Analysis and Design of a Siphon Aqueduct

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***Abstract* - An aqueduct structure is a complex structure as compared to bridge, as it takes canal water across stream and canal traffic over the trough. The water tightness and free expansions - contractions of trough, canal water load as well as traffic load on the trough involves complex load combinations, for which the superstructure and substructure of it is required to be planned and designed. The object of this research is to develop an optimized hydraulic design, by considering various theories which are applicable. In this research we are considering an aqueduct which is located in Puligadda and it is designed as a syphon aqueduct for reducing the damages during floods and to avoid flood intensity across the delta areas which provide irrigation water to fertile soil in Diviseema. For this design the analysis is done by using STAAD.Pro software (trough side walls).**

***Keywords* - Aqueduct, trough, total energy line(TEL),head loss(HL)**

I. INTRODUCTION

We are considering puligadda Aqueduct near Avanigada in Krishna dt, Andhra Pradesh.Aqueduct cum-Road bridge constructed in 1934 near puligada.
Government of Madras member for Irrigation and Revenue Sir.A.Y.G Campbell laid the foundation stone for the Aqueduct cum-Bridge in 1934 and cannon Dunkerley and Co-Limited completed it.
In syphon Aqueduct, the HFL of the drain is much higher above the canal bed, and water runs under siphonic action through the Aqueduct barrels.

Fig(1). Siphon Aqueduct

II. LITERATURE REVIEW

- In the quotation given below, Vitruvius - a Roman architect from the 1 c BC who left a treatise called 'de architecture' - makes reference to the Venter (the lowest part of the siphon) and the geniculus (vertical bend), but he also introduced the term ‘ colliviaria' a term which has caused much discussion (see Hodge1983 and Kessener2001). One of its function could have been to let the air-bubbles escape from the pipes.
- Frontinus is another classical author who wrote about aqueducts. Around 100 AD he was supervising the aqueducts of Rome and wrote the book 'de aqua duct'. Remarkably he did not make any reference to siphons although in his time they were already applied all over the Greek and Roman world and even in and around
- A.T. Hodge - Hodge1983: Siphons in Roman aqueducts (in: PBSR Vol 51 (1983) pag 174)
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- H.P.M. Kessener -Kessener2001: Vitruvius and the conveyance of water (in: Bulletin Antieke beschaving vol 76 (2001) pag 139 ff

III. Experimental Methodology

Collection of data required for the design of Puligadda Syphon Aqueduct .

1. Discharge of river and canal .
2. HFL &BL of drain and FSL of canal.
3. Traffic volume of road.
4. Width of the river canal.

Design of Puligadda Aqueduct as Puligadda Syphon Aqueduct

1.Design of drainage water way

2. Design of canal waterway
3. Design of contraction transition
4. Design of expansion transition
5. Design of the trough

IV. Characteristics of flows

Uniform flow in canal and aqueduct trough (dy/dx = 0, dV/dx = 0, dQ/dx = 0) Where y is a depth, V is the velocity of flow and Q is the discharge. Non-uniform flow in upstream and downstream transition zone of aqueduct . (dy/dx ≠ 0, dV/dx ≠ 0, dQ/dx = 0). Steady flow for canal and aqueduct (dQ / dx = 0). For Steady non-uniform flow, that is, GVF in upstream transition zone,

$$\text{Energy losses} = HL_1 = K1\,(V_2^2 - V_1^2) / 2g \,. \qquad \text{-------- Eq(1)}$$

Where K1 ranges from 0.1 to 0.3; and 0.2 is considered for aqueduct design.

For Steady non-uniform flow, that is, GVF in downstream transition zone,

$$\text{Energy losses} = HL_3 = K_2 (V_2^2 - V_3^2) / 2g \qquad \text{----------Eq(2)}$$

Where K2 ranges from 0.2 to 0.4; and 0.3 is considered for aqueduct design.

Steady-uniform flow in aqueduct trough, energy losses shall be.

$$\text{Energy losses} = HL_2 = n^2 V_2^2 L / R^{4/3} \qquad \text{-----------Eq(3)}$$

Where n is Manning's coefficient & R = hydraulic mean depth.

1.Continuity equation :

$$Q1 = Q2 = Q3 \qquad \text{----------Eq(4)}$$

2.Energy equation (Bernoulli's equation) :

$$E = Z1 + y1 + V1^2/2g = Z2 + y2 + V2^2/2g + \text{Losses} = Z3 + y3 + V3^2 / 2g + \text{Losses} \qquad \text{-----------Eq(5)}$$

Where E denotes total energy, Z denotes potential head,
y denotes depth and V denotes velocity

V. DESIGN OF VENT WAY

Designed discharge of old aqueduct = 4500 m^3 /sec
Maximum flood observed in 1977 = 6000m^3/sec
We are designing for discharge = 7050m^3/sec
Bed Level of the drain = 10 m
Maximum Flood Level = 15.5 m

