PROJECT REPORT

Q1- a. Gummanur is a village in Dharmapuri district in the state of Tamil Nadu in India.

Latitude - 12.555° N

Longitude - 78.1386°E

The runoff in the river "Ponnaiyar" is more in the month of October November since the southern state gets most of its rainfall from retreating monsoon.

The site has the following characteristics :-

1. A straight reach of the river
2. A narrow channel with firm banks
3. In erodible strata close to the riverbed
4. Proximity to a direct alignment of the road to be connected
5. Absence of sharp curves in the approaches to reduce expensive river training works.
6. Suitable high banks above highest flood level (HFL)
7. The approach road is free of any obstacles such as hills and frequent drainage crossing, and the land is economically available

Thus, the site is suitable for bridge construction.

b.

Parameter estimation for exponential distribution using method of moment method					
λ=1/mean of the sample					
Thus, λ=	0.002702204				
Parameter estimation for gamma distribution using method of moment method(λ & n)					
λ = (mean of the sample)/(standard deviation of sample)^2					
λ =	0.00027746				
n = λ*mean of the sample					
n =	0.102679267				

Parameter estimation for gumbel distribution using method of moment method(a & b)				
a = ((standard deviation of sample)^2/1.645)^0.5				
a =	900.445834			
b = mean of the sample - 0.577*a				
b =	-149.4889129			

Parameter estimation for log normal distribution using method of moment method(μy & σy)				
σy = (Ln[1+(σx/μx)^2])^0.5 or,				
σy = (ln(1+(standard deviation of sample/mean of sample)^2)^0.5				
σy =	1.540742671			
μy = ln(μx)-σy^2/2				
μy = ln(mean of sample) - (standard deviation of sample)^2/2				
μy =	4.726743685			
Parameter estimation for log normal distribution using maximum likelihood method(μy & σy)				
μy = (Σln(xi))/n				
μy =	4.74403251			
σy = (standard deviation of lnxi * (N-1)/N)^0.5				
σy =	1.180601843			

NOTE:- Chi square and KS-test is performed on all the distribution in excel sheet(here only two are shown which passes both the chi-sq and KS test)

3. lognormal(μy=4.727 σy=1.541)							
Chi-Square test							
Range							
Low	High	Number of outcomes(n)	fs(xj)=n/N	Low normalised	High nor-malised	p(xj)=F(high norm.) - F(low norm.)	chi-squ. value
0	100	21	0.5	-∞	-0.079058932	0.468	0.091897436
100	200	13	0.3095238	-0.079058932	0.37074456	0.1763	4.228250163
200	300	2	0.047619	0.37074456	0.633862735	0.0926	0.917689149
300	400	2	0.047619	0.633862735	0.820548051	0.0569	0.063580233
400	∞	4	0.0952381	0.820548051	∞	0.2062	2.507889723
		N=42					7.809306705
∑chi-sqaure value=7.8093							
degree of freedom(ν) = m-1-q = 5-1-1 = 3							
where, m = number of intervals & q = number of parameters							
X^2(p=0.05,ν=3)=7.815, where p= significance level							
since, X^2(p=0.05,ν=3)>∑chi-sqaure value(X^2)>0,μy=4.727 σy=1.541 is accepted							
p(xj) is calculated using normal table							
KS test							
x	No of occurrence<x	Sn(x)	z=(ln(x)-μy)/σy	F(z)	d=|Sn(x)-F(z)|		

100	21	0.5	-0.079059	0.468	0.032		
200	34	0.80952381	0.3707446	0.6443	0.16522381		
300	36	0.85714286	0.6338627	0.7369	0.120242857		
400	38	0.9047619	0.8205481	0.7938	0.110961905		
KS,statistics=max(d)=0.16522							
Significance level(p)=0.05							
KS,reference(n=4,p=0.05)=0.624							
KS,statistics<KS,reference,μy=4.727 σy=1.541 is accepted							
F(z) is calculated using normal table							

Chi square and KS test on log normal parameter calculated from method of moments

4. lognormal(μy=4.744 σy=1.1806)							
Chi-Square test							
Range							
Low	High	Number of outcomes(n)	fs(xj)=n/N	Low normalised	High nor-malised	p(xj)=F(high norm.) - F(low norm.)	chi-squ. value
0	100	21	0.5	-∞	-0.117592592	0.4531	0.203892342
100	200	13	0.3095238	-0.117592592	0.46952174	0.2273	1.249237588
200	300	2	0.047619	0.46952174	0.812961608	0.1114	1.533720065
300	400	2	0.047619	0.812961608	1.056636072	0.0629	0.155919161
400	∞	4	0.0952381	1.056636072	∞	0.1453	0.724433317
		N=42					3.867202473
∑chi-sqaure value=3.867							
degree of freedom(ν) = m-1-q = 5-1-1 = 3							
where, m = number of intervals & q = number of parameters							
X^2(p=0.05,ν=3)=7.815, where p= significance level							
since, X^2(p=0.05,ν=3)>∑chi-sqaure value(X^2)>0,μy=4.744 σy=1.1806 is accepted							
p(xj) is calculated using normal table							
KS test							
x	No of occurrence<x	Sn(x)	z=(ln(x)-μy)/σy	F(z)	d=|Sn(x)-F(z)|		
100	21	0.5	-0.117593	0.4531	0.0469		
200	34	0.80952381	0.4695217	0.6804	0.12912381		
300	36	0.85714286	0.8129616	0.7918	0.065342857		
400	38	0.9047619	1.0566361	0.8547	0.050061905		
KS,statistics=max(d)=0.12912							
Significance level(p)=0.05							
KS,reference(n=4,p=0.05)=0.624							
KS,statistics<KS,reference,μy=4.744 σy=1.1806 is accepted							
F(z) is calculated using normal table							

Chi square and KS test on log normal parameter calculated from maximum likelihood

a. Lognormal parameters calculated using method of moments
μy=4.727 σy=1.541
AIC

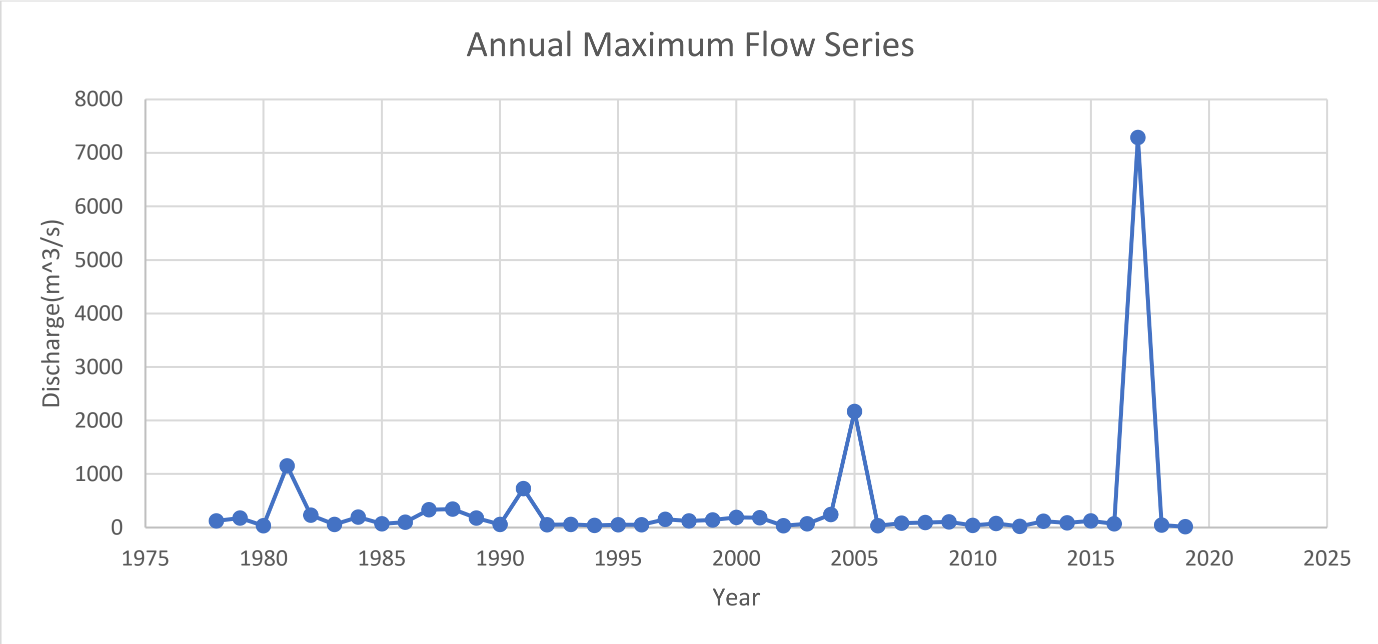
L=∏f(xi|μy,σy) = 2.9176 * 10^(-117)						
AIC = -2*ln(L) +2*q, where q=numbers of parameters in the model, for lognormal q=2						
AIC =	540.663389					
BIC = -2*ln(L) +ln(N)*q, where q=numbers of parameters in the model, for lognormal q=2, N(number of years)=42						
BIC =	544.1387283					
B. Lognormal parameters calculated using maximum likelihood estimation						
μy=4.744 σy=1.1806						
AIC						
L=∏f(xi|μy,σy) = 3.6058 * 10^(-116)						
AIC = -2*ln(L) +2*q, where q=numbers of parameters in the model, for lognormal q=2						
AIC =	535.6346543					
BIC = -2*ln(L) +ln(N)*q, where q=numbers of parameters in the model, for lognormal q=2, N(number of years)=42						
BIC =	539.1099935					
We know that smaller the AIC & BIC value the better the parameters						
Thus, the given discharge fits well with log normal distribution with parameters μy=4.744 σy=1.1806						

Thus, using AIC & BIC criteria the log normal fitting using maximum likelihood estimation is the best fitting

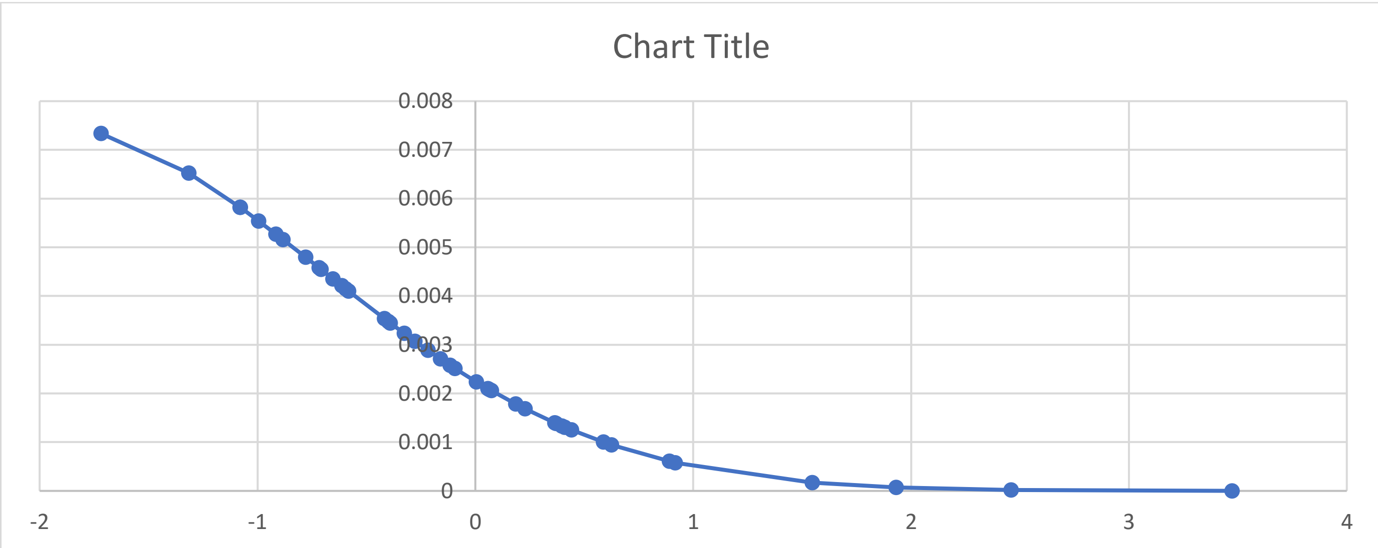
μy=4.744 σy=1.1806 are the parameters of the log normal.

