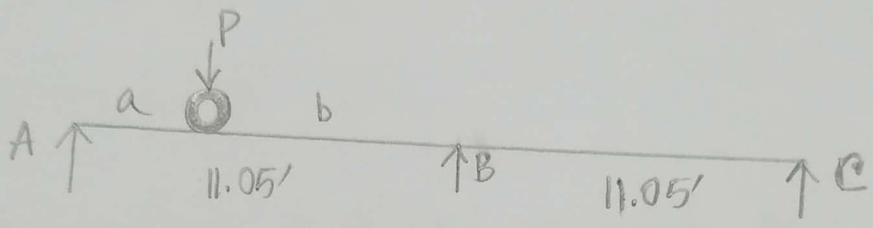

Fig - 2.8: Live Load Calculation

Here,

$a = 0.43 \times S_{acf} = 0.43 \times 11.05 = 4.75'$

$b = 0.57 \times S_{acf} = 0.57 \times 11.05 = 6.3'$

$P = 16\,k$ [Maximum wheel load]

DF = Distribution Factor

$= \frac{S}{5.5}$ [S = Spacing of stringer, $S_{max} = 14'$]

$= \frac{5}{5.5}$

$= 0.91$

Maximum (+)ve live load moment, $M_{(+)L}$

$$= \frac{Pab}{4(S_{acf})^3}\left[4(S_{acf})^2 - a(S_{acf}+a)\right] \times DF$$

$$= \frac{16 \times 4.75 \times 6.3}{4 \times (11.05)^3} \times \left[4 \times (11.05)^2 - 4.75 \times (11.05 + 4.75)\right] \times 0.91$$

$= 33.37$ k-ft

Maximum (-)ve live load moment, $M_{(-)L}$

$= 0.0964 \times P \times S_{acf} \times DF$

$= 0.0964 \times 16 \times 11.05 \times 0.91$

$= 15.51$ k-ft

Maximum shear for live load, $V_L = P \times DF$

$= 16 \times 0.91$

$= 14.56$ k

e) Impact Load Calculation :

Impact coefficient, $IC = \frac{50}{S_{acf} + 125}$ $[IC_{max} = 30\%]$

$$= \frac{50}{11.05 + 125}$$

$= 0.37$

≈ 0.3

Impact (+)ve live load moment, $IM_{(+)L}$

$= IC \times M_{(+)L}$

$= 0.3 \times 33.37$

$= 10.011$ k-ft

Impact (-)ve live load moment, $IM_{(-)L}$

$= IC \times M_{(-)L}$

$= 0.3 \times 15.51$

$= 4.653$ k-ft

Impact live load shear, $IV_L = IC \times V_L$

$= 0.3 \times 14.56$

$= 4.368$ k

Table-2.1: Design Shear

Sign	Dead Load Shear [k]	Live Load Shear [k]	Impact Load Shear [k]	Total Shear [k]	Design Shear [k]
(+)ve	3.33	14.56	4.368	22.258	22.258

Table-2.2: Design Moment

Sign	Live Load Moment [k-ft]	Live Load Moment [k-ft]	Impact Load Moment [k-ft]	Total Moment [k-ft]	Design Moment [k-ft]
(+)ve	4.15	33.37	10.011	47.531	47.531
(-)ve	7.34	15.51	4.653	27.503	

Section Choice :

Design Moment, $M = 47.531$ k-ft

Allowable steel stress, $\sigma = \frac{Mc}{I}$

$$\Rightarrow \sigma = \frac{M}{Z} \quad [\because Z = \frac{I}{c}]$$

$$\Rightarrow Z = \frac{M}{\sigma}$$

$$= \frac{47.531 \times 12}{24}$$

$$= 23.7655 \text{ in}^3$$

$$= 23.7655 \times (25.4)^3 \text{ mm}^3$$

$$= 389.45 \times 10^3 \text{ mm}^3$$

Table - 2.3: Properties of Wide flange section [I section]

Designation	Area (in^2)	Depth (in)	Web thickness (in	Flange Width (in)	Flange Thickness (in)	I (in^4)	$S = \frac{I}{c}$ (in^3)	$r = \sqrt{I/A}$ (in)
W12X26	7.65	12.22	0.230	6.490	0.380	204	33.4	5.17
$\boxed{\text{W12X22}}$	6.48	12.31	0.260	4.030	0.425	156	25.4	4.91
W12X19	5.57	12.16	0.235	4.005	0.350	130	21.3	4.82

Shear Check :

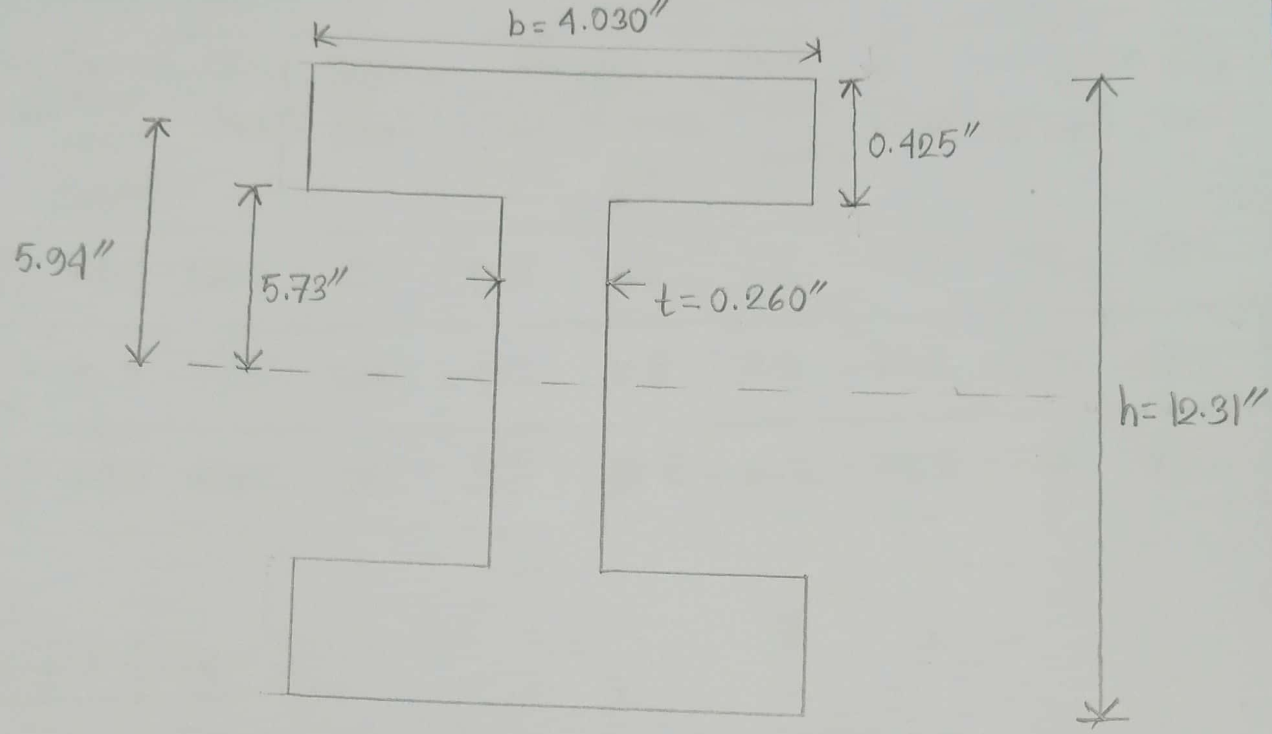

Fig - 2.9 : I - Section

Shearing stress , $\tau = \frac{VQ}{Ib}$

$Q = A' \times \bar{y}$

$= (4.030 \times 0.425 \times 5.94) + (0.260 \times 5.73 \times \frac{5.73}{2})$

$= 14.44 \text{ in}^3$

$\therefore \tau = \frac{22.258 \times 14.44}{156 \times 0.260}$

$= 7.92 \text{ ksi} < 10 \text{ ksi}$

So, the design is okay

Table – 2.4: Properties of Wide flange section (I-section)

Designation	Theoretical mass (kg/m)	Area (mm²)	Depth (mm)	Flange Width (mm)	Flange Thickness (mm)	Web Thickness (mm)	I (10^6 mm⁴)	$S=\frac{I}{C}$ (10^3 mm³)	$r=\sqrt{I/A}$ (mm)
W200 x46	46.0	5860	203	203	11.0	7.2	45.5	448	88.1
W200 x42	41.7	5310	205	166	11.8	7.2	40.9	399	87.1
W200 x36	35.9	4580	201	165	10.2	6.2	34.4	342	86.1

Weight Check:

Weight of chosen section = 41.7 kg/m

$$= \frac{41.7 \times 2.2046}{3.28 \times 1000} \text{ k/ft}$$

$$= 0.028 \text{ k/ft}$$

$$\left[\begin{array}{l} 1\text{kg} = 2.2046 \text{ lb} \\ 1\text{m} = 3.28 \text{ ft} \end{array} \right]$$

Weight of assumed section = 0.032 k/ft

$\because 0.028 \text{ k/ft} < 0.032 \text{ k/ft}$

$\therefore$ Comment: Design is okay.

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RAJSHAHI UNIVERSITY OF ENGINEERING & TECHNOLOGY

Department Of Civil Engineering

Expt No. 03

Name of Expt. Design of a Plate Girder Bridge (Floor Beam Design)

	SUBMITED BY :
SUBJECT: Structural Analysis & Design Sessional-I	NAME: Most. Afrin Sultana
COURSE NO. : CE 3112	GROUP :
DATE OF EXPT. : 16-01-2021	ROLL NO: 1700082
DATE OF SUB. : 06-03-2021	SESSION : 2017-18

Chapter No: 03 Date: 16-01-2021

Chapter Name: Design of a Plate Girder Bridge. (Floor Beam Design)

Floor Beam Design:

Span = Width of the bridge, $W = 40.5'$

Dead load calculation:

(1) Assume self weight of floor beam, $w_D = 0.2$ klf

(2) Dead load of slab $= \frac{\text{slab thickness}}{12} \times 0.15 \times S_{acs} \times S_{acf}$

$= \left(\frac{8}{12} \times 0.15 \times 4.5 \times 11.05\right)$ k

$= 4.97$ k

(3) Load of stringer = unit weight of stringer $\times S_{acf}$

$= (0.028 \times 11.05)$ k $= 0.31$ k

P_I = Concentrated load [Interior] $= [(2)+(3)]$ k

$= 4.97 + 0.31$

$= 5.28$ k

P_E = Concentrated load [Exterior] $= [0.5 \times (2) + (3)]$ k

$= [(0.5 \times 4.97) + 0.31]$ k

$= 2.79$ k

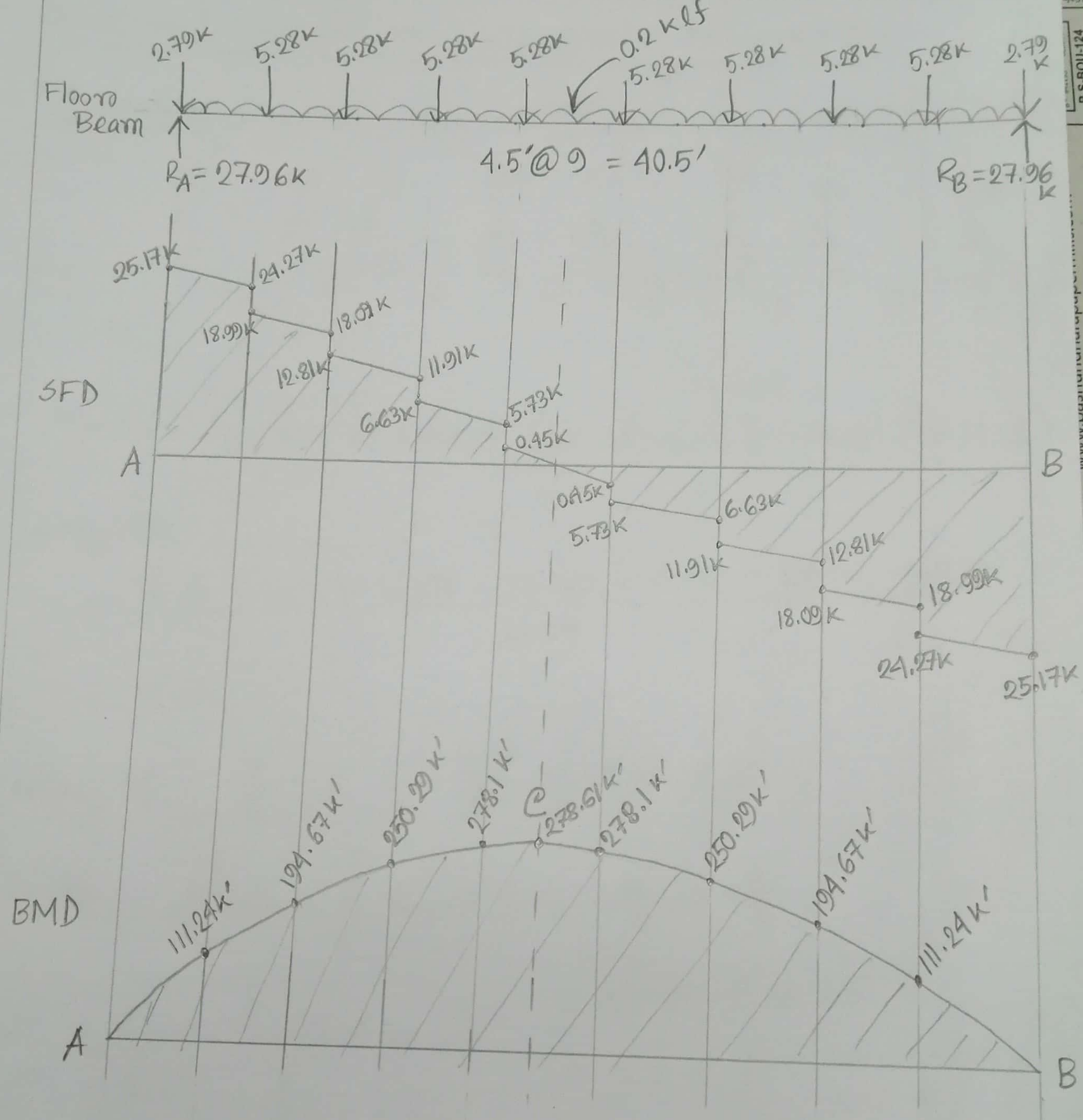

Fig 3.1: SFD & BMD of floor beam dead load.