Waterway design

Length of the clear waterway = Q/DV = 7050/(6.5×3.25) = 334m
velocity in the river is 3m/sec ,but at the siphon barrel the velocity should be limiting 3.25m/sec because to reduce the silt deposit in the barrel.
depth of the water D = bottom level of the trough - 8.5m = 15 - 8.5 = 6.5m
Provide clear water way of 9.25m of 36 spans are separated by 1.25m width piers
Total length of clear water way = 9.25x36 = 333m
Length occupied by piers = 35x1.25 = 43.75m
Total length of the water way = 376.75m = 377m
Center to center length of piers = 9.25 + 1.25/2 + 1.25/2 = 10.5m
hence provide 36 spans of length 10.5 m

VI. Design of RCC road way

Provide the width of road way as 5.5m with footpath = 0.5m.(intermediate lane)
Hence provide 36 spans of 10.5m span.
Total load = 9.82 KN/m^2

Moments

$M_x = {}_x \times w \times (l_x)^2 = 0.086 \times 9.82 \times 5.62^2 = 26.67$ KN-m

$M_y = {}_y \times w \times (l_x)^2 = 0.0526 \times 9.82 \times 5.62^2 = 16.31$ KN-m

Reinforcement in edge strip is provide 8mm diameter bars @ 250 mmc/c

VII. ANALYSIS OF TROUGH SIDEWALLS

total load on side walls = 566.568KN

Reactions calculation

Fixed end moments M_a & M_b = $wl^2/12$ =495.67KN-m

VIII DESIGN OF TROUGH

Design of side walls

Water pressure = 16.2 KN/m^2
Bending moment due to water pressure = 9.72 KN-m

A_{st} = 1894.68 mm^2

Hence provide 20 mm bars170 mm c/c

IX. **DESIGN OF TROUGH SLAB**

Live load on RCC road way = 6.93 KN/m^2
Self-weight of road way = 2.88 KN/m^2
Self-weight of trough side walls = 0.5×24 =12 KN/m^2
Assume trough thickness of 500mm
Therefore self-weight of trough slab = 0.5×24 = 12 KN/m^2

Water load = depth×density of water
= 1.8×9.81 = 17.66 KN/m^2

Moment calculation

M = wl2/10
M = (51.47×10.5×10.5)/10 = 567.46 KN-m

Effective depth =500-50 = 450 mm

Reinforcement calculation

Mu = 0.87 fy ×Ast ×d (1- (fy×Ast)/(fck×bd))
567.457×106 = 0.87×415×Ast ×450 (1- (415×Ast)/(20×1000×450))
567.457×106 =162472.5 Ast-7.49 Ast2
Ast=4375 mm2

Transverse reinforcement

Let us provide 32 mm
No of bars are 5.44(say) 6bars.
Spacing =170 mm (c/c)
Hence provide 6 bars **@170**mmc/c

Longitudinal reinforcement

Let us provide 25mm

No of bars = =8.9 (say) 10 bars.

Spacing = =100 mm
Hence provide 10 bars @100mmc/c

X FIXING OF LEVELS

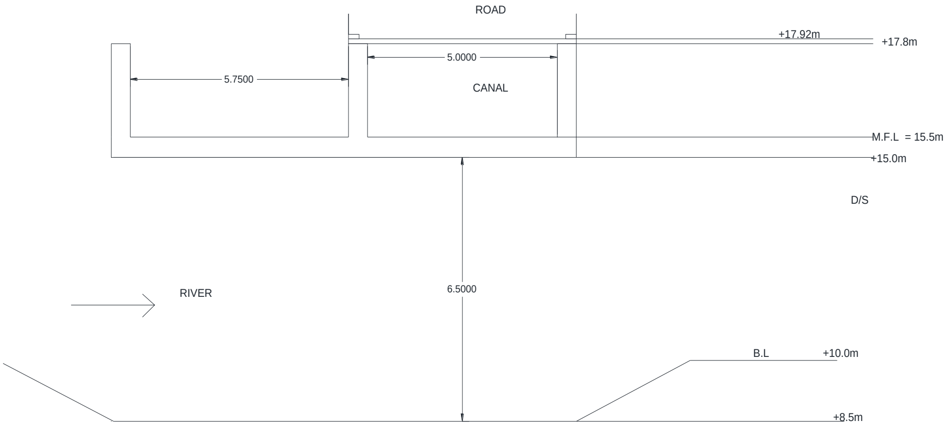
The Afflux is calculated by using Unwin's inverted formula.