Generally, Bridges are designed for 50 year return period (IRC 5: Clause 103)
T=50 => p=1/T=0.02
F(z) = 1-p =1-0.02 = 0.98
For F(z) = 0.98, we have to calculate z using standard normal distribution table
F(z) = 0.9798 when z = 2.05 , F(z) = 0.9803 when z = 2.06 ; thus using interpolation z = 2.054 for F(z) = 0.98
We know, z = (y-μy)/σy => y = z*σy+μy = 2.054*1.1806+4.744 = 7.16895
Thus, Design Discharge(Q) = e^(7.16895) = 1298.48 ~ 1300m^3/s

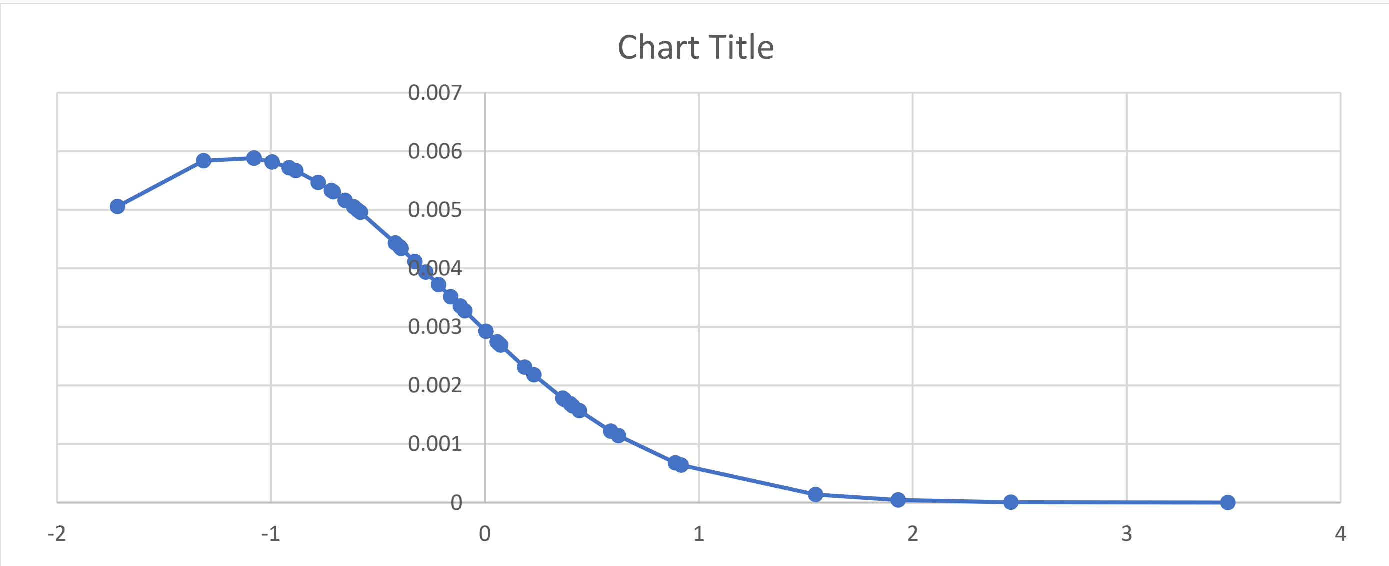
Thus , the Design Discharge is 1300 m^3/s.



ANNUAL MAXIMUM FLOW SERIES



LOG NORMAL FITTING WITH PARAMETER FROM METHOD OF MOMENTS



LOG NORMAL FITTING WITH PARAMETER FROM MAXIMUM LIKELIHOOD ESTIMATION

Q1 c,d,e

1. Bridge Design.

Design Discharge calculated = $1300\ m^3/s$

c) effective waterway length of the bridge = W (let)

Using Lacey's regime theory

$W = C\sqrt{Q}$ (IRC 5: Clause 104

where Q = design maximum discharge in m^3/s.

C = a constant taken as 4.8 for regime channels

$W = 4.8 \times (1300)^{0.5} = 173.066\ m$

$W \simeq 175m$.

d) Mean sediment diameter = 0.14mm (d_m)

$K_{sf} = 1.76\sqrt{d_m}$. (IRC: SP 13 – Article 7

$= 1.76\sqrt{0.14}$

$= 0.658$

Discharge for scour estimation is greater than the design discharge. depending upon the catchment area.

As per IRC and RDSO

if the catchment area is between 5000 – 25000 sq. km ~~the~~ discharge is increased by (10-20)% of the design discharge.

The catchment area for Pannaiyan basin is $15679\ km^2$ which lies between (5000 – 25000) sq km

Thus, increasing design discharge ~~to~~ by 15%.

$Q' = 1.15 \times 1300$

$= 1495\ m^3/s$

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$\because$ effective waterway length (L) = Lacey's P

Normal scour depth, $D = 0.473 \left(\frac{Q}{K_{sf}}\right)^{1/3}$

$$D = 0.473 \times \left(\frac{1495}{0.658}\right)^{1/3}$$

$$= 6.22 m.$$

Now, maximum scour depth for piers = 2.0D (IRC:78 Clause 703.3

" " " for abutments = 1.27 D

Thus, maximum scour depth (D_{max}),

$$D_{max, piers} = 2D = 2 \times 6.22 = 12.44 m$$

$$D_{max, abutments} = 1.27 D = 1.27 \times 6.22 = 7.90 m.$$

e) For discharge between [300, 3000] m^3/s, minimum vertical clearance is 1200mm IRC:5 Clause 106.2.

$\because$ design discharge is 1300 m^3/s lying between [300, 3000] m^3/s vertical clearance to be provided is 1200mm.

According to IRC:5 Clause 107,

Freeboard should be more than 1.75 m.

Thus Freeboard provided = 1.75m.

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Q2 a

a. Cumulative of inflow rate (cumecs) => 194138.3672
Maximum constant inflow (cumecs) = (Cumulative of inflow rate/Number of inflow observations) = (194138.3672/14586) = 13.30991136
Maximum monthly demand (m^3) = Maximum constant inflow (m^3/s) * (86400s/1day) * (30day) = 34499290m^3

the *maximum monthly demand* (in cubic meter) that can be met from the water available at the site is **34499290m^3**

b.

b. Live storage capacity is calculated using the sequent peak algorithm
Live storage capacity (m^3) = 7423838922

the *live storage capacity* of the reservoir to meet the maximum monthly demand is **7423838922m^3**

c.

c. We have to calculate dead storage capacity assuming design life of reservoir is 100 years
Capacity of reservoir (C) = 7423838922 m^3
Average inflow rate (I)= 13.30991136 m^3/s * (86400s/1day) * (365days/1year) = 419741365 m^3/year
C/I = 7423838922/419741365 = 17.6866984 years
Trap efficiency (using Brune's Curve) = (C/I)/(0.012+1.02*(C/I))*100 = 97.97%
Sediment load in the reservoir is = 167.46 mg/L
Accumulated sediment in 100 years = 0.9797 * 167.46 mg/L * (1kg/1000000mg) * 419741365*1000 L/year * 100 years = 6886300423 kg
Assume that the reservoir normally has a moderate drawdown,
W1 = 1490 , B1 = 0 ; W2 = 1185 , B2 = 43.3 ; W3 = 737 , B3 = 171.4
Where W1 , W2 , W3 are in Kg/m^3
$WT = Ps/100 * (W1 + B1 \log T) + Pm/100 * (W2 + B2 \log T) + Pc/100 * (W3 + B3 \log T)$
Where, WT is density of sediment deposit after T years
Given, Sand (%) = 56.08 ; Silt (%) = 32.94 ; Clay (%) = 10.98
Using the formula W(100) ~ 1373 Kg/m^3
Where W(100) is density of sediment deposit after 100 years
Thus the Dead Storage Capacity = (Accumulated sediment in 100 years/W(100)) = (6886300423/1373) ~ 5015515 m^3

the required *dead storage capacity* for the given sediment load characteristics is **5015515 m^3**

Q 3 a.

3. Spillway and Stilling Basin Design.

01 we have to calculate discharge for 100 year return period flood

$T = 100$ years

$p = \frac{1}{T} = 0.01$

$F(z) = 1 - p = 0.99.$

For $F(z) = 0.99$, we have to calculate z using standard normal distribution table.

$F(z) = 0.9898$ when $z = 2.32$

$F(z) = 0.9901$ when $z = 2.33.$

$F(z) = 0.99 \Rightarrow z = 2.326.$ (Using interpolation)

we know $z = \frac{y - \mu_y}{\sigma_y}$

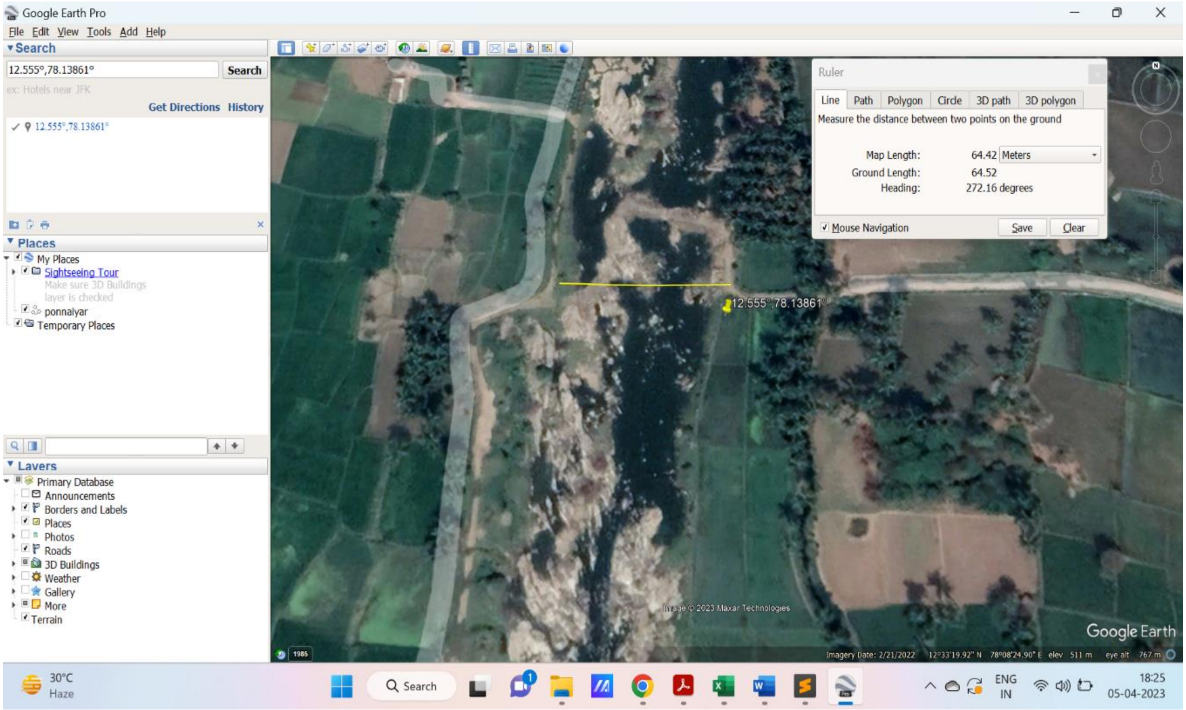
The best fit is log normal with $\mu_y = 4.744$, $\sigma_y = 1.1806.$

$$2.326 = \frac{y - 4.744}{1.1606}$$

$$y = 7.49$$

Thus, Design Discharge $(Q) = e^{y} = e^{7.49} \simeq 1790\ m^3/s$

b.