Reaction at point A = R_A = 27.96 k

Maximum shear at point A, V_A = 25.17 k

Maximum moment at point C, M_C = 278.61 k-ft

Live load calculation : Absolute Moment :

Case-1:

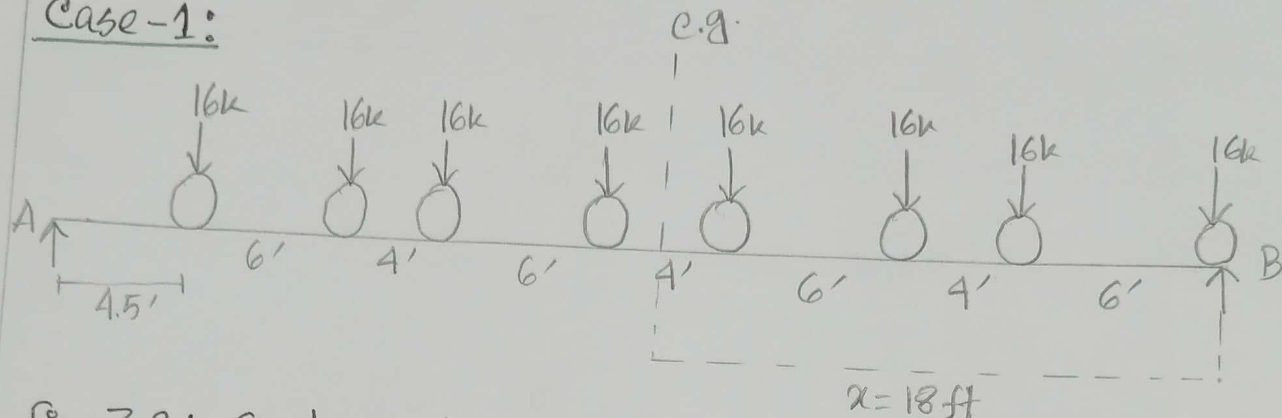

Fig-3.2: System of wheel for absolute moment (case-1)

$\Sigma M_B = 0;$

$$x = \frac{16\times6 + 16\times10 + 16\times16 + 16\times20 + 16\times26 + 16\times30 + 16\times36}{16\times8}$$

$= 18$ ft

Now, $a/2 = \frac{W}{2} - x = \frac{40.5}{2} - 18 = 2.25$ ft

Position of wheel 4 from B $= \frac{W}{2} + \frac{a}{2}$

$= 20.25 + 2.25$

$= 22.5$ ft

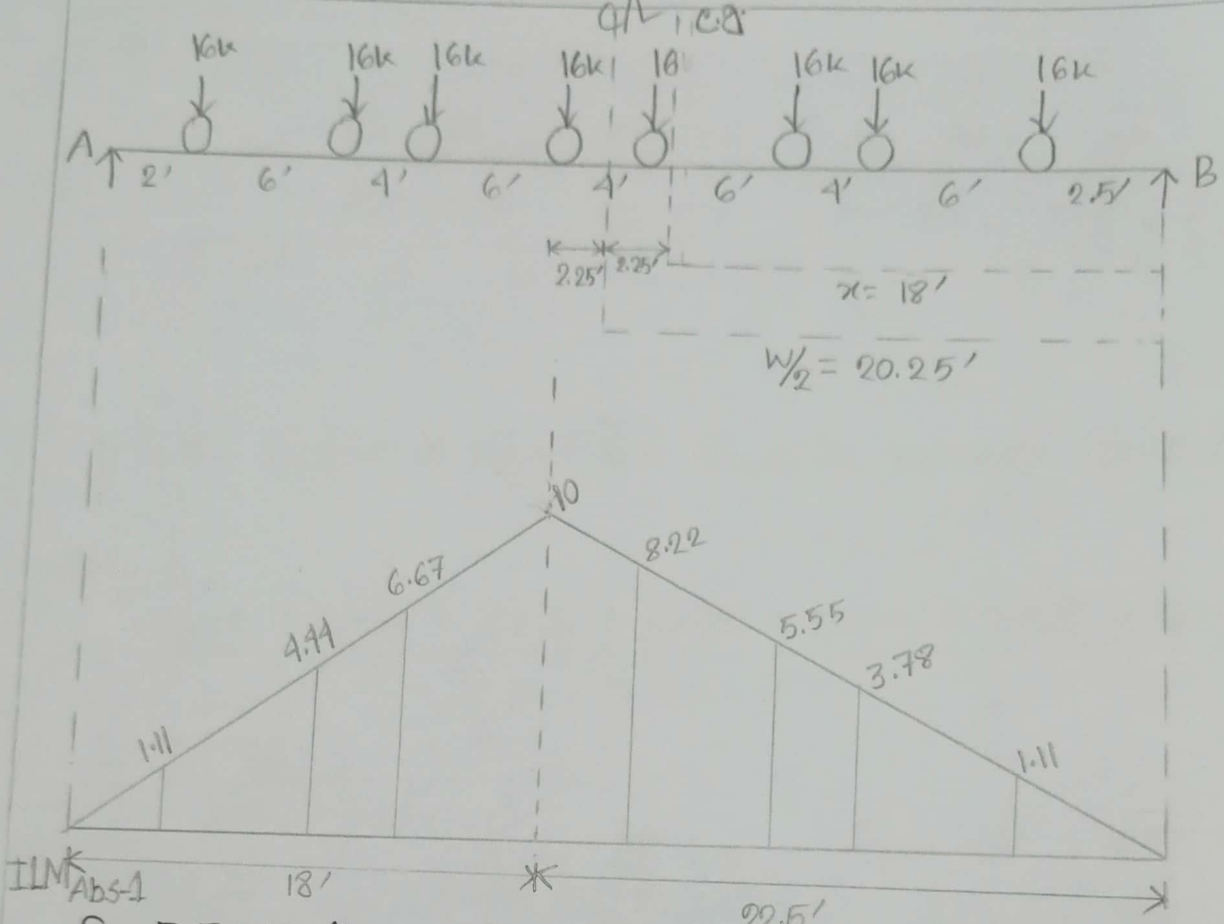

Fig 3.3: Influence line for absolute moment (case-1)

Maximum absolute live load moment,

$$LLM_{Abs1} = 16\times10 + 16\times6.67 + 16\times4.44 + 16\times1.11 + 16\times8.22 + 16\times5.55 + 16\times3.78 + 16\times1.11$$

$$= 654.08 \text{ k-ft}$$

Impact Co-efficient,

$$IC = \frac{50}{W+125} = \frac{50}{40.5+125} = 0.3 \quad [IC_{max} = 30\%]$$

$$IM_{Abs1} = IC \times LLM_{Abs1}$$

$$= 0.3 \times 654.08 = 196.224 \text{ k-ft}$$

Case-2:

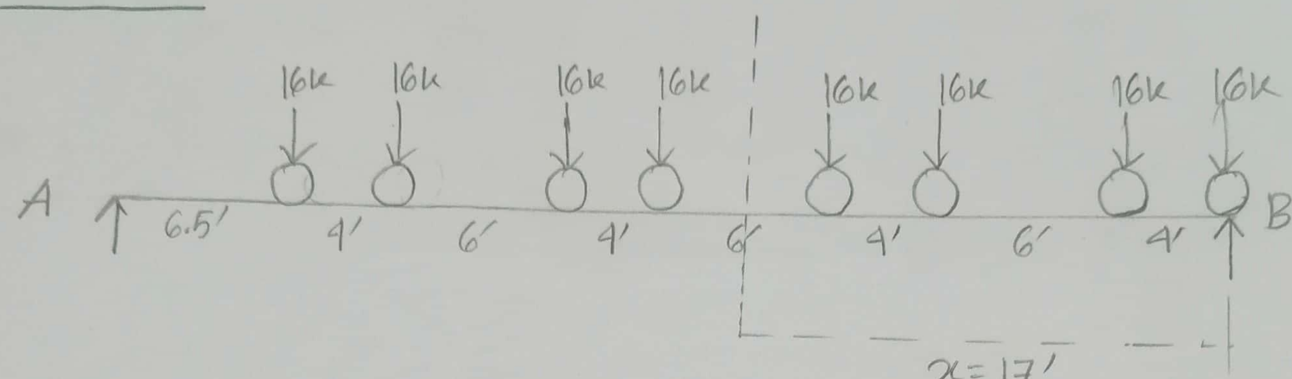

Fig 3.4: System of wheel for absolute moment (case-2)

$\Sigma M_B = 0$;

$$x = \frac{16\times4 + 16\times10 + 16\times14 + 16\times20 + 16\times24 + 16\times30 + 16\times34}{16\times8}$$

$$= \frac{2176}{128}$$

$= 17$ ft $\qquad \frac{a}{2} = \frac{W}{2} - x = \frac{40.5}{2} - 17 = 3.25$ ft

Position of wheel 4 from B $= \frac{W}{2} + \frac{a}{2}$

$= 20.25 + 3.25$

$= 23.5'$

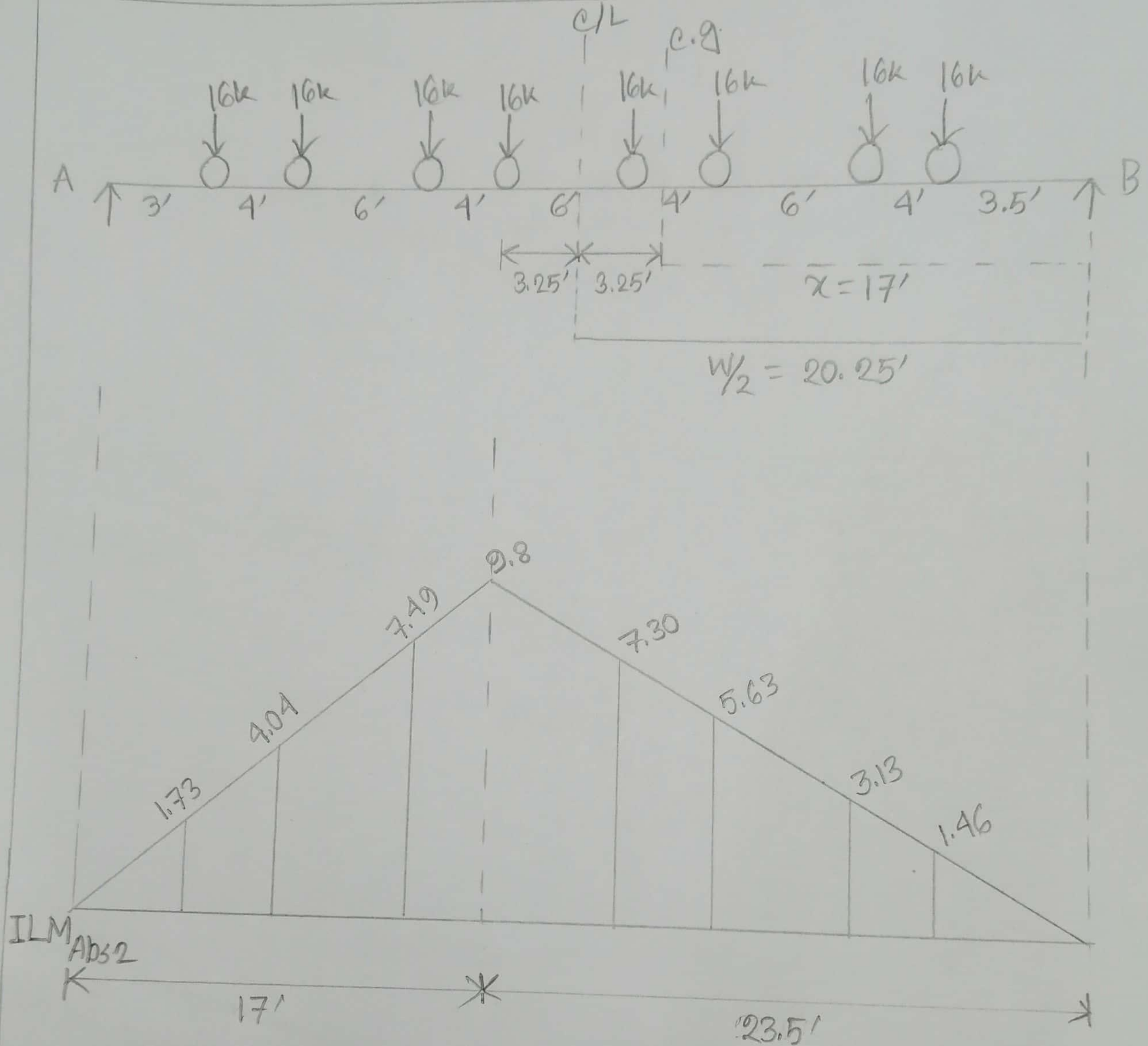

Fig 3.5: Influence line for absolute moment (case-2)

$$LLM_{Abs2} = 16\times1.73 + 16\times4.04 + 16\times7.49 + 16\times9.8 + 16\times7.30 + 16\times5.63 + 16\times3.13 + 16\times1.46$$

$$= 649.28 \text{ k-ft}$$

$$IM_{Abs2} = IC \times LLM_{Abs2}$$

$$= 0.3 \times 649.28$$

$$= 194.784 \text{ k-ft}$$

Live load calculation : Shear:

Case-1:

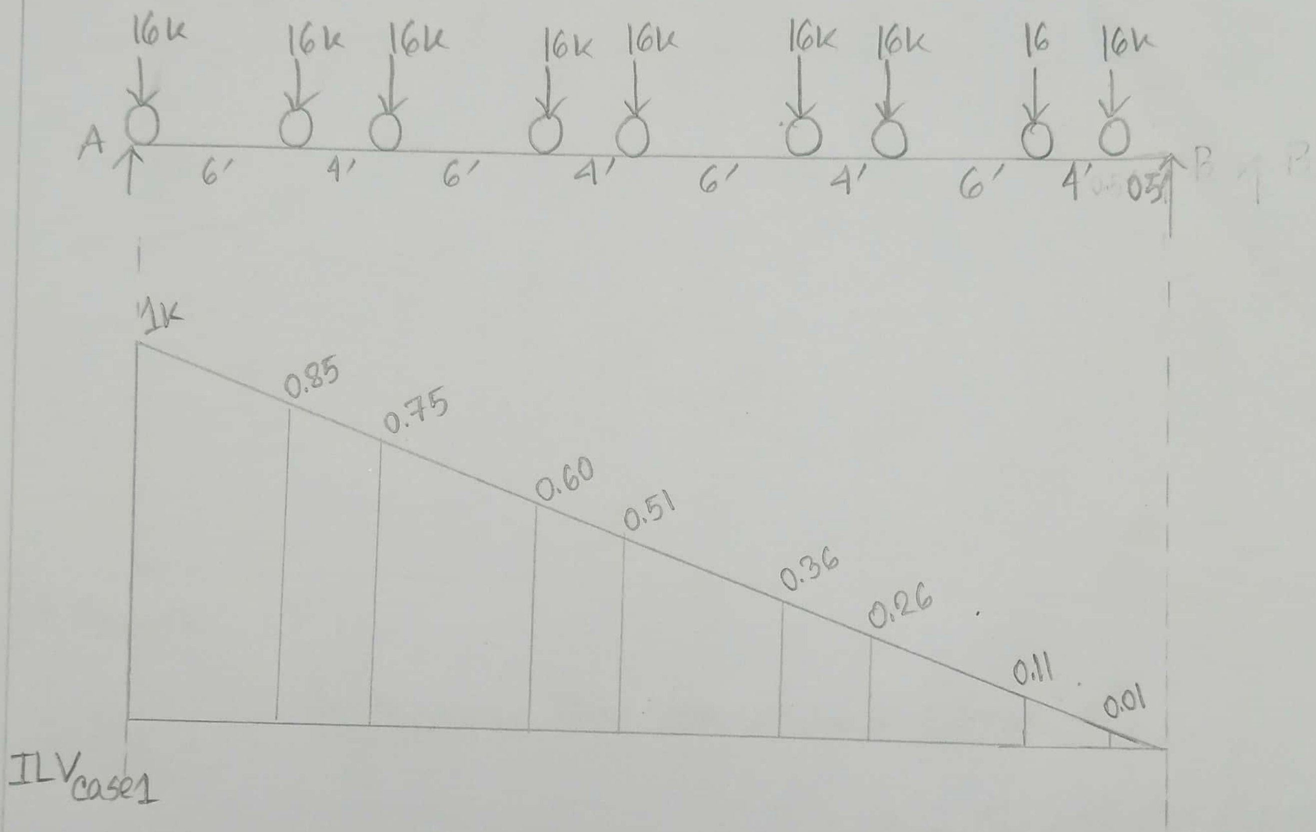

Fig 3.6: Influence line for shear (Case-1).

$$LLV_{case1} = (16\times1)+(16\times0.85)+(16\times0.75)+(16\times0.60)+(16\times0.51)+(16\times0.36)+(16\times0.26)+(16\times0.11)+(16\times0.01)$$

$$= 71.2\ k$$

$$IV_{case1} = IC \times LLV_{case1}$$

$$= 0.3 \times 71.2$$

$$= 21.36\ k$$

Case-2:

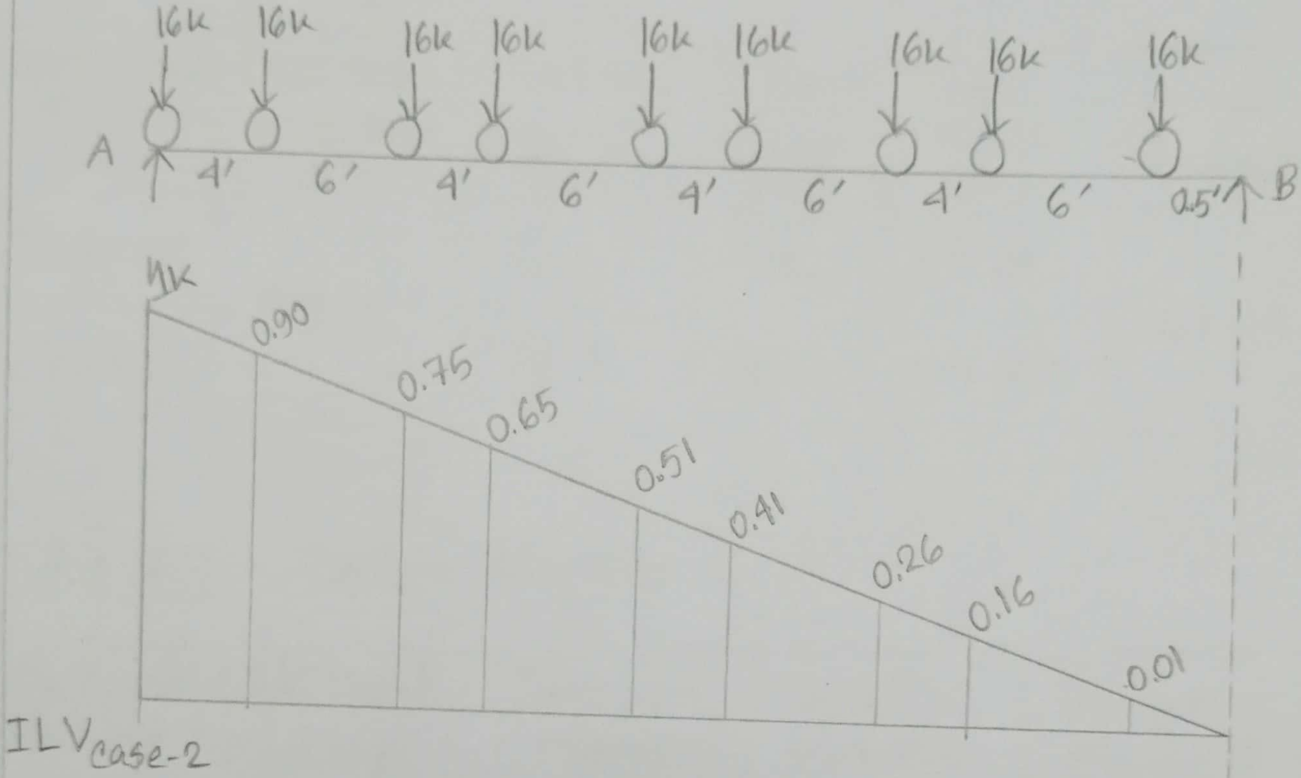

Fig 3.7: Influence line for shear (Case-2)

$$LLV_{Case2} = (16\times1) + (16\times0.90) + (16\times0.75) + (16\times0.65) + (16\times0.51) + (16\times0.41) + (16\times0.26) + (16\times0.16) + (16\times0.01)$$

$$= 74.4\ k$$

$$IV_{Case2} = 0.3 \times 74.4 = 22.32\ k$$

Table 3.1: Design Shear

Dead Load Shear [k]	Dead Load Shear Max [k]	Live Load Shear [k]	Impact Shear [k]	Total Shear [k]	Design Shear [k]
(+) 25.17	25.17	71.2	21.31	117.68	121.89
(−) 25.17		74.4	22.32	121.89	

Table 3.2: Design Moment

Dead Load Moment [k-ft]	Live Load Moment [k-ft	Impact Moment [k-ft]	Total Moment [k-ft]	Design Moment [k-ft]
278.61	654.08	196.224	1128.914	1128.914
	649.28	194.784	1122.674	

Section Choice:

Design Moment, $M = 1128.914$ k-ft

Allowable stress, $\sigma = \frac{Mc}{I} = \frac{M}{I/c} = \frac{M}{Z}$

$$\therefore Z = \frac{M}{\sigma}$$

$$= \frac{1128.914 \times 1000 \times 12}{24000}$$

$$= 564.457 \text{ in}^3$$

Table 3.3: Properties of wide flange section [I section]

Designation	Area (in^2)	Depth (in)	Web Thickness (in)	Flange Width (in)	Flange Thickness (in)	I (in^4)	$S=\frac{I}{c}$ (in^3)	$r=\sqrt{I/A}$ (in)
W36X182	53.6	36.33	0.725	12.075	1.180	11300	623	14.5
W36×170	50.0	36.17	0.680	12.030	1.100	10500	580	14.5
W36×160	47.0	36.01	0.650	12.000	1.020	9750	542	14.4

Section for shear Check:

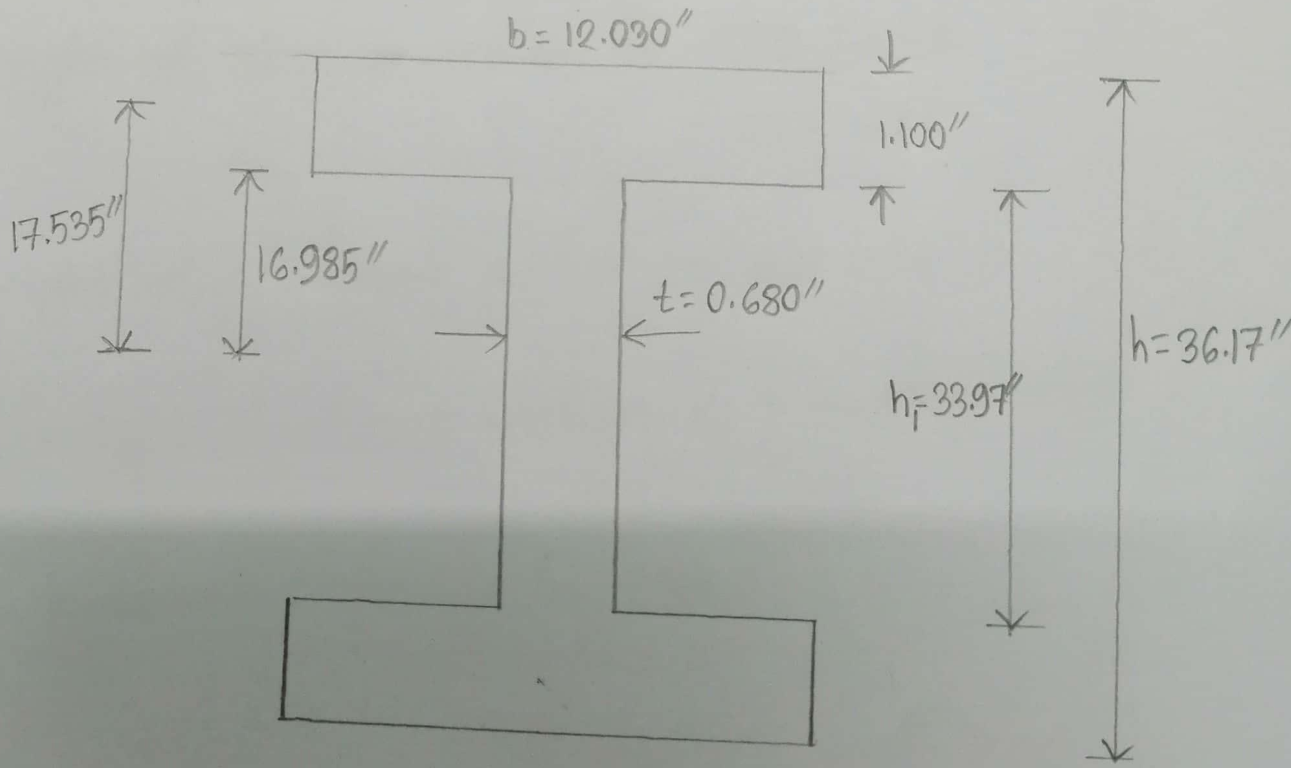

Fig 3.8: I section

$$Q = A' \times \bar{y} = (12.030 \times 1.100 \times 17.535) + \left(0.680 \times \frac{16.985^2}{2}\right)$$

$$= 330.13 \text{ in}^3$$

Shearing stress,

$$\tau = \frac{VQ}{Ib}$$

$$= \frac{121.89 \times 330.13}{10500 \times 0.680}$$

$$= 5.64 \text{ ksi} < \text{Allowable shearing stress } 10 \text{ ksi}$$

Section for weight check:

Weight of chosen section is 170 lb/ft
= 0.17 klf

Weight of assumed section = 0.2 klf

∴ Weight of chosen section < Assumed weight

∴ Comment : Design is okay.