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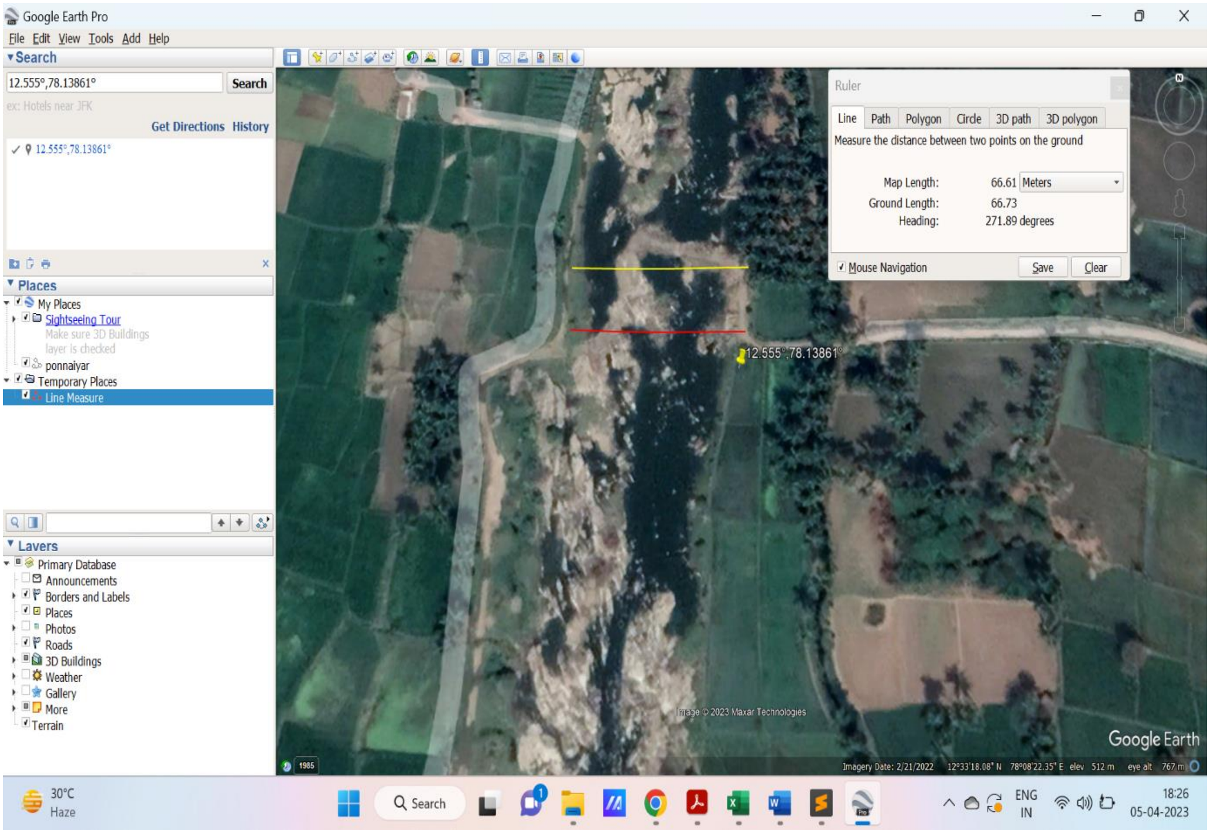

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Measure the distance between two points on the ground
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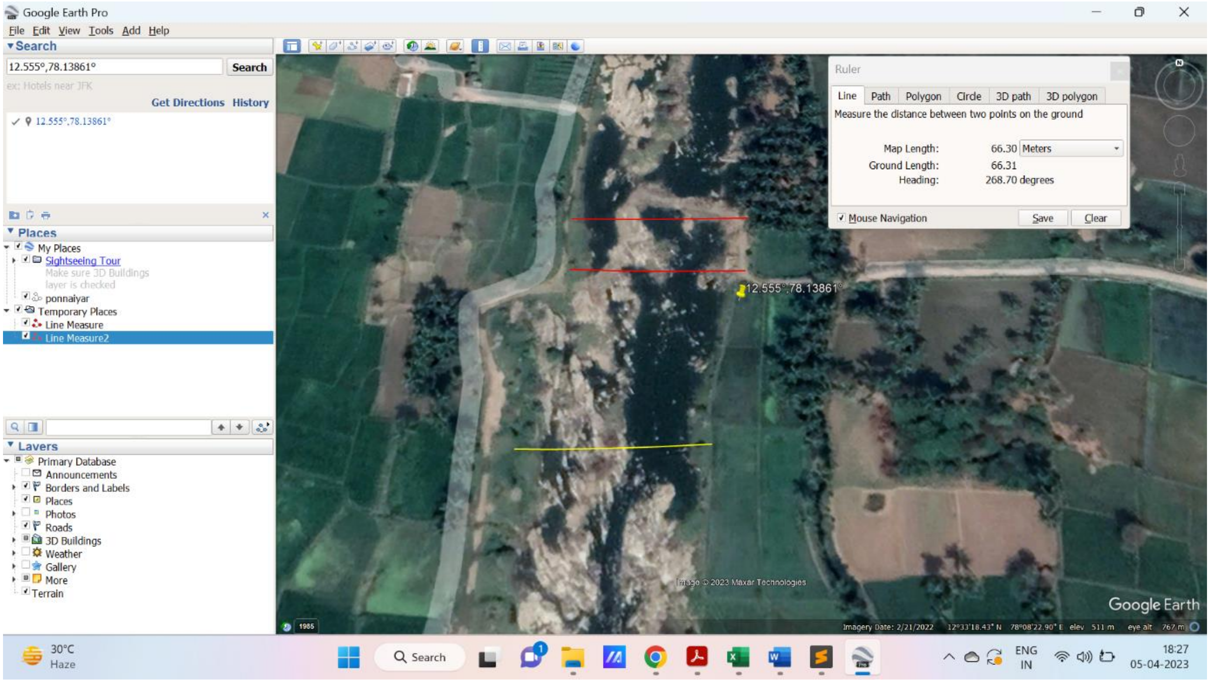
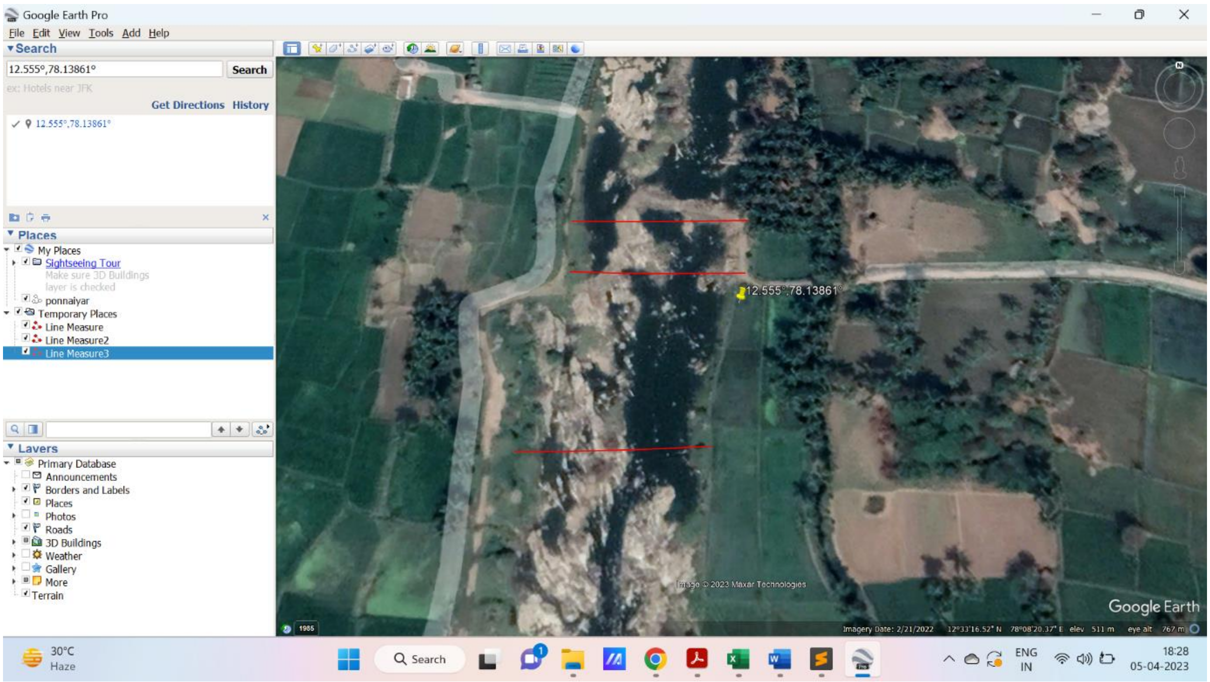




Using Google Earth the current width of the river is found out to be 66m

b) Maximum allowable water depth in the reservoir from reservoir bottom behind the spillway = 57.5 m.

Slope (n) of the straight channel downstream of the spillway (nV : 1H) = 2.5.

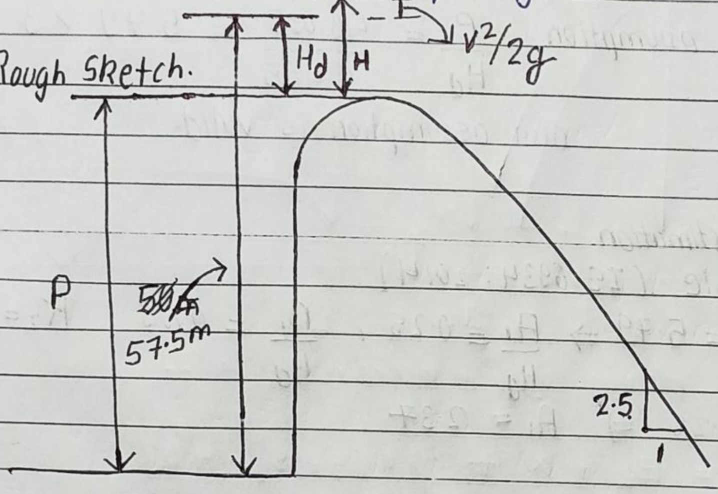
Existing river width ≃ 66 m.

Thus, Effective crest length = 66/2 = 33 m.

Design Discharge = 1790 m^3/s.

Also, upstream face of the spillway is vertical.

Rough Sketch.



$$Q = 1790 \ m^3/s$$

$$L_e = 33 m.$$

Assuming $P/H_d > 3$

and since $C = f(P/H_d)$

$C = 0.74$ for $P/H_d > 3$

we have.

$$Q = \frac{2}{3}\sqrt{2g}\ C\ L_e\ H^{3/2}$$

$$1790 = \frac{2}{3}(2 \times 9.81)^{0.5} \times 0.74 \times 33 \times H^{3/2}.$$

$$H = 8.51 m.$$

$$V = \frac{Q}{L_e \times 57.5} = \frac{1790}{33 \times 57.5} = 0.943\,m/s$$

$$\frac{V^2}{2g} = \frac{(0.943)^2}{2 \times 9.81} = 0.045\,m.$$

$$H = H_d + \frac{V^2}{2g} \Rightarrow 8.51 = H_d + 0.045$$

$$H_d = 8.465\,m.$$

$$P = 57.5 - H_d = 57.5 - 8.465$$

$$P = 49.035\,m.$$

Checking the assumption $\frac{P}{H_d} = \frac{49.035}{8.465} = 5.79 > 3$

our assumption is valid.

Now, profile estimation.

Using Table (IS 6934 : 2014)

When $\frac{P}{H_d} = 5.79 \Rightarrow \frac{A_1}{H_d} = 0.28$, $\frac{B_1}{H_d} = 0.165$, $K_2 = 2.0$

$$\frac{A_1}{8.465} = 0.28 \Rightarrow A_1 = 2.37$$

$$\frac{B_1}{8.465} = 0.165 \Rightarrow B_1 = 1.40$$

(a) Thus, upstream quadrant profile.

$$\frac{X_1^2}{A_1^2} + \frac{Y_1^2}{B_1^2} = 1 \Rightarrow \frac{X_1^2}{(2.37)^2} + \frac{Y_1^2}{(1.40)^2} = 1$$

(b) Downstream quadrant profile

$$X_2^{1.85} = K_2 H_d^{0.85} Y_2.$$

$$K_2 = 2$$

$$X_2^{1.85} = \cancel{K_2}\, 2\, H_d^{0.85} Y_2.$$

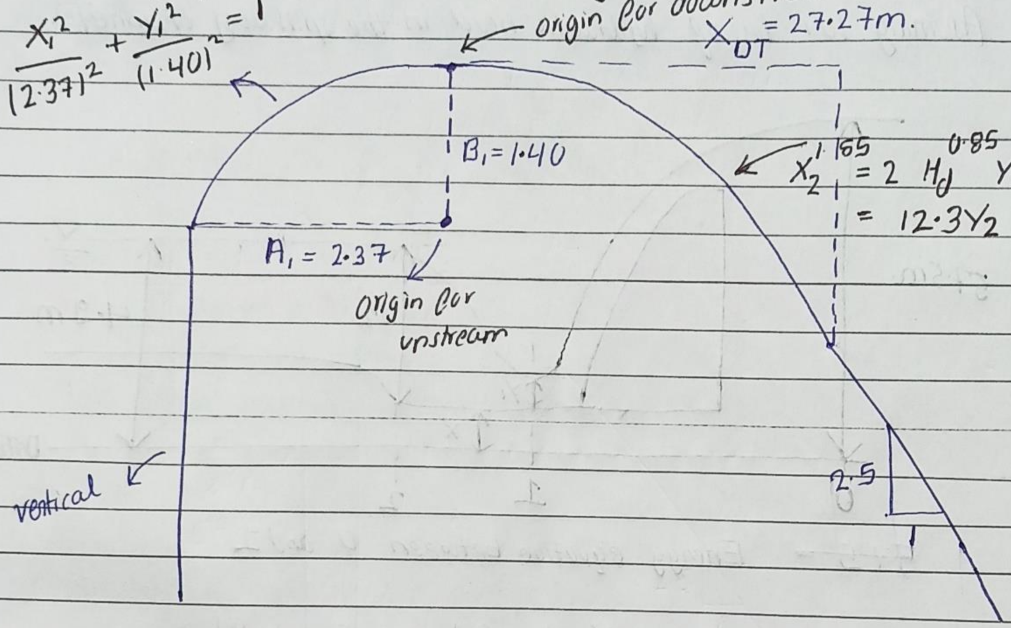
Also, $X_{DT} = 0.485\ H_d\ (K_2 \alpha)^{1/0.85}$

where $K_2 = 2 \quad \alpha = 2.5 \quad (\because n = 2.5)$

$X_{DT} = 0.485 \times 8.465 \times (2 \times 2.5)^{1/0.85}$

$= 27.27\ m.$

Thus, profile view of the spillway.



c.

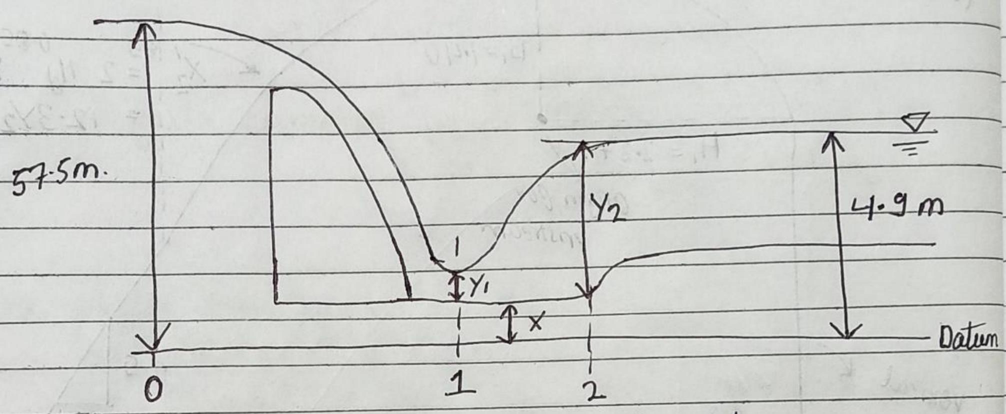
c) Dischange = 1790 m^3/s.

Assuming width of the spillway is same as effective crest length, thus b = 33m.

Water surface elevation on the tailwater = 4.9m (given

Water surface elevation in the reservoir = 57.5m

Assuming 10% loss of hydraulic head in the spillway channel.



~~57.5 =~~ Energy equation between 0 and 1

$$57.5 = E_1 + X + 0.1(57.5 - 4.9)$$

$$E_1 = 57.5 - X - 5.26$$

$$E_1 = 52.24 - X$$

$$Y_1 + \frac{Q^2}{2g\,b^2\,Y_1^2} = 52.24 - X \qquad -(1)$$

Momentum equation

$$\frac{Y_2}{Y_1} = \frac{1}{2}\left[\sqrt{1+8F_r^2} - 1\right]$$

$$\frac{4.9 - X}{Y_1} = \frac{1}{2}\left[\sqrt{1 + 8\frac{Q^2}{b^2 g Y_1^3}} - 1\right] \qquad -(2)$$

$$Y_1 + \frac{1790^2}{2\times 9.81\times 33^2\times Y_1^2} = 52.24 - X$$

$$Y_1 + \frac{150}{Y_1^2} = 52.24 - X. \qquad — 3$$

$$\frac{4.9 - X}{Y_1} = \frac{1}{2}\left[\sqrt{1 + \frac{8\times 1790^2}{33^2\times 9.81\times Y_1^3}} - 1\right]$$

$$\frac{4.9 - X}{Y_1} = \frac{1}{2}\left[\sqrt{1 + \frac{\cancel{239}\ 2399.38}{Y_1^3}} - 1\right] \qquad — 4.$$

~~$Y_1 = 2.325m.$~~

~~$X = 22166m.$~~

The above equation are not giving any proper solution for. X and Y_1

If it could be solved.

then we will have to calculate the velocity and froude number to decide the stilling basin.

and after that different characteristics of stilling basin could be determined.