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RAJSHAHI UNIVERSITY OF ENGINEERING & TECHNOLOGY

Department Of Civil Engineering

Expt No. 04

Name of Expt. Design of a Plate Girder Bridge (Girder Design)

SUBJECT: Structural Analysis & Design Sessional-I	**SUBMITED BY :**
COURSE NO. : CE 3112	**NAME:** Most. Afrin Sultana
DATE OF EXPT. : 23-01-2021	**GROUP :**
DATE OF SUB. : 06-03-2021	**ROLL NO:** 1700082
	SESSION : 2017-18

Chapter No: 04

Date: 23-01-2021

Chapter Name: Design of a Plate Girder Bridge. (Girder Design)

Girder Design:

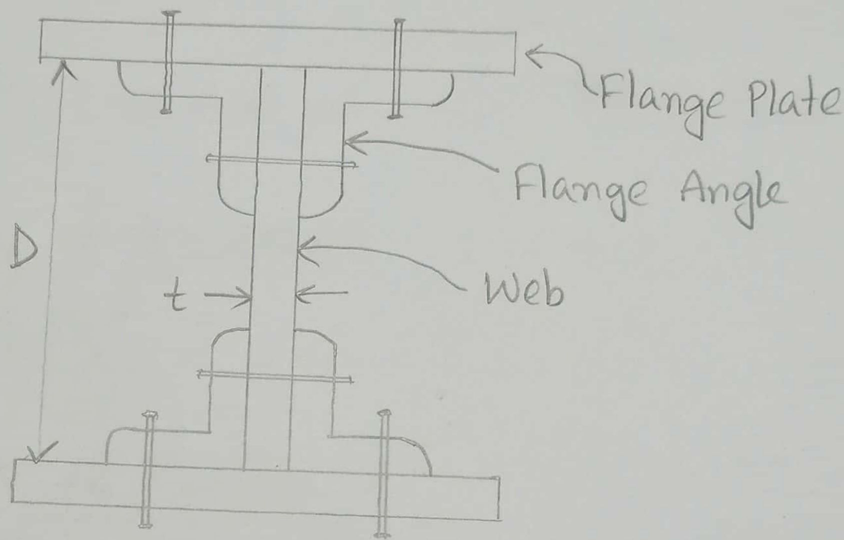

Fig 4.1: Girder

Depth of web plate, D depends on maximum moment

Thickness of web plate, t depends on maximum shear.

Maximum web thickness [AASHTO]

$$t_{max} = \frac{D\sqrt{f_b}}{2300} \qquad f_b = 0.55 f_y$$

tmin	f_y (psi)
D/165	36000
D/150	42000
D/145	46000
D/140	50000

Girder - Section design :

Minimum depth of Girder, $D = (\text{Span}/12)''$

$$= \frac{77.33}{12}$$

$$= 6.44 \text{ in}$$

$$f_y = \frac{24000}{0.4} = 60,000 \text{ psi}$$

$$f_b = 0.55\, f_y = 33,000 \text{ psi}$$

$$t_{max} = \frac{D\sqrt{f_b}}{2300}$$

$$= \frac{6.44\sqrt{33000}}{2300}$$

$$= 0.51$$

According to table $t_{min} = \dfrac{D}{140} = \dfrac{6.44}{140}$

$$= 0.05$$

$\therefore\ t_{max} > t = (3/8)'' > t_{min}$

Girder - Flange Design :

No. of Girders = 2

Section of Girders = $t'' \times D''$

$= 0.375 \times 6.44$

$= 2.42 \text{ in}^2$

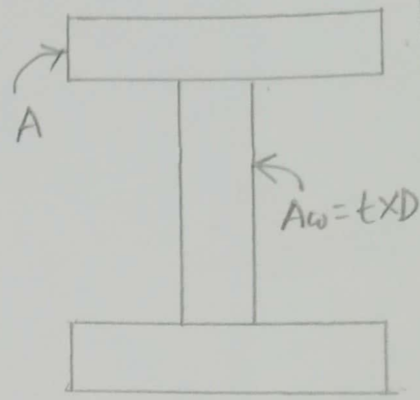

Fig 4.2 : Girder Flange

Assumption

Net Area of one Flange = A

Area of web = A_w

Effective depth = D

Web thickness = t

Bending moment = M

Maximum fiber stress = f

Moment of Inertia of total section = I

Now,

$$I = 2A\left(\frac{D}{2}\right)^2 + \frac{tD^3}{12}$$

$$= 2A\frac{D^2}{4} + \frac{tD^3}{12}$$

$$= \frac{D^2}{2}\left(A + \frac{tD}{6}\right)$$

$$I = \frac{D^2}{2}\left(A + \frac{A\omega}{6}\right)$$

$$Z = \frac{I}{c} = \frac{\frac{D^2}{2}\left(A + \frac{A\omega}{6}\right)}{D/2}$$

$$= D\left(A + \frac{A\omega}{6}\right)$$

$$f = \frac{Mc}{I} = \frac{M}{Z}$$

$$M = f \times Z$$

Putting value of Z

$$M = f \times D\left(A + \frac{A\omega}{6}\right)$$

Assume pitch of the rivet is 4 times the diameter of the rivet holes.

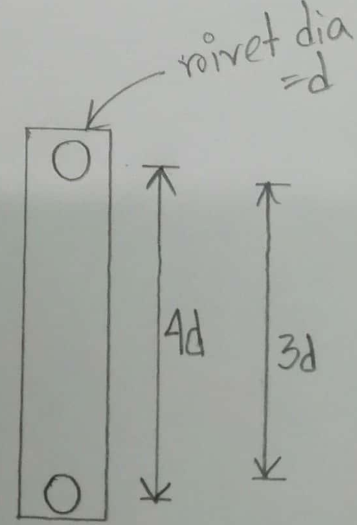

Fig 4.2: Rivet

Effective web area $= \frac{3}{4} A_\omega$

$= \frac{3}{4} Dt$

$= \frac{3}{4} \times 2.42$

$= 1.81 \text{ in}^2$

Self weight of web plate, $\omega_D = \frac{3/4\, Dt \times 12 \times 12}{12 \times 12 \times 12} \times 0.490$

$= \frac{1.81 \times 12 \times 12}{12 \times 12 \times 12} \times 0.490$

$= 0.07 \text{ k/ft}$

Assume, web area, A_ω = Flange area, A

Assumed Girder weight = 3 × (self weight of web plate)

$= 3\,\omega_D$

$= 3 \times 0.07$

$= 0.22 \text{ k/ft}$

Dead load calculation:

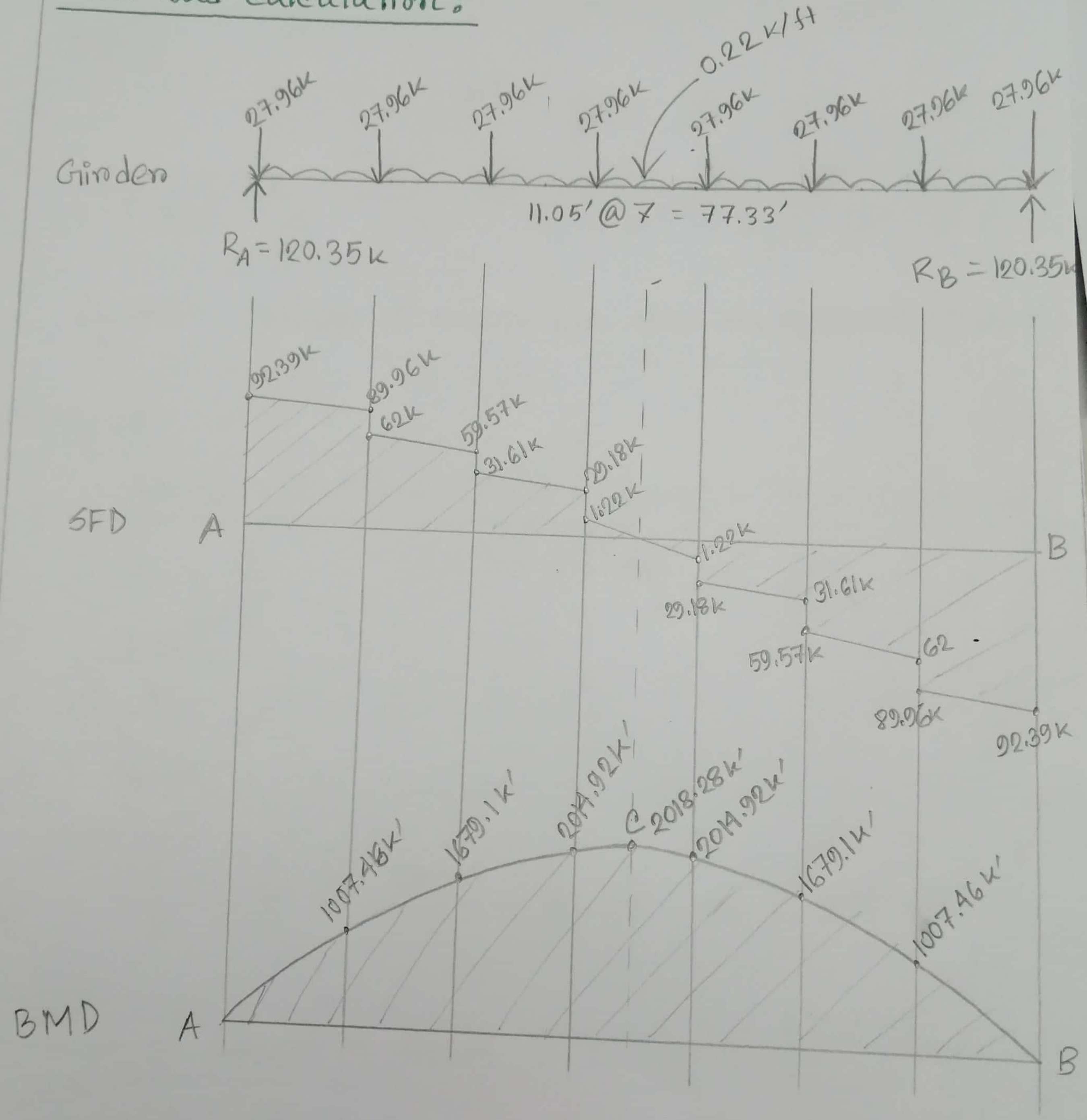

Fig 4.4: SFD & BMD for dead load

Reaction at point A, $R_A = 120.35\ k$

Maximum shear at point A, $V_A = 92.39\ k$

Maximum moment at point C, $M_C = 2018.28\ k\text{-}ft$

Live load calculation : Absolute Moment:

Case-1:

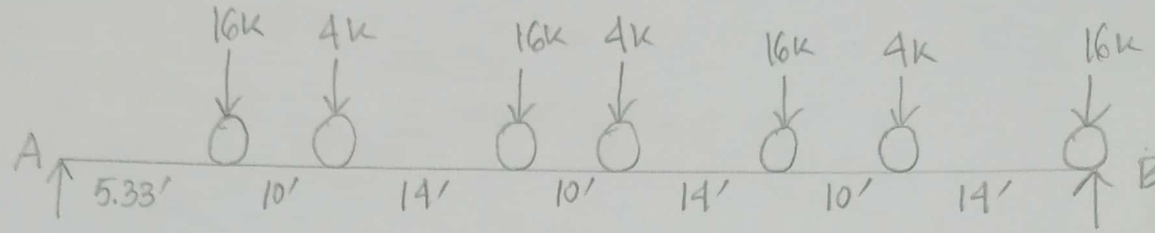

Fig 4.5: System of wheel forr absolute moment (case-1)