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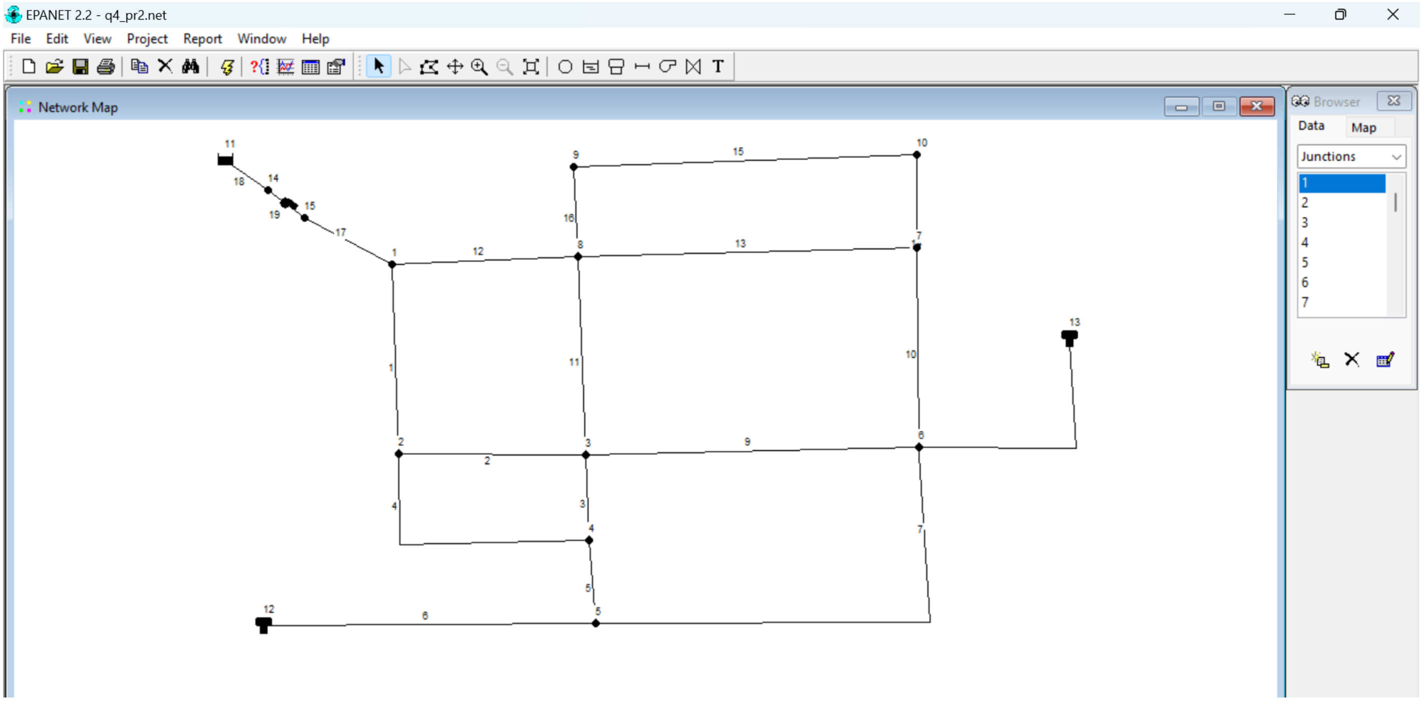
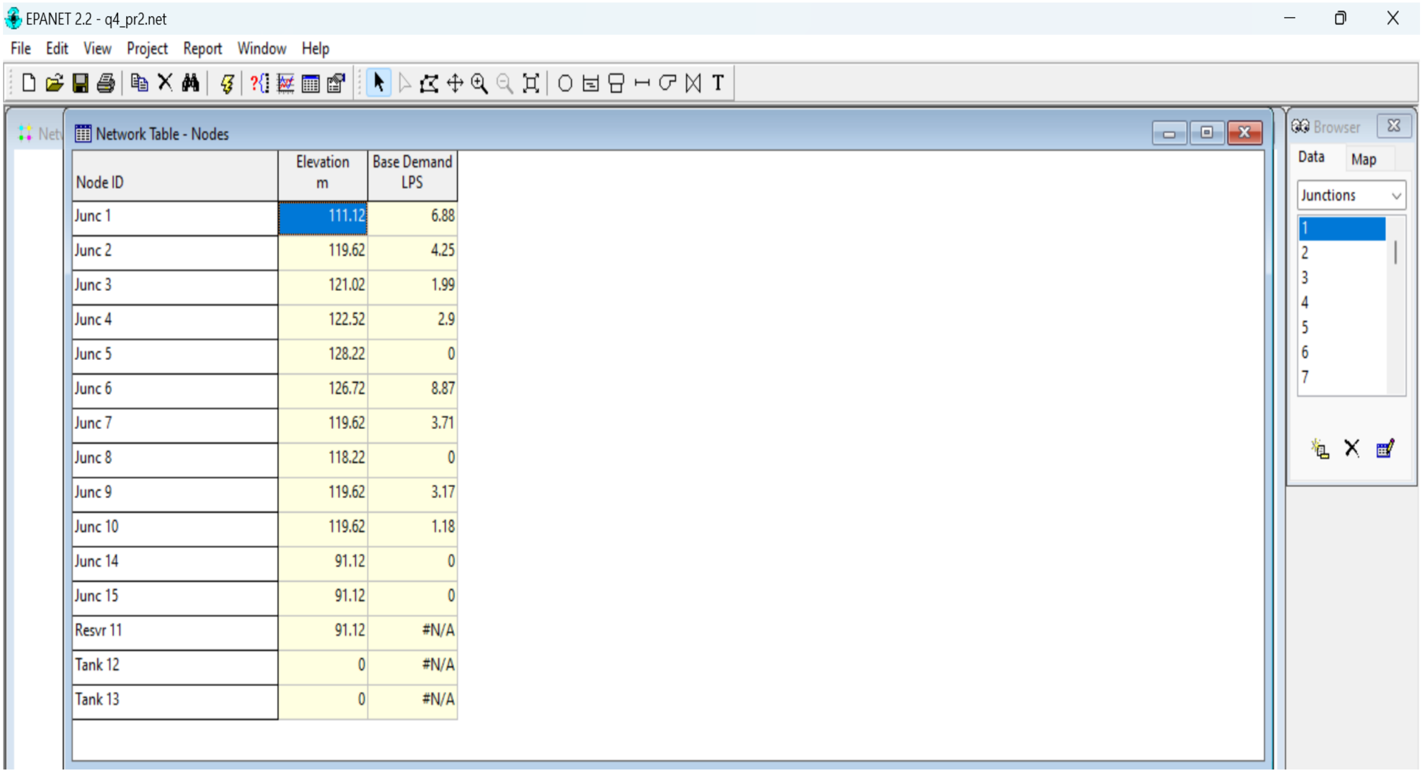
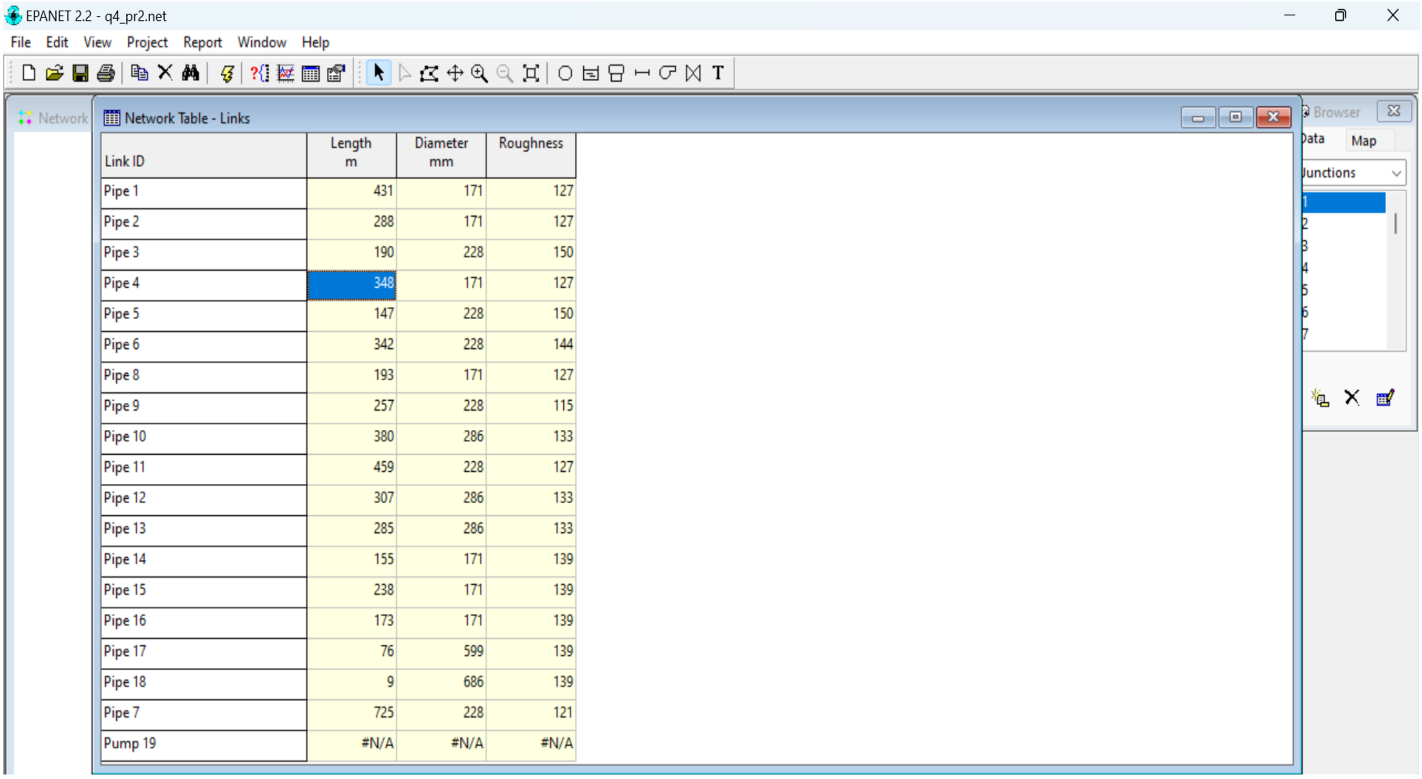
Q 4



Network



Node ID	Elevation m	Base Demand LPS
Junc 1	111.12	6.88
Junc 2	119.62	4.25
Junc 3	121.02	1.99
Junc 4	122.52	2.9
Junc 5	128.22	0
Junc 6	126.72	8.87
Junc 7	119.62	3.71
Junc 8	118.22	0
Junc 9	119.62	3.17
Junc 10	119.62	1.18
Junc 14	91.12	0
Junc 15	91.12	0
Resvr 11	91.12	#N/A
Tank 12	0	#N/A
Tank 13	0	#N/A

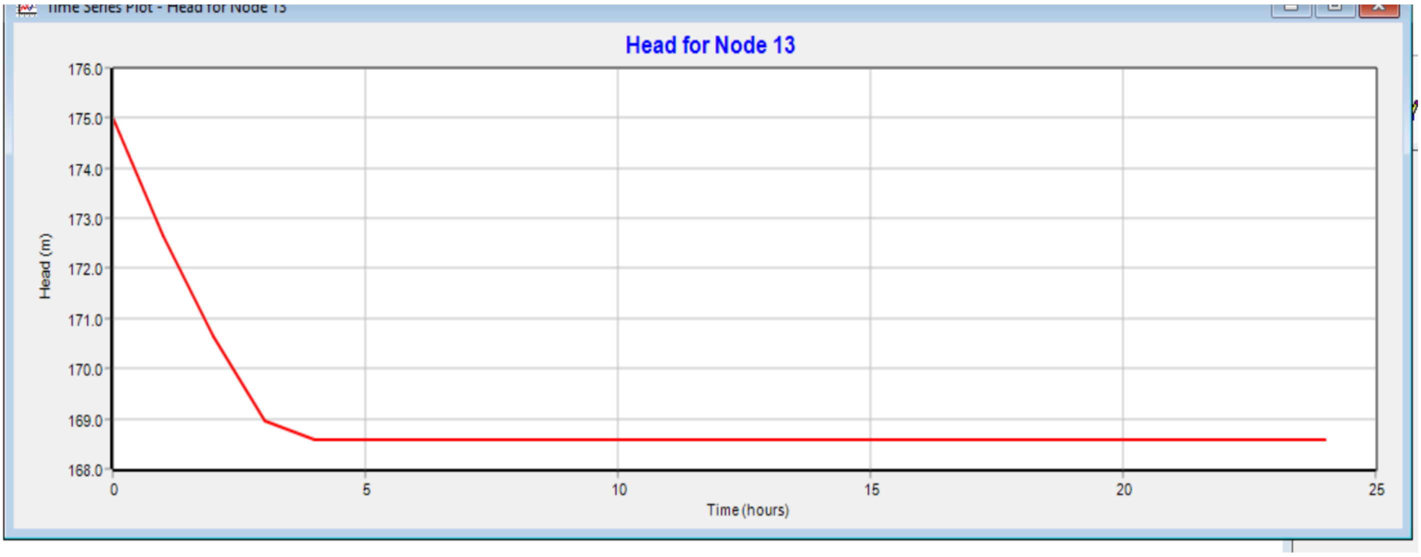


Link ID	Length m	Diameter mm	Roughness
Pipe 1	431	171	127
Pipe 2	288	171	127
Pipe 3	190	228	150
Pipe 4	348	171	127
Pipe 5	147	228	150
Pipe 6	342	228	144
Pipe 8	193	171	127
Pipe 9	257	228	115
Pipe 10	380	286	133
Pipe 11	459	228	127
Pipe 12	307	286	133
Pipe 13	285	286	133
Pipe 14	155	171	139
Pipe 15	238	171	139
Pipe 16	173	171	139
Pipe 17	76	599	139
Pipe 18	9	686	139
Pipe 7	725	228	121
Pump 19	#N/A	#N/A	#N/A

Data of network and nodes

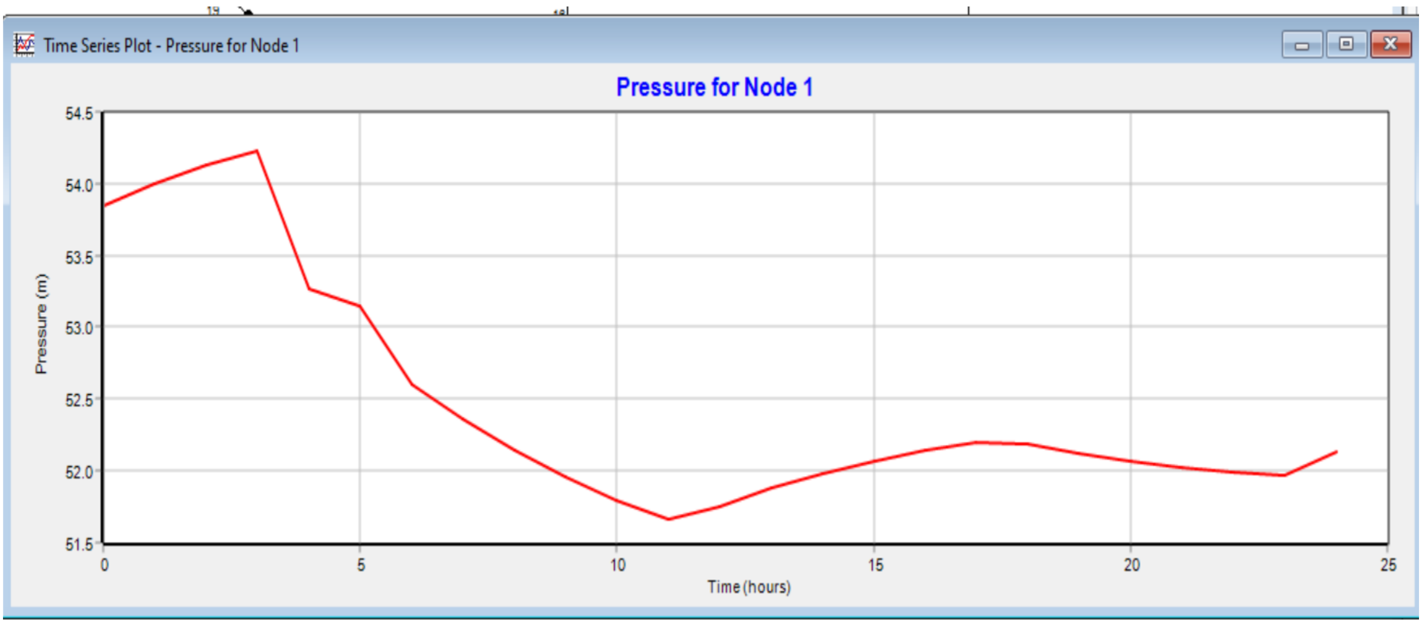
a.

Hydraulic grid line for tank 1

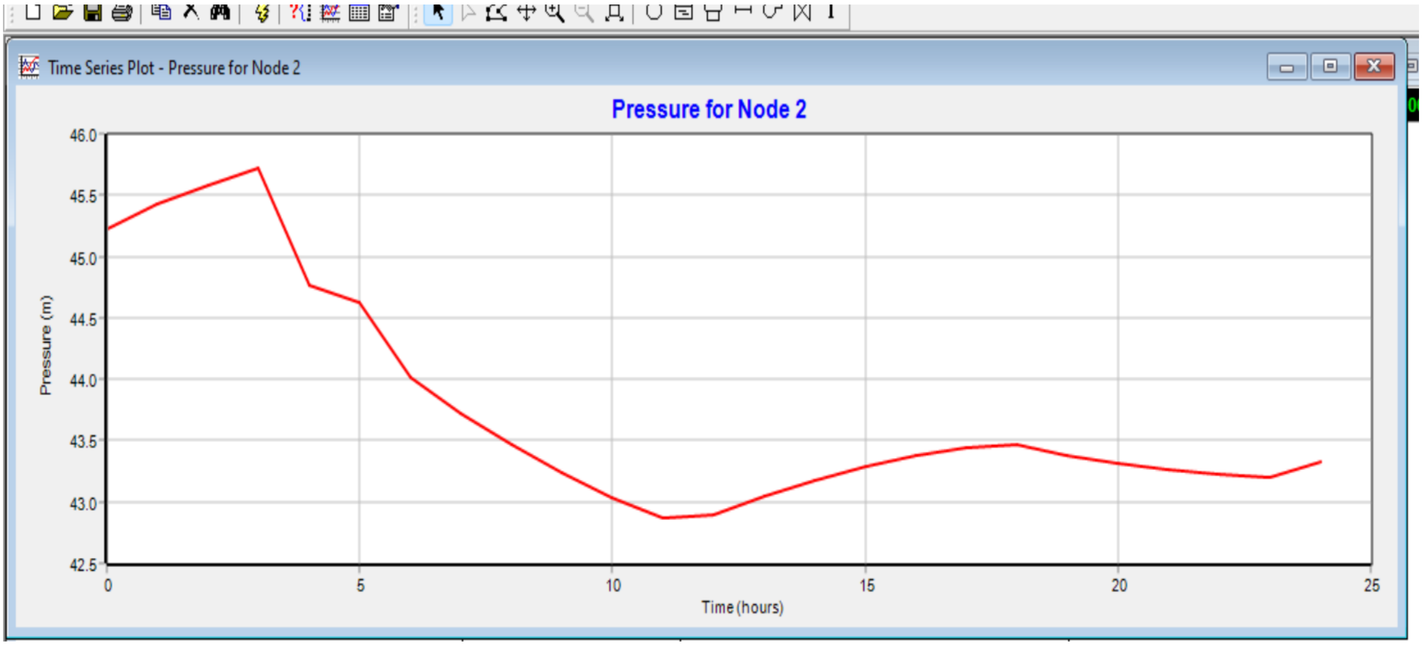


Hydraulic grid line for tank 2

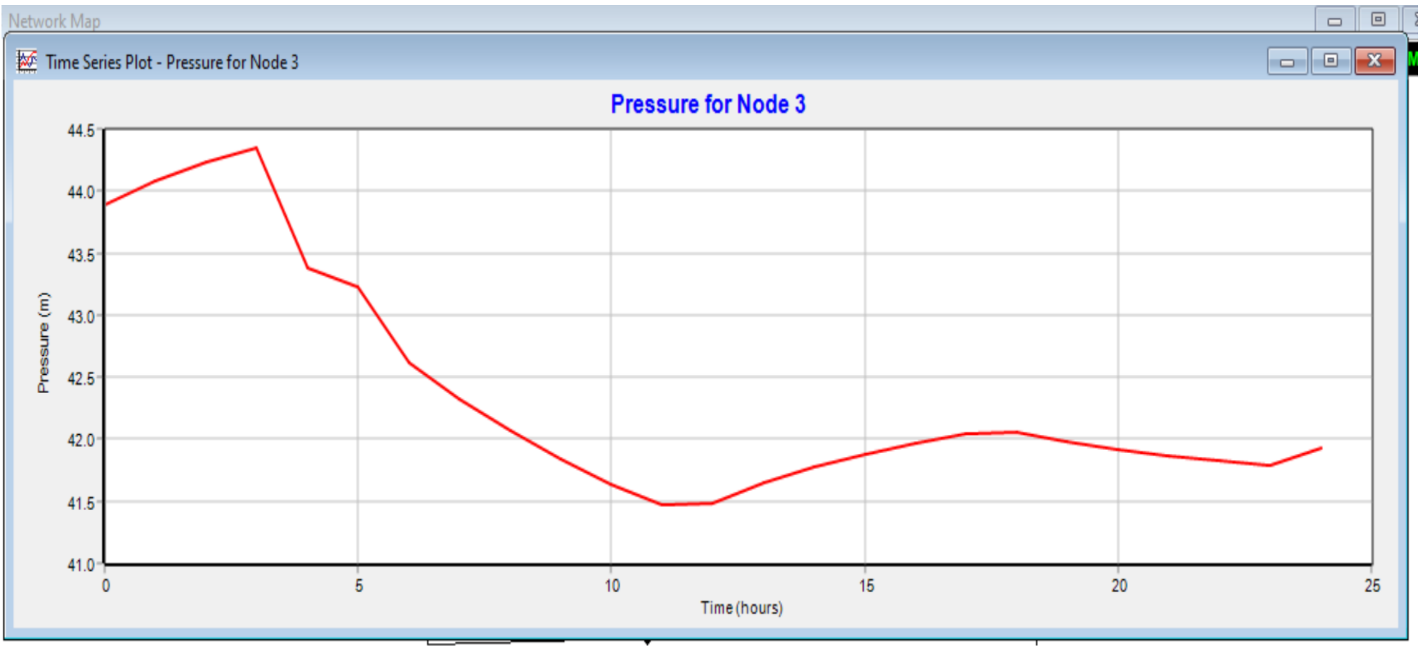
b.



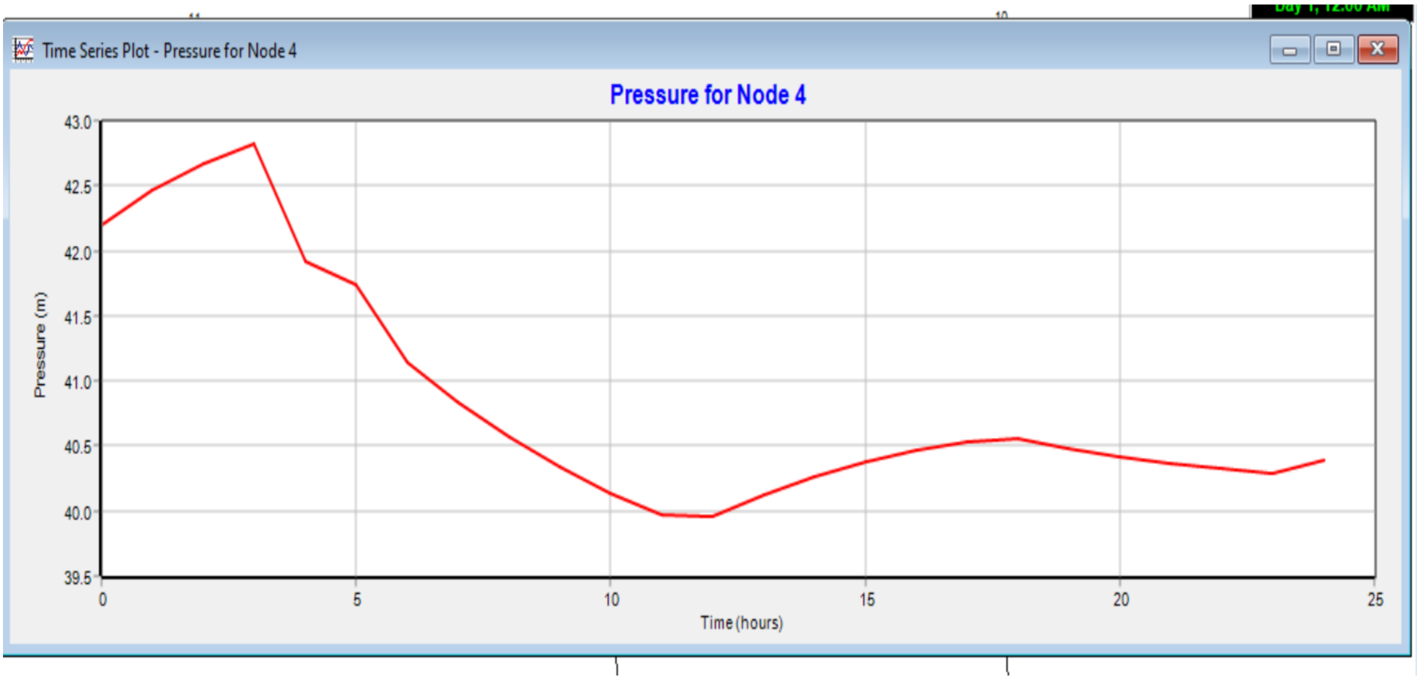
time-series plots of pressure for node 1