$\Sigma M_B = 0$;

$$x = \frac{(4\times14)+(16\times24)+(4\times38)+(16\times48)+(4\times62)+(16\times72)}{(4\times3)+(16\times4)}$$

$$= \frac{2760}{76}$$

$$= 36.32'$$

$$\frac{a}{2} = \frac{L}{2} - x = \frac{77.33}{2} - 36.32 = 2.345 \text{ ft}$$

Position of wheel 4 froom B $= \frac{L}{2} + \frac{a}{2}$

$$= 38.665 + 2.345$$

$$= 41.01 \text{ ft}$$

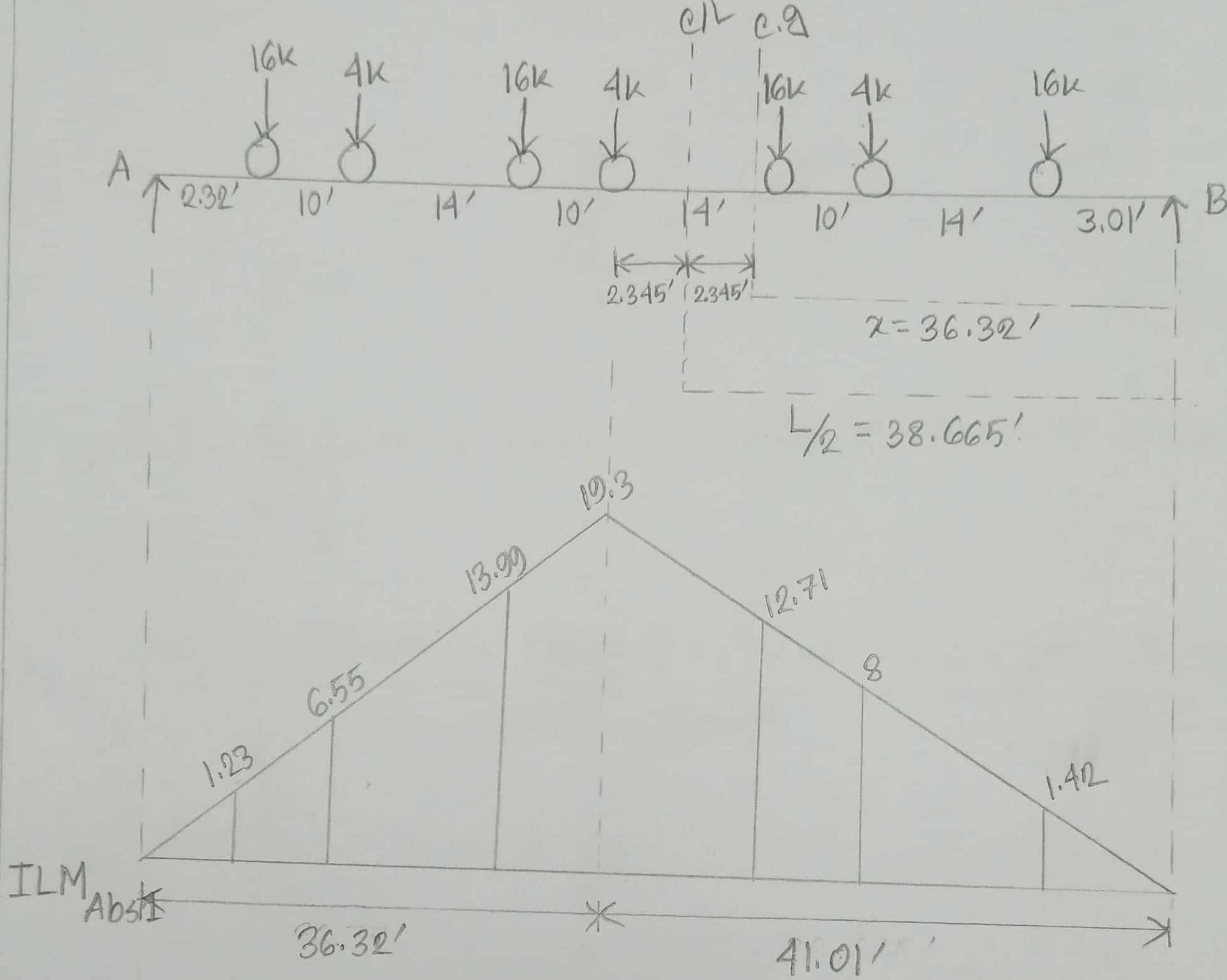

Fig 4.6: Influence line for absolute moment (Case-1)

$$LLM_{Abs1} = (16 \times 1.23) + (4 \times 6.55) + (16 \times 13.99) + (4 \times 19.3) + (16 \times 12.71) + (4 \times 8) + (16 \times 1.42)$$

$$= 605 \text{ k-ft}$$

$$IC = \frac{50}{L+125} = \frac{50}{77.33+125} = 0.25 \quad [IC_{max} = 30\%]$$

$$IM_{Abs1} = IC \times LLM_{Abs1}$$

$$= 0.25 \times 605 = 151.25 \text{ k-ft}$$

Case-2:

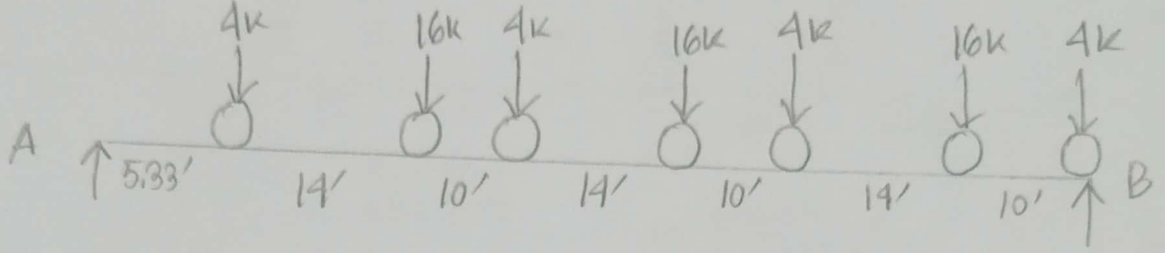

Fig 4.7: System of wheel for absolute moment (Case-2)

$\Sigma M_B = 0$;

$$x = \frac{(16\times10)+(4\times24)+(16\times34)+(4\times48)+(16\times58)+(4\times72)}{(4\times4)+(16\times3)}$$

$$= \frac{2208}{64} = 34.5 \text{ ft}$$

$$\frac{a}{2} = \frac{L}{2} - x = \frac{77.33}{2} - 34.5 = 4.165 \text{ ft}$$

Position of wheel 3 from B $= \frac{L}{2} + \frac{a}{2}$

$= 38.665 + 4.165$

$= 42.82'$

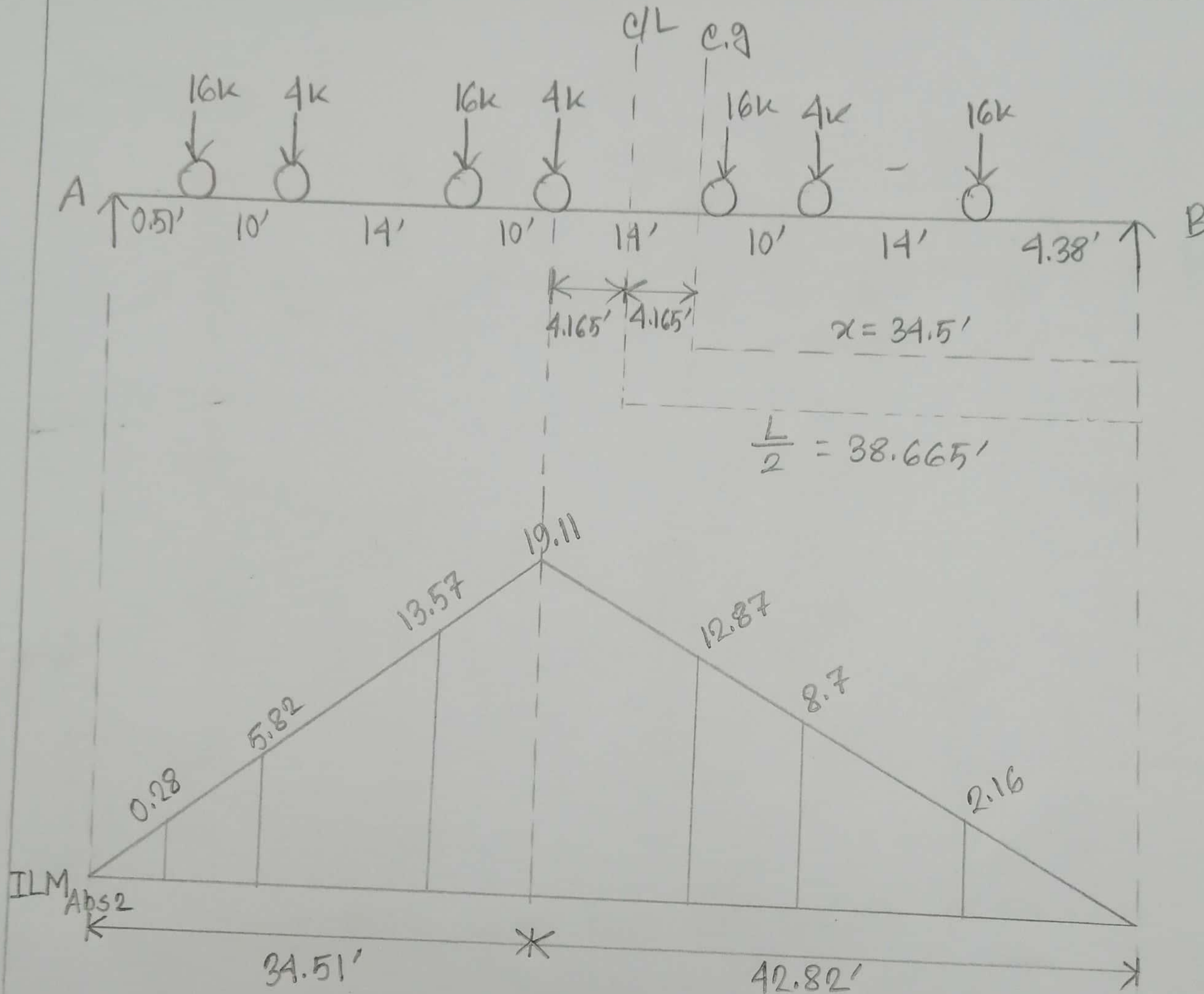

Fig 4.8: Influence line for absolute moment (case-2)

$$LLM_{Abs2} = (16 \times 0.28) + (4 \times 5.82) + (16 \times 13.57) + (4 \times 19.11) + (16 \times 12.87) + (4 \times 8.7) + (16 \times 2.16)$$

$$= 595.4 \text{ k-ft}$$

$$IM_{Abs2} = IC \times LLM_{Abs2}$$

$$= 0.25 \times 595.4$$

$$= 148.85 \text{ k-ft}$$

Live load calculation : shear :

Case-1:

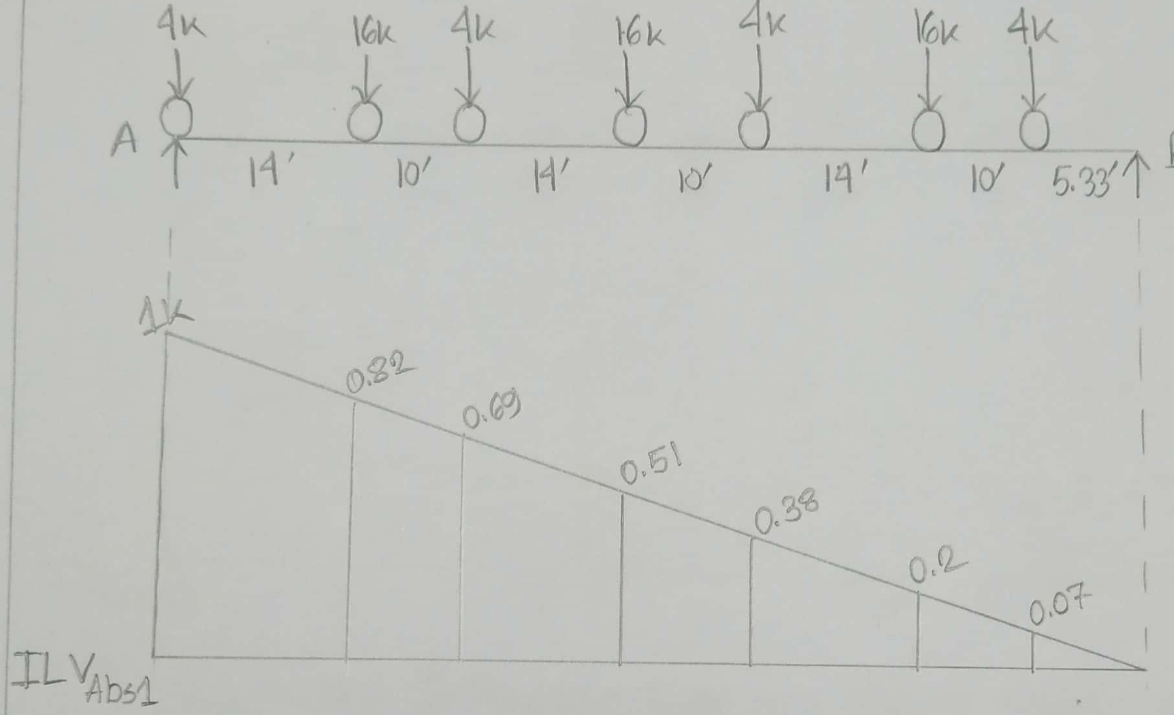

Fig. 4.9: Influence line for shear (case-1)

$$LLV_{case1} = (4\times1)+(16\times0.82)+(4\times0.69)+(16\times0.51)$$
$$+(4\times0.38)+(16\times0.2)+(4\times0.07)$$
$$= 33.04\ k$$

$$IV_{case1} = IC \times LLV_{case1}$$
$$= 0.25 \times 33.04$$
$$= 8.26k$$

Case-2:

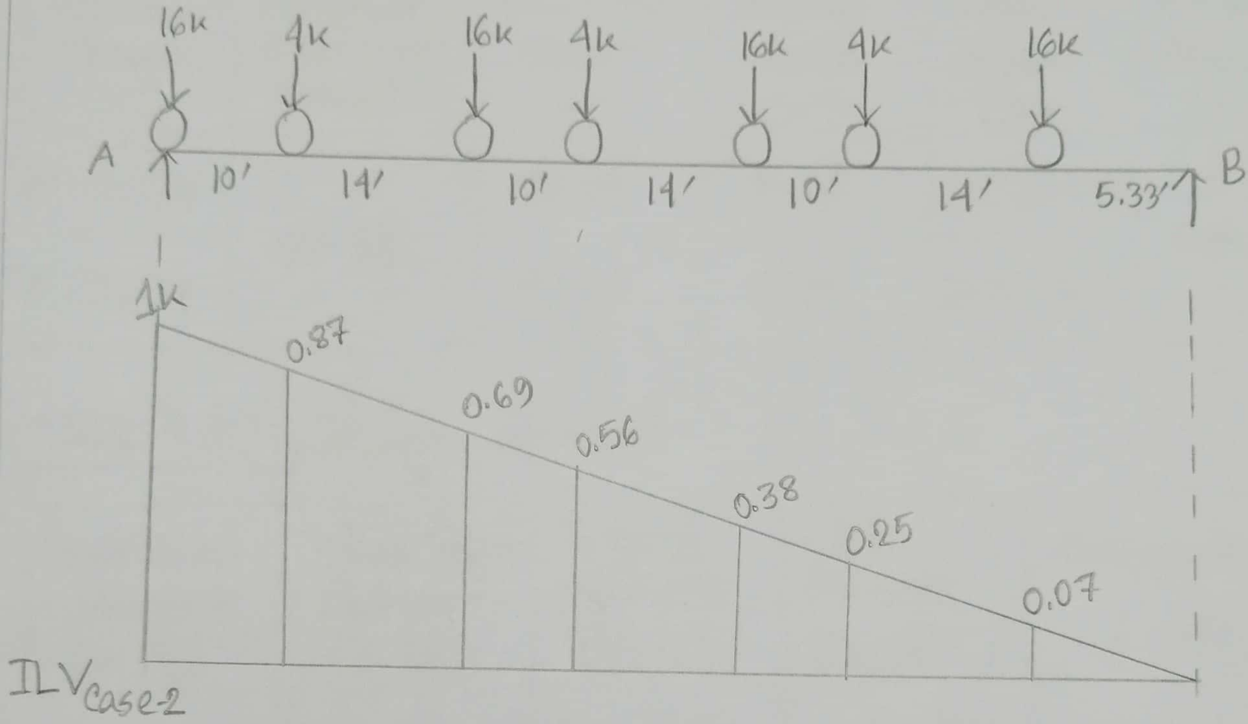

Fig 4.10 : Influence line for shear (case-2)

$$LLV_{case\,2} = (16\times1) + (4\times0.87) + (16\times0.69) + (4\times0.56) + (16\times0.38) + (4\times0.25) + (16\times0.07)$$

$$= 40.96\ k$$

$$IV_{case\,2} = IC \times LLV_{case2}$$

$$= 0.25 \times 40.96$$

$$= 10.24\ k$$

Table 4.1: Design Shear

Dead Load Shear [k]	Maximum Dead Load Shear [k]	Live Load Shear [k]	Impact Shear [k]	Total Shear [k]	Design Shear [k]
(+) 92.39	92.39	33.04	8.26	133.69	143.59
(-) 92.39		40.96	10.24	143.59	

Table 4.2: Design moment

Dead Load Moment [k-ft]	Live Load Moment [k-ft]	Impact Moment [k-ft]	Total Moment [k-ft]	Design Moment [k-ft]
2018.28	605	151.25	2774.53	2774.53
	595.4	148.85	2762.53	

Girder-Section Design:

$$M = f \times D \left(A + \frac{\text{Effective } A_w}{6}\right)$$

$$\Rightarrow 2774.53 = 24 \times 6.44 \times \left(A + \frac{1.81}{6}\right)$$

$$\Rightarrow A = 17.64 \text{ in}^2$$

Section for weight check:

$$\text{Total weight} = \frac{(2A + \text{Effective } A_\omega) \times 12}{12 \times 12 \times 12} \times 0.49$$

$$= \frac{(2 \times 17.64 + 1.81) \times 12}{12 \times 12 \times 12} \times 0.49$$

$= 0.13$ k/ft Assumed Girder weight

$3\omega_D = 0.22$ k/ft

Section for shear check

$A = 17.64 \text{ in}^2$

$t = 0.375$ in

$B = 47.03$ in

Thickness of web $= 0.375$ in

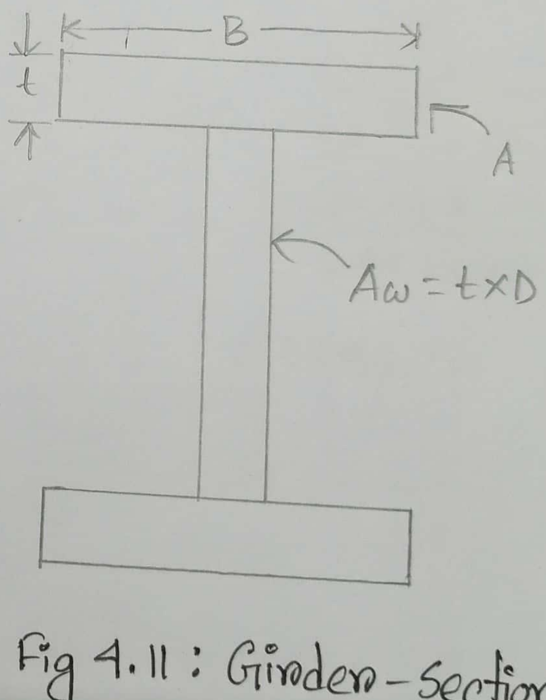

Fig 4.11 : Girder-Section

$$I = \frac{D^2}{2}\left(A + \frac{A_\omega}{6}\right)$$

$$= \frac{6.44^2}{2}\left(17.64 + \frac{2.42}{6}\right)$$

$= 374.597 \text{ in}^4$

$$\tau = \frac{VQ}{Ib} = \frac{143.59 \times 943.59}{374.597 \times 47.03}$$

$= 7.69$ ksi $<$ Allowable shearing stress 10 ksi

Intermediate Stiffener Design :

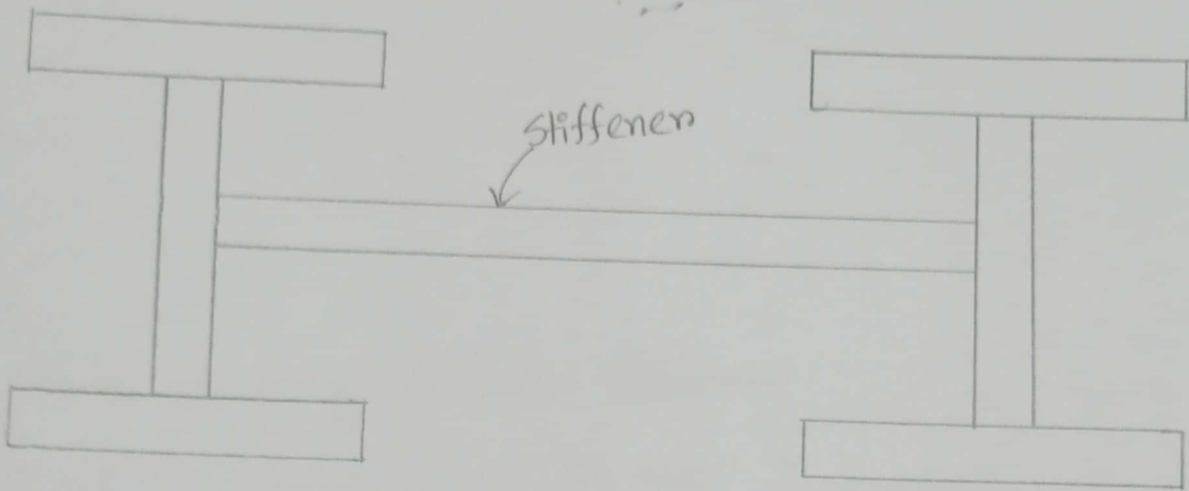

Fig 4.12 : Stiffener

$$t_{check} = \frac{D\sqrt{f_v}}{7500}$$

$$= \frac{6.44 \times \sqrt{59457.56}}{7500}$$

$$= 0.21 < t = 0.375$$

$$f_v = \frac{V}{Dt}$$

$$= \frac{143.59 \times 1000}{6.44 \times 0.375}$$

$$= 59416.56$$

So, no need of stiffener

Maximum spacing between stiffener

$$d_{max} = \frac{11000 \times t}{\sqrt{f_v}}$$

$$= \frac{11000 \times 0.375}{\sqrt{59416.56}}$$

$$= 16.92 \text{ in}$$

$$f_v = \frac{V}{Dt}$$

$$= 59416.56$$

Number of stiffener

$$N_s = \frac{L}{d_{max}} + 1$$

$$= \frac{77.33 \times 12}{16.92} + 1$$

$$= 56$$

Actual spacing between stiffener

$$d_a = \frac{L}{N_s - 1} = \frac{77.33 \times 12}{56 - 1} = 16.872 \text{ in}$$

Intermediate stiffener/section design:

$$J = \left| 25 \times \left(\frac{D}{d_a}\right)^2 - 20 \right|$$

$$= \left| 25 \times \left(\frac{6.44}{16.872}\right)^2 - 20 \right|$$

$$= 16.36$$

Value of $J = 5$ $[J \leq 5]$

Moment of Inertia, $I = \frac{d_a \times t^3 \times J}{10.92}$

$$= \frac{16.872 \times (0.375)^3 \times 5}{10.92}$$

$$= 0.41 \text{ in}^4$$

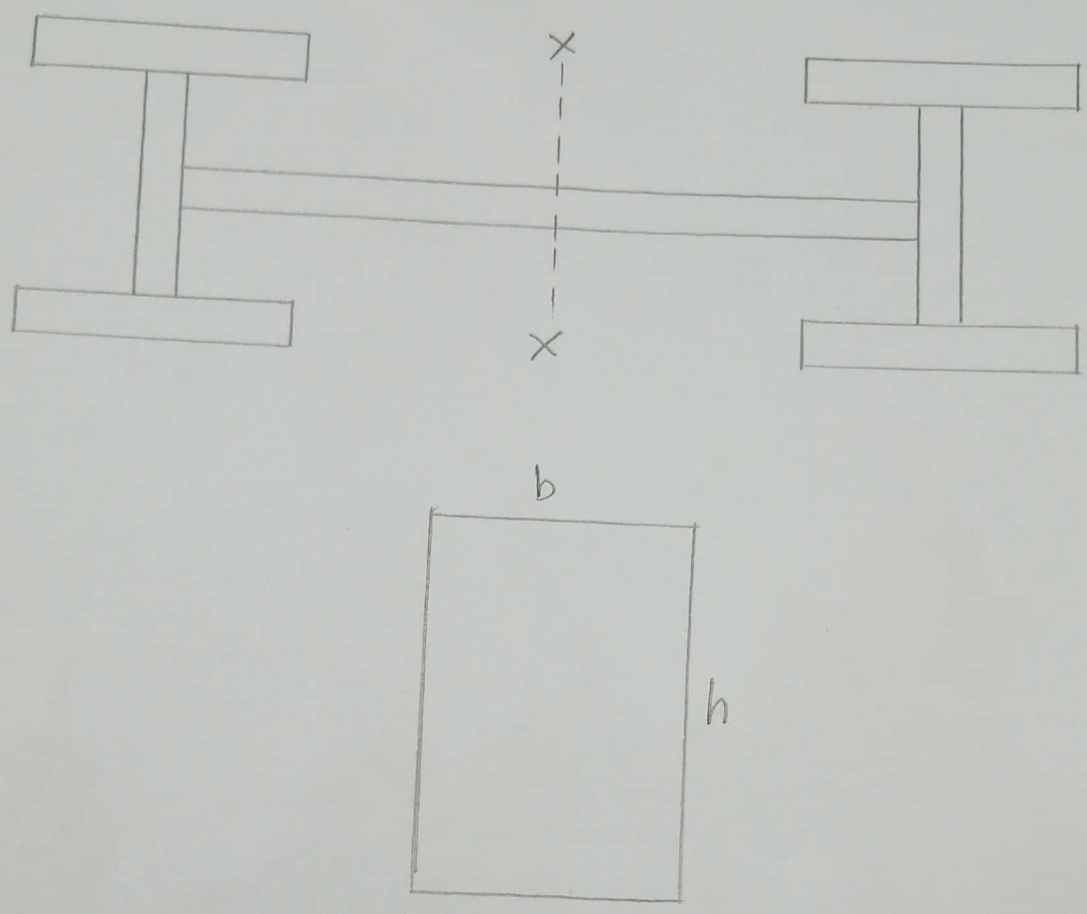

Fig 4.13 : Intermediate stiffener / section

Assume, $b = \left(\frac{5}{16}\right)''$

$I = 0.41 \text{ in}^4$

$$I = \frac{bh^3}{12}$$

$$\Rightarrow 0.41 = \frac{5/16 \times h^3}{12}$$

$$\Rightarrow h = 2.51''$$

Section $5/16'' \times 2.5''$

Heaven's Light Is Our Guide

RAJSHAHI UNIVERSITY OF ENGINEERING & TECHNOLOGY

Department Of Civil Engineering

Expt No. ....05........