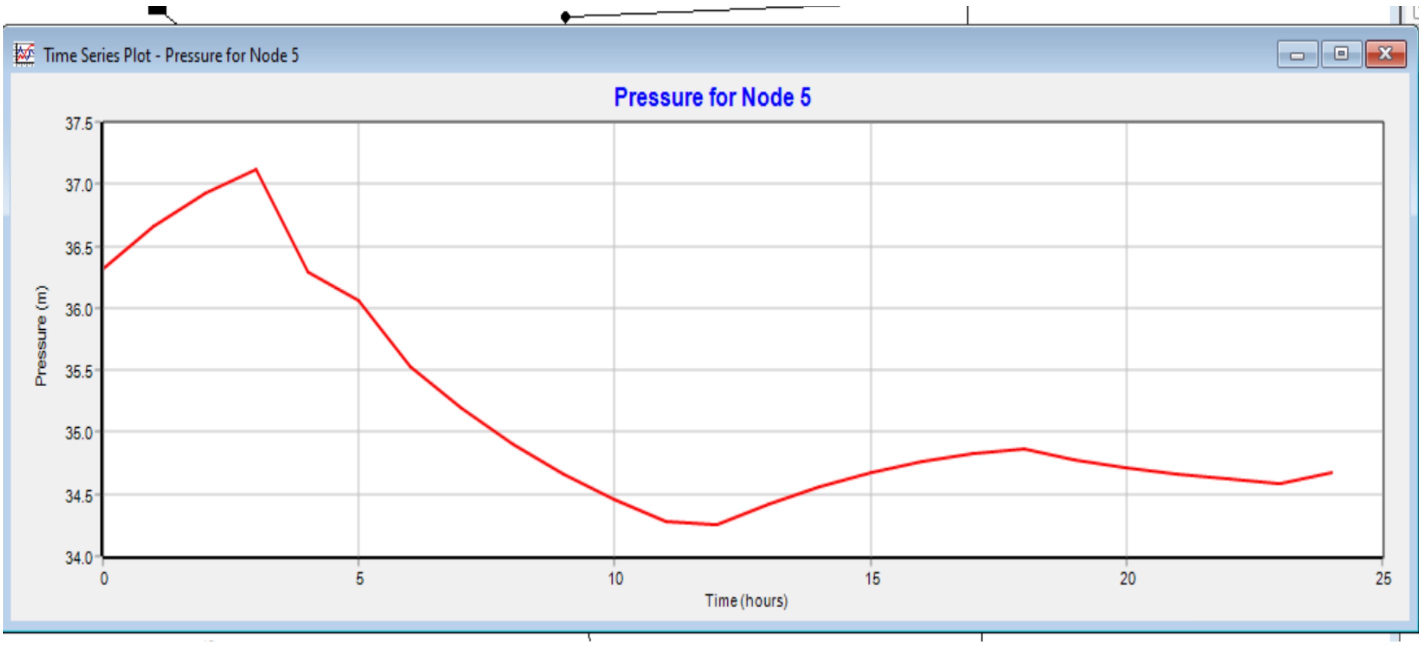
time-series plots of pressure for node 2



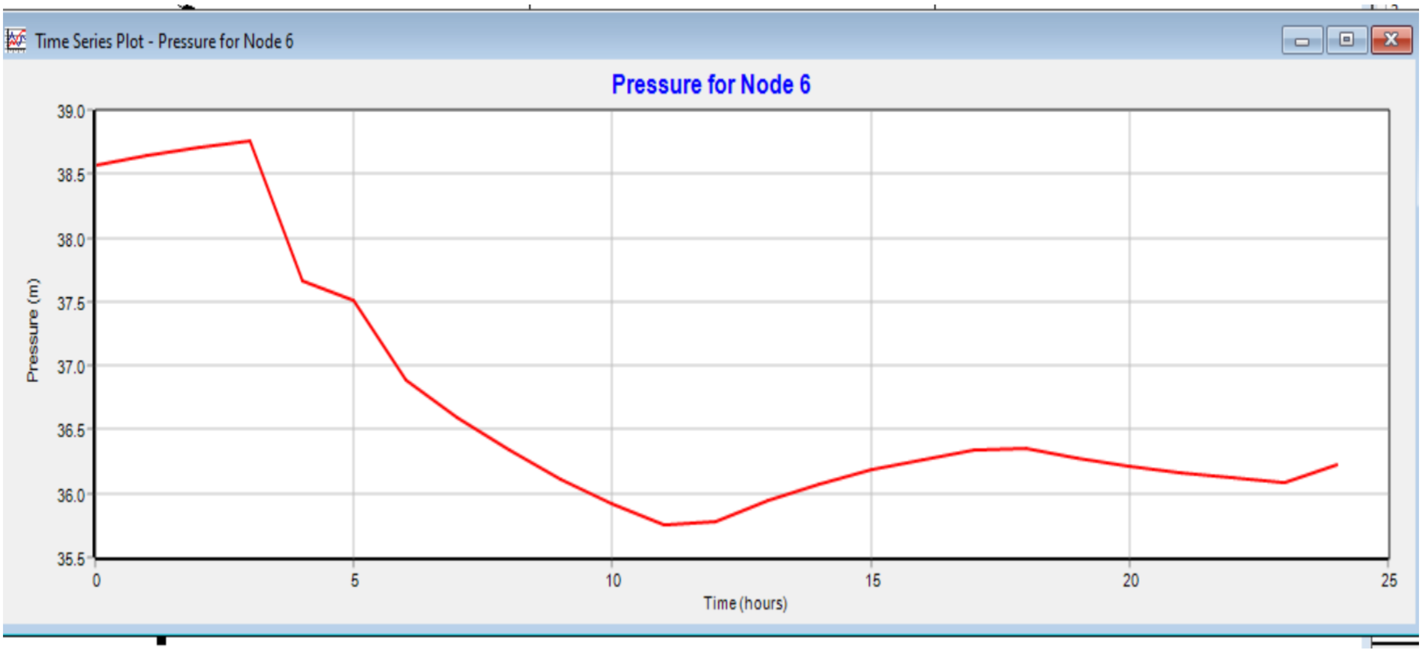
time-series plots of pressure for node 3



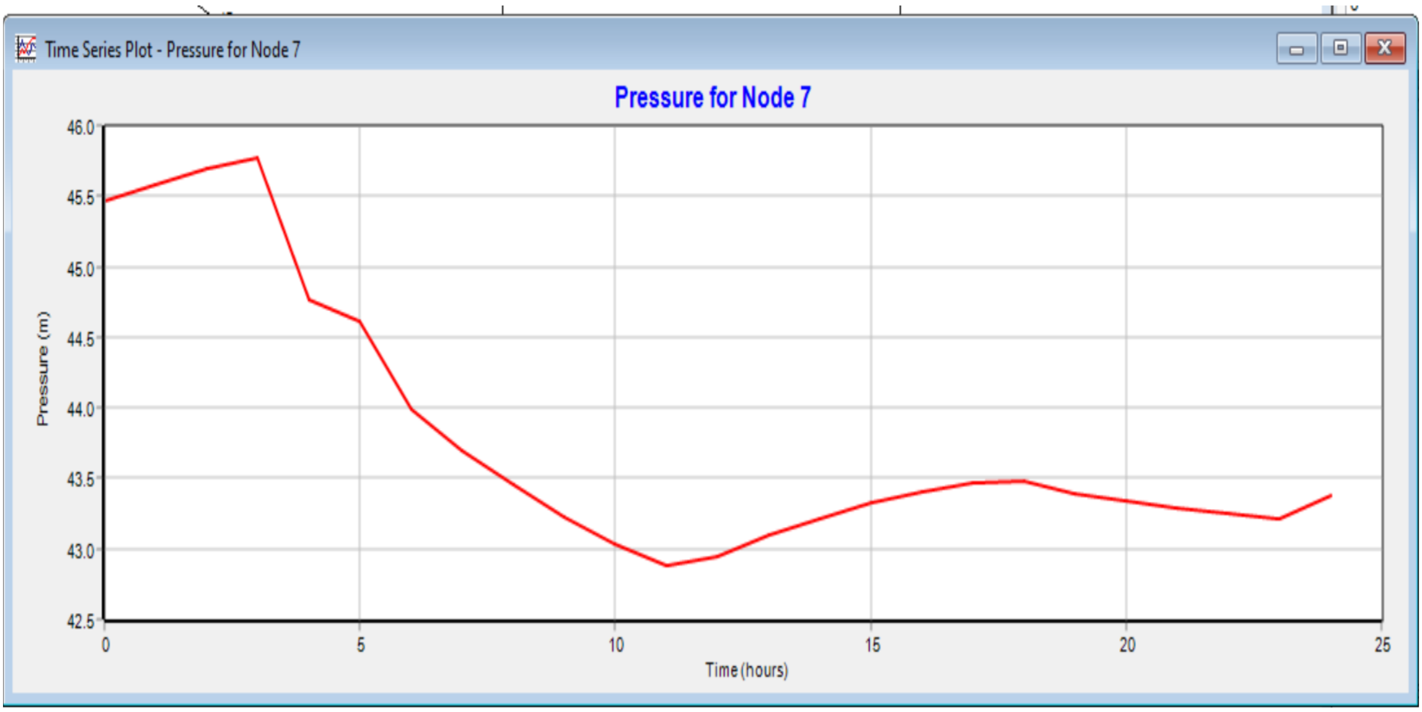
time-series plots of pressure for node 4



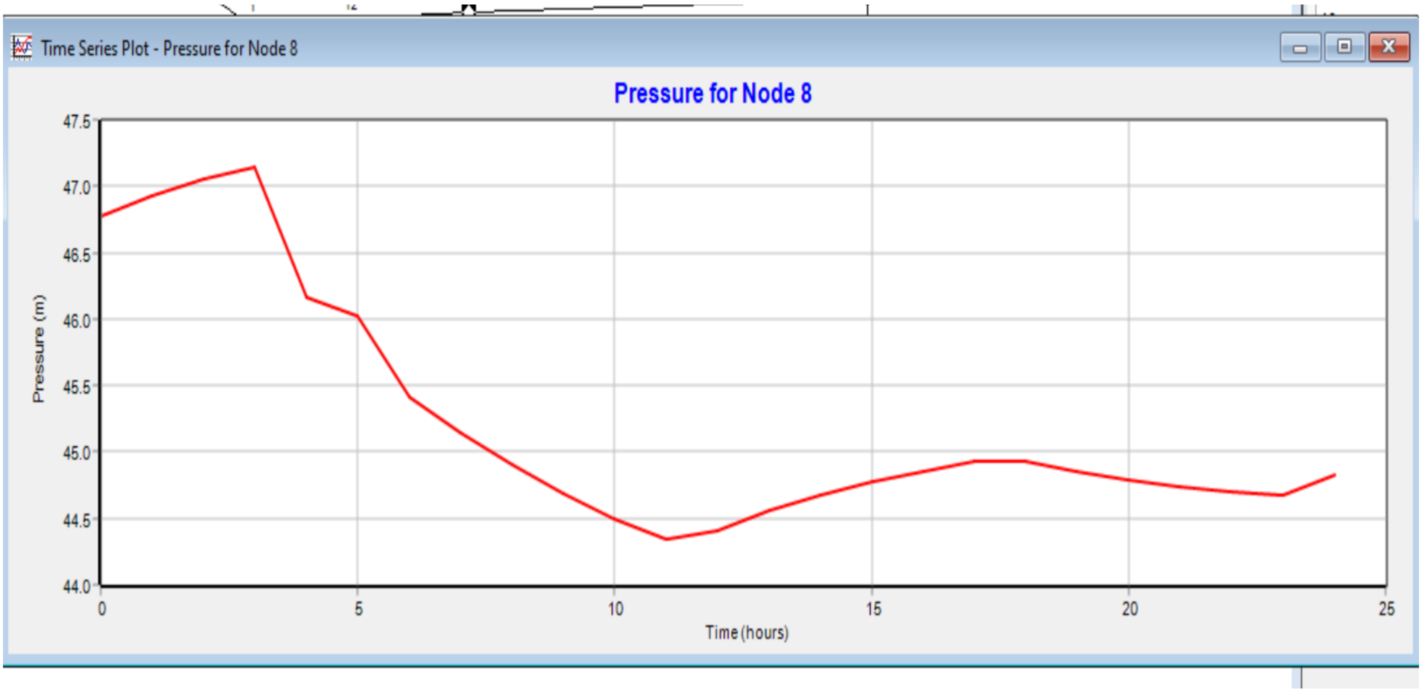
time-series plots of pressure for node 5