Name of Expt. ....Introduction of Truss..

....

SUBJECT:....Structural Analysis & Design Sessional-I	SUBMITED BY :
COURSE NO. :....CE 3112..........	NAME:..Most. Afrin Sultana..........
DATE OF EXPT. :..30-01-2021..	GROUP :........................
DATE OF SUB. :..06-03-2021....	ROLL NO:....1700082..................
	SESSION :.....2017-18................

Chapter No: 05 Date: 30-01-2021

Chapter Name: Introduction of truss.

Introduction: The truss or framed structure are composed of interconnecting members and are supported in such a manner that they are capable of holding applied external force in static equilibrium. It must hold in equilibrium gravity forces that are applied as a consequence of its own weight.

Definition of truss: A truss is a configuration, which is designed to sustain inclined, vertical, horizontal loads occuring at or between its points of support.

Characteristic of a truss:

a) The general configuration of a truss is triangle.

b) The members are connected of their end by frictionless pin.

c) The loads are applied only at joints and not at the intermediate points of a member.

Advantages of truss over beam:

Manufactured roof trusses provide a structurally efficient alternative to beams. They place greater emphasis on axial loading of members and less on bending. Associated advantages of trusses include:

i) Strong but light to erect

ii) Can be made to suit most roof shapes

iii) Less onsite fabrication, therefore less site labour and less effected by bad weather

iv) Factory production allows automated production.

v) Better quality control is possible.

vi) Trusses make maximum structural use of the timber.

vii) Trusses are capable of long spans.

viii) Internal walls are usually non-load bearing therefore lighter weight internal walls are possible.

Types of truss:

They are mainly three types

a) Simple

b) Compound

c) Complex

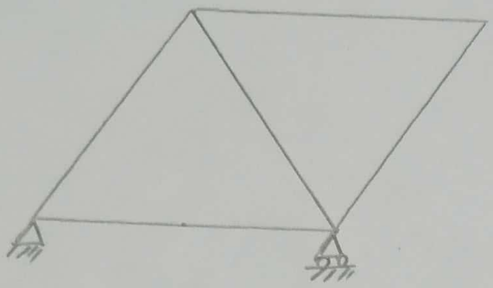

Fig 5.1 :(a) Simple truss

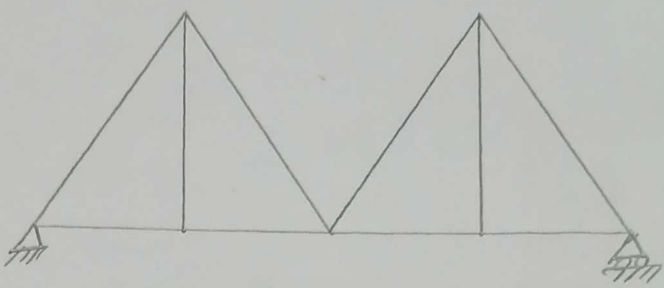

Fig 5.2 :(b) Compound truss

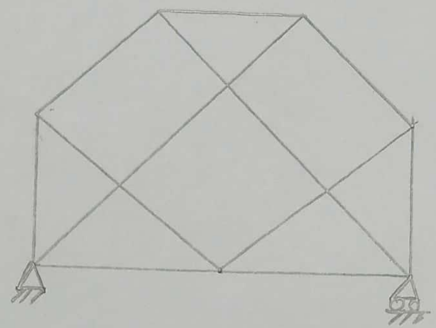

Fig 5.3 : (c) Complex truss

Types of roof truss:

a) Pratt truss,

b) Howe truss,

c) Saw tooth truss,

d) Bow string,

e) Crescent,

f) Three Hinged Arch.

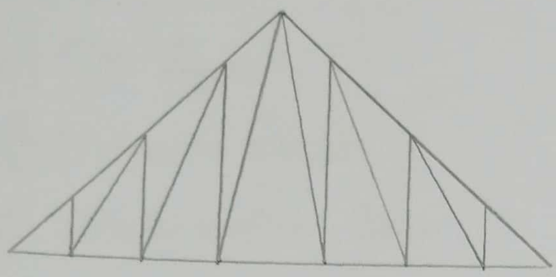

Fig. 5.4: Pratt truss

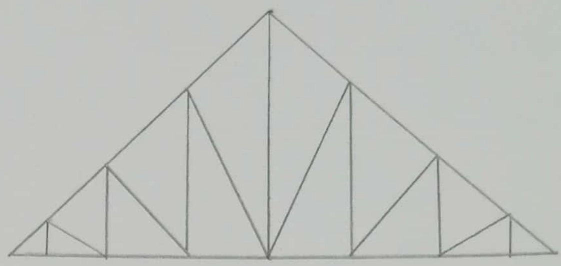

Fig 5.5: Howe truss

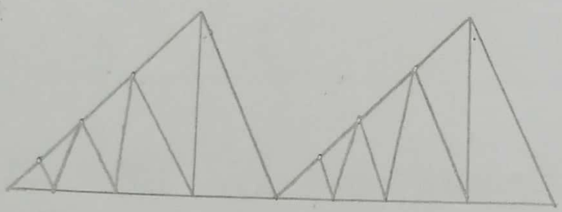

Fig 5.6: Saw Tooth truss

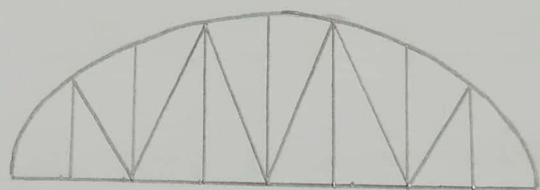

Fig 5.7: Bow string truss

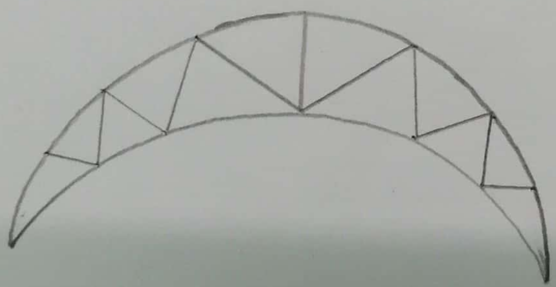

Fig 5.8: Crescent truss

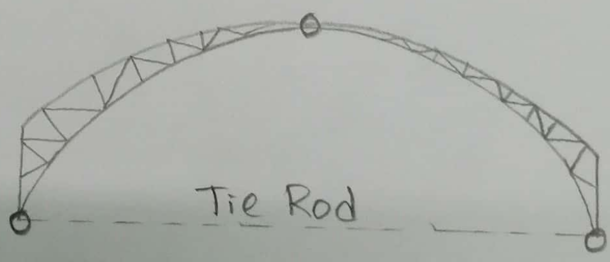

Fig 5.9: Three-Hinged Arch

Type of fink truss :

a) Fan fink
b) Warren fink
c) Compound fink.

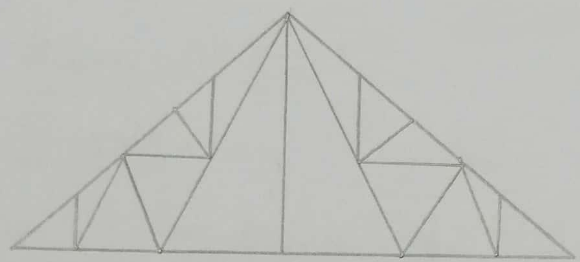

Fig 5.10: Fan fink truss

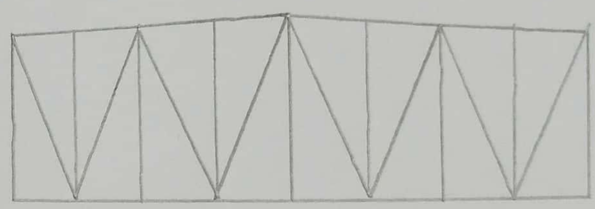

Fig 5.11: Warren truss

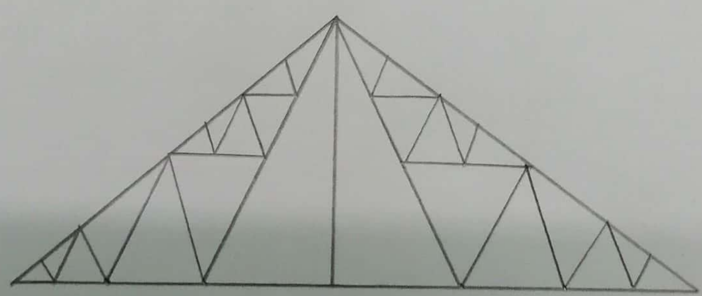

Fig 5.12: Compound fink truss

Assumption in truss:

a) It is assumed that the joints of a truss are made with frictionless pin or hinges.

b) The change in shape of the truss due to change in length of members under stress is so small as to have negligible effect on the computation of axial stress.

Truss Terminology:

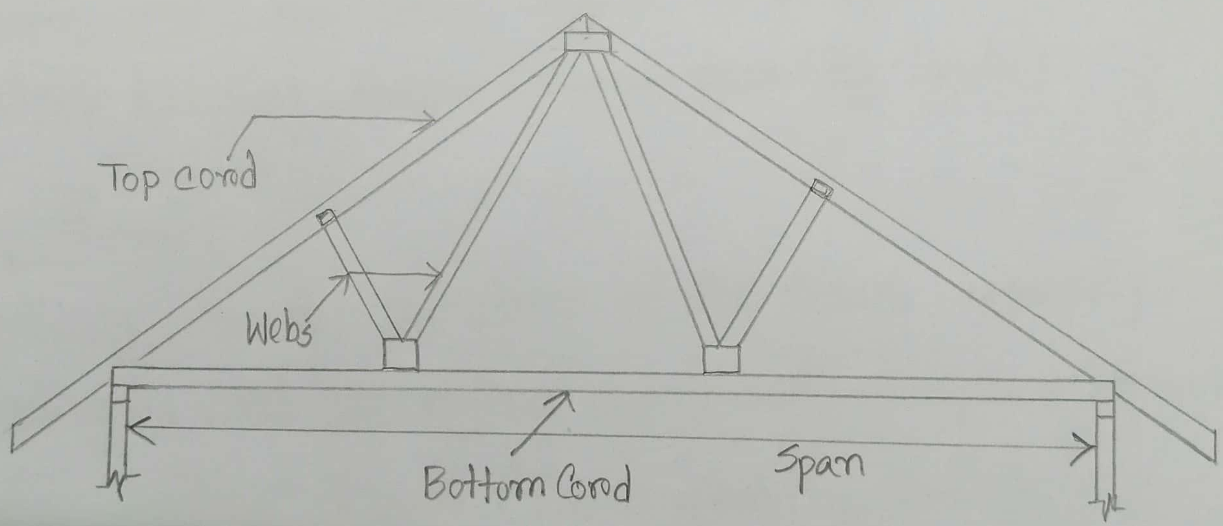

Fig 5.13: Truss Terminology

Given the previous discussion, a truss can be described as a pre-fabricated, engineered building component which functions as a structural support member.

There are three types of trusses but the same basic terms apply :

Members are either top chords, bottom chords or webs.

Each will be in tension or compression according to the type of truss involved

Bottom Chord :

Defines the bottom member of the truss, usually horizontal, and carrying a combined tension and some bending stress (from gravity loads).

Top Chord :

Defines the top members of the truss, usually sloping, and carrying combined-compression and some bending stress (from gravity loads).

Web :

Webs are members joining top and bottom chords to form a truss. They may be in tension or compression depending on the truss design.

Apex:

The top point where two chords meet. This can be either a Top Chord Apex or much less commonly a Bottom Chord Apex (not shown). The Top Chord Apex of multiple trusses in a row, forms the ridge line of the roof.

Heel:

The point on a truss where the undersides of the top and bottom chords join.

Panel points:

The points where web members and chord members meet.

Span:

The distance between the outer edges of the load bearing walls supporting the trusses (usually heel to heel).