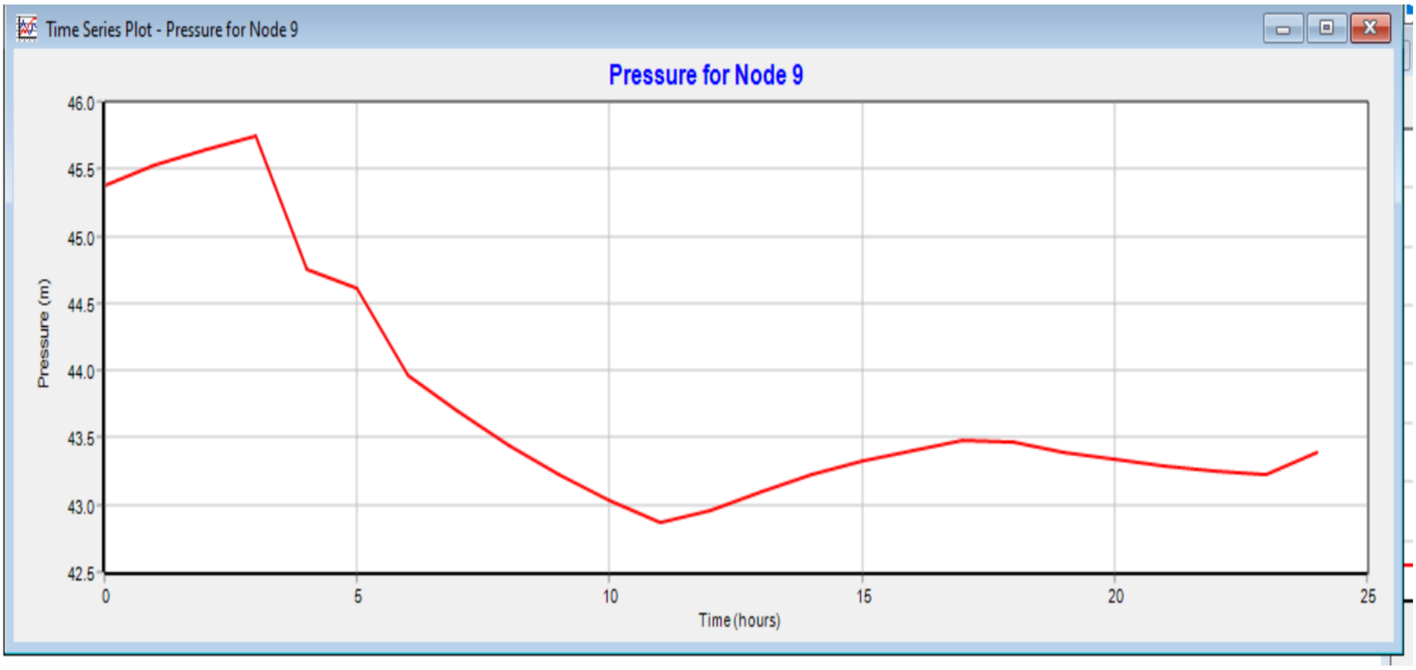
time-series plots of pressure for node 6



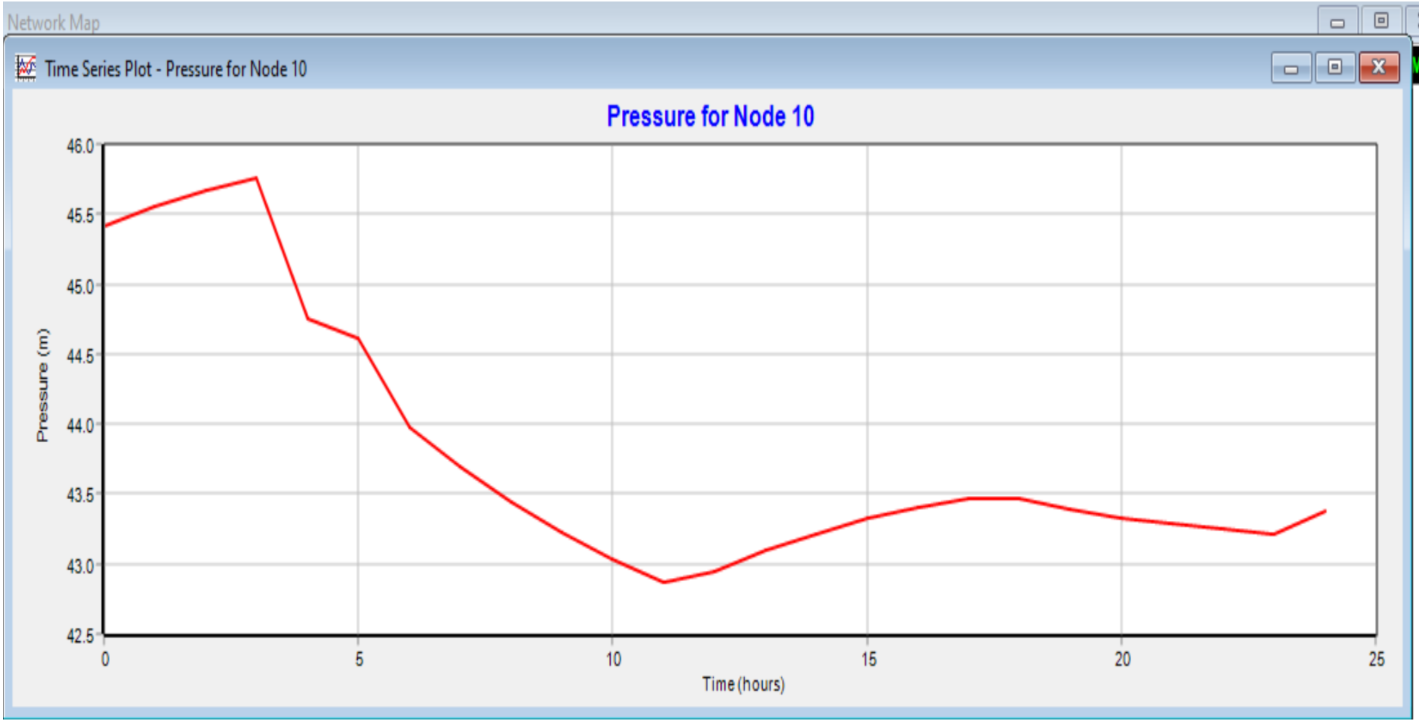
time-series plots of pressure for node 7



time-series plots of pressure for node 8

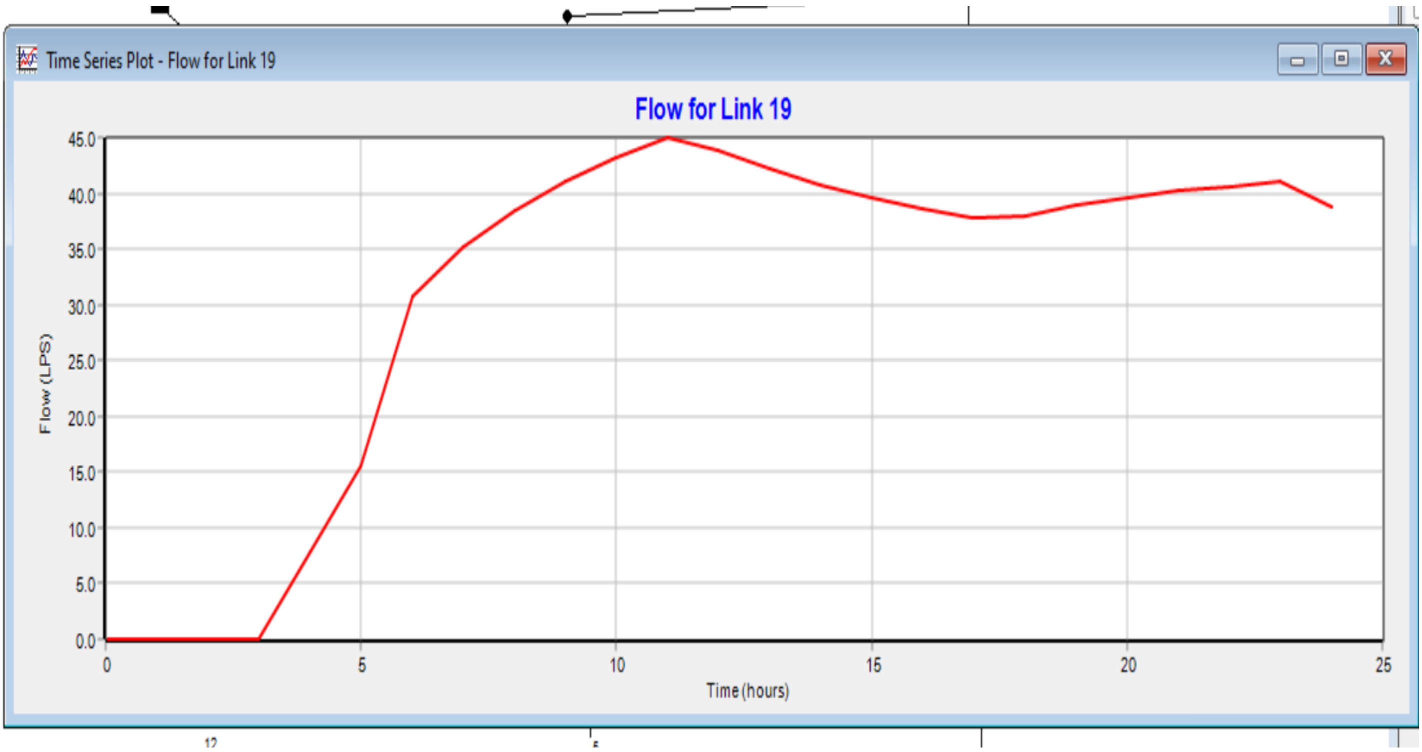


time-series plots of pressure for node 9



time-series plots of pressure for node 10

c.



Discharge from the pump as a function of time.