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RAJSHAHI UNIVERSITY OF ENGINEERING & TECHNOLOGY

Department Of Civil Engineering

Expt No. 06

Name of Expt. Design of a Fink Type Roof Truss (Purlin Design)

	SUBMITTED BY :
SUBJECT: Structural Analysis & Design Sessional - I	NAME: Most. Afrin Sultana
COURSE NO. : CE 3112	GROUP :
DATE OF EXPT. : 30-01-2021	ROLL NO: 1700082
DATE OF SUB. : 06-03-2021	SESSION : 2017-18

Chapter No: 06 Date: 030-01-2021

Chapter Name: Design of a Fink Type Roof Truss.
(Purlin Design)

Specification:

Area of workshop = 10000 sq.ft

Span of the roof truss $= 70 + \frac{82}{3}$

$= 97.33$ ft

Height of the truss $= \frac{1}{4} \times 97.33 = 24.33$ ft

Spacing of the truss $= \frac{1}{5} \times 97.33 = 19.47$ ft

Wind velocity, $V = 146.74$ mph [Gaibandha]

Roofing shall be corrugated galvanized iron sheet no. 20 B.W.G. (Birmingham Wire Gauge) $= 2.25\ lb/ft^2$

Allowable tensile strength, $f_s = 20$ ksi

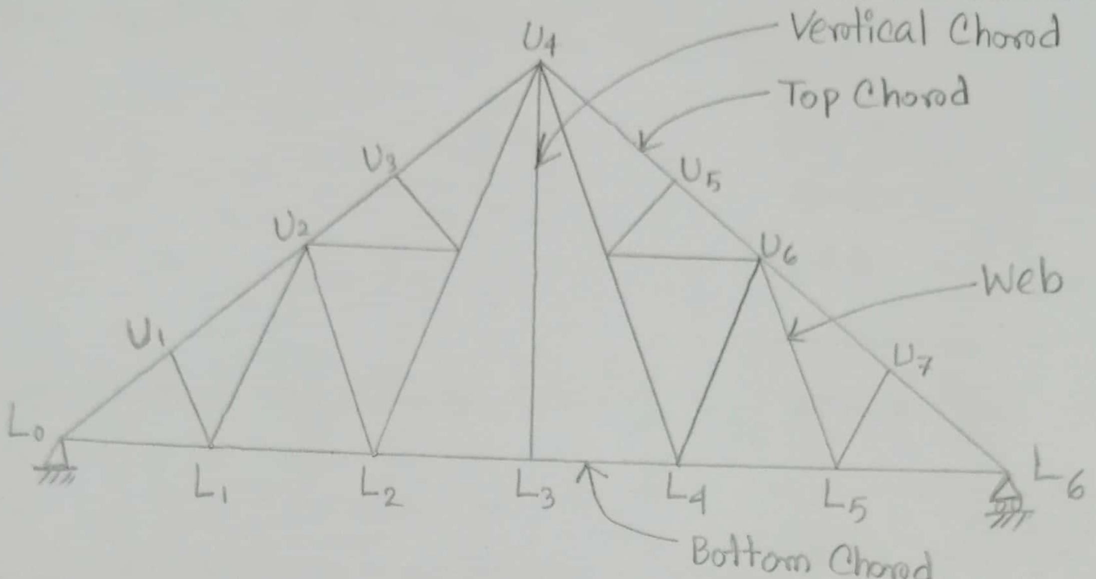

Figure 6.1 : Compound Fink Truss

Design Specification:

Span of the truss, $S_p = 97.33$ ft

Height of the truss, $H = \frac{S_p}{4} = 24.33$ ft

Spacing of the truss, $S = \frac{S_p}{5} = 19.47$ ft

Length of the workshop, $L = \frac{\text{Area}}{S_p}$

$$= \frac{10000}{97.33}$$

$$= 102.74 \text{ ft}$$

No. of truss, $N_t = \frac{L}{S} + 1$

$$= \frac{102.74}{19.47} + 1 = 6$$

Actual spacing of the trouss, $S_a = \frac{L}{N_t - 1}$

$$= \frac{102.74}{6-1}$$

$$= 19.47$$

Length of the top chord [Left] $= \sqrt{H^2 + \left(\frac{Sp}{2}\right)^2}$

$$= \sqrt{(24.33)^2 + \left(\frac{97.33}{2}\right)^2}$$

$$= 54.41 \text{ ft}$$

Angle of inclination top chord, $\theta = \tan^{-1} \frac{H}{Sp/2}$

$$= \tan^{-1} \frac{24.33}{\frac{97.33}{2}}$$

$$= 26.57°$$

Load, $P = kv^2$

$$= 0.003 \times (\text{Wind velocity})^2$$

$$= 0.003 \times 146.74$$

$$= 64.60 \text{ lb/ft}$$

Load Normal, $P_n = P \times \frac{2\sin\theta}{1+\sin^2\theta}$

$= 64.60 \times \frac{2\sin(26.57)}{1+\sin^2(26.57)}$

$= 48.15$ lb/ft

Purlin Design :

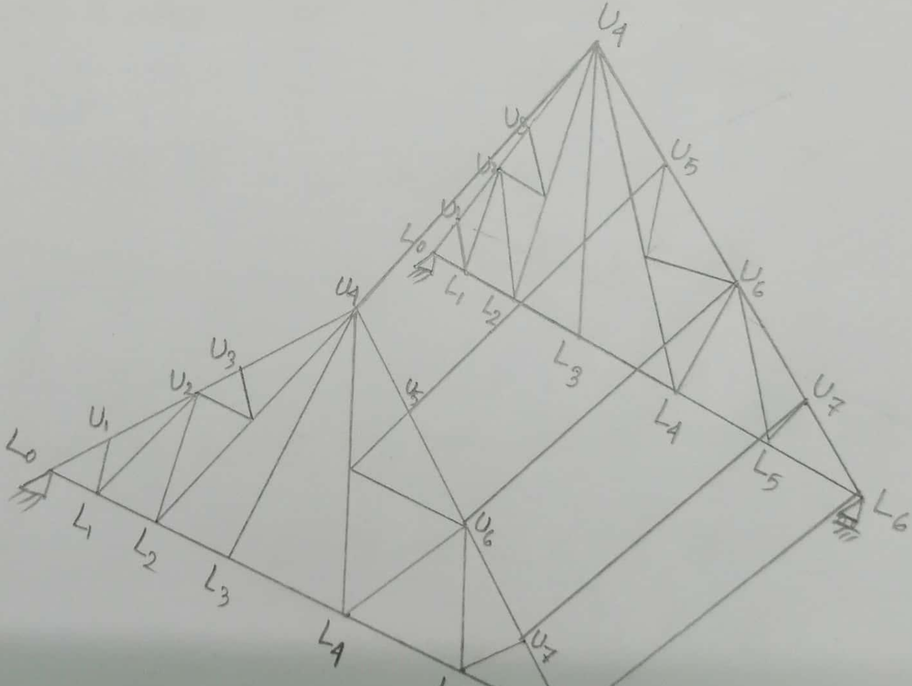

Fig 6.2: Purlin

Length of purlin = Actual spacing of the truss

= 19.47 ft

$$\text{Spacing of purlin} = \frac{\text{Length of top chord}}{4}$$

$$= \frac{54.41}{4}$$

$$= 13.60 \text{ ft}$$

Number of purlin = 4 + 1 = 5

Dead load calculation:

1. Dead load from roof = Spacing of purlin × 1 × wt. of CGI sheet

$= 13.60 \times 1 \times 2.25$

$= 30.61$ lb/ft

2. Load due to wind = Spacing of purlin × 1 × P_n

$= 13.60 \times 1 \times 48.15$

$= 654.95$ lb/ft

3. Self weight of purlin = Spacing of purlin × 1 × 3

$= 13.60 \times 1 \times 3$

$= 40.81$ lb/ft

Total load on purlin $= \omega_D = 30.61 + 654.95 + 40.81$

$= 726.37$ lb/ft

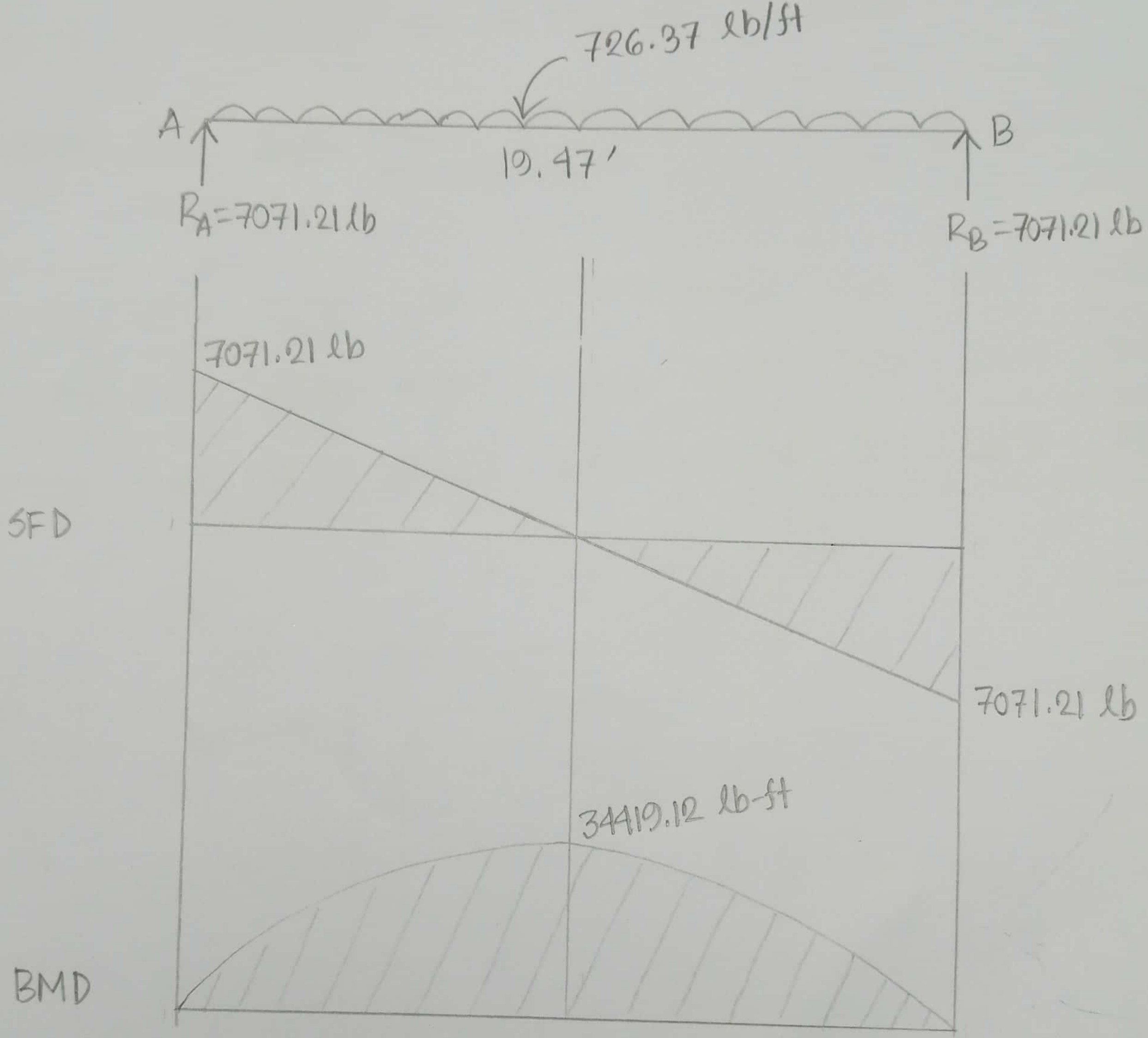

Fig 6.3: SFD & BMD for purlin

$$M_{max} = \frac{\omega_D (S_a)^2}{8} = \frac{726.37 \times (19.47)^2}{8}$$

$$= 34407.11 \text{ lb-ft}$$

$$Z = \frac{M_{max}}{f_s} = \frac{34407.11 \times 12}{20 \times 1000}$$

$$= 20.64 \text{ in}^3$$

Section Choice :

Table 6.1: Prooperoties of channel sections

Designation	Area (in^2)	Depth (in)	Web thickness (in)	Flange Width (in)	Flange Avg. Thickness (in)	I (in^4)	$S = \frac{I}{C}$ (in^3)	$r = \sqrt{\frac{I}{A}}$ (in)
C12×25	7.35	12.00	0.387	3.047	0.501	144	24.1	4.43
C12×20.7	6.09	12.00	0.282	2.942	0.501	129	21.5	4.61
C10×30	8.82	10.00	0.673	3.033	0.436	103	20.7	3.42

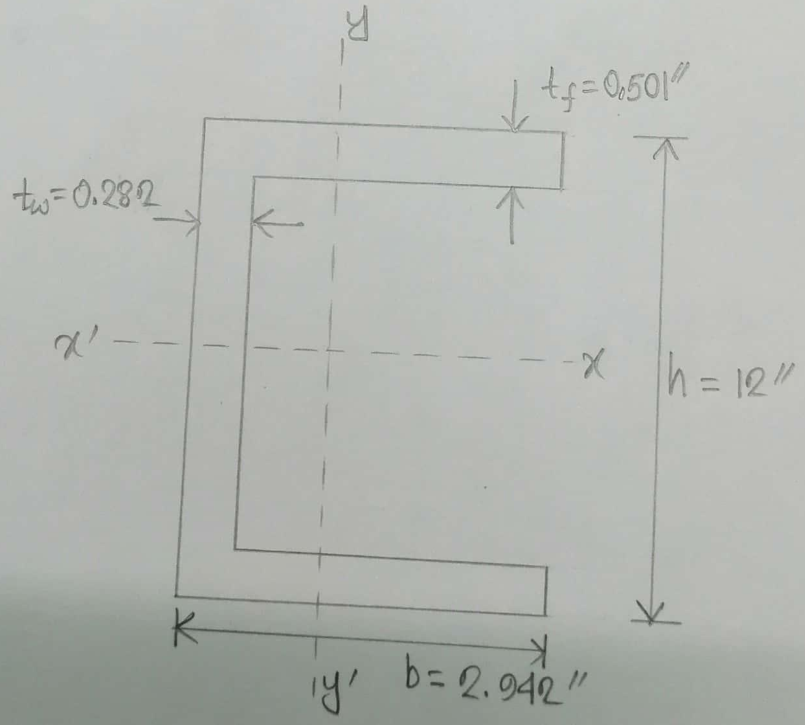

Fig 6.4: Choosen Section.

Design Section = 2.942'' × 12''

Unit weight = 20.7 lb/ft < Self weight of purolin 40.81 lb/ft