h= (1+f_1+f_2

here f1 = 0.505
f2= a (1+0.3b/R)
a=0.003
b=0.10

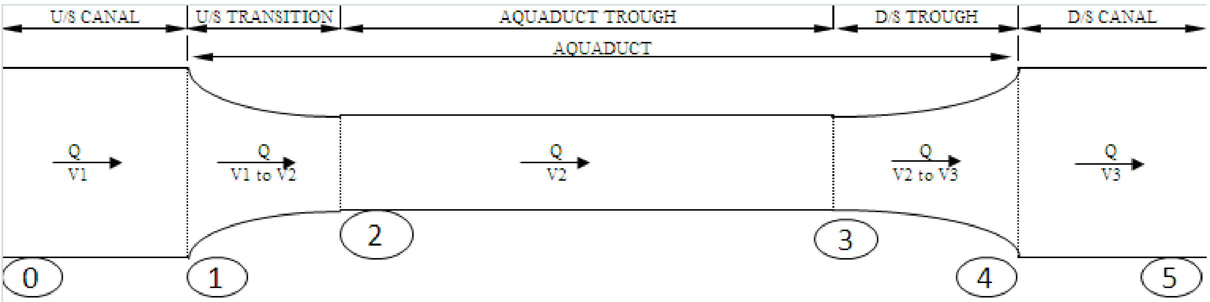
The afflux required on U/S to push the maximum flood discharge is 0.82m.

MFL@U/S= MFL@D/S+ Afflux
=15.5+0.82
=16.32 m
Maximum Flood Level on upstream side is 16.32 m



Fig(2) Fixing of Levels

XI HEAD LOSS CALCULATIONS IN CANAL TROUGH



Fig(3) Transistion zone

Normal canal section
total e level at AA = 17.3+0.035 = 17.335 m
Section at the upside transition
Eddy loss due to change in velocity

= = = 0.016

TEL at B-B = 17.349 m
Section at the entrance of trough
TEC at CC = 17.296 m
Mannings formula :

$$V = R^{2/3} S^{1/2} \quad \text{-----------} \quad Eq(6)$$

T.E.L @ Exit = 17.296 – 0.126 = 17.17 m
T.E.L at end w.r.t to end section of trough= 17.146 m

Provide 12m width of Reinforced Concrete trough with a velocity of water in the trough in 1.5m/sec. Hence loss of Energy is reduced.

XII PIER DESIGN

Section Properties

cross section area of pier = 12.5 x 1.25 = 15.625 m^2
I_{xx} = 203.45 m^2
I_{yy} =.035 m^2
z = 3.19 m^2

s.no	description of element	force(KN)	moment(KN-m)	stress(KN/m2)

		Vertical	**horizontal**	**along bridge axis**	**along pier axis**	
1	Deadload	3320.9				212.54
2	buoyancy effect	-1015.625				65
3	liveload on one span	400.25		180.11		82.077
4	longitudinal force from super structure & live load (i) traction /braking (ii)frictional		80.05 144.23	520 937.51		163 293.89
5	water current		29	125.67		3.86
6	wind force		90.33	587.13		18.04

Table(1). Details of Forces on Pier

XIII DESIGN OF ABUTMENT

provide abutment of 1.5m top width and 2m bottom width the face of abutment is vertical in water pressure and slopes to 1:2 on earth supporting side

w1 = 1.5 x 6.5 x 1 x 24 = 234 KN

w2 = 1/2 x 0.5 x 6.5 x 24 x 1 = 39 KN

w3 = 641.825 KN

w4 = 1/2 x 0.5 x 6.5 x 19 = 30.875 KN

Total vertical forces = 945.7 KN

Horizontal force

Pressure p = kγH =23KN/m^2

Total pressure = 253.77 KN

Table(2). Details of Moments on Abutment

S.NO	WEIGHT	LEVERARM	MOMENT
w_1	234	0.75	175.5
w_2	39	1.67	65
w_3	641.325	1	641.325
w_4	30.875	1.83	56.60
p_1	224.25	6.25/2	728.8
p_2	29.53	6.25/3	63.98

XIV. DESIGN OF WELL FOUNDATION

Table(3) Properties of soil

S.no	Property	symbol	Value
1	**Gravel %**		**0**
2	**Sand %**		**10%**
3	**Fine silt/clay**		**65%**
4	**Liquid limit %**	W_L	**47%**
5	**Plastic limit %**	W_p	**23%**
6	**Plasticity index**	I_P	**24%**
7	**Soil caly**	**CI**	**Inorganic clay of medium plasticity**
8	**Specific gravity**	**G**	**2.33**
9	**Optimum moisture content**	**OMK**	**30%**
10	**Maximum dry density**	**M.D.D**	**1.63 gm/cc = 15.99 KN/m^2**
11	**Cohesion**	**C**	**14.458 t/m^2 =141.8 KN/m^2**
12	**Angle of internal friction**	**ϕ**	**3.65**

Table.(4) Bearing Capacity Factors

Φ	N_c	N_q	N_Υ
3.65	6.12	1.18	0.002

Inclination Factor

i_c = 1 - μ/2CBL = 1 - 3738/2x141.8x7x15 = 0.87

i_q = 1 - 1.5H/V = 1 - (1.5 x 3738)/26979.3 = 0.79

q_u = 1099.43 KN/m^2

total load on foundation = 7674.468 KN

Determination of scour depth

According to IS : 3955 - 1967 the depth of foundation should not be less than 1.33 times the maximum scour depth.

The depth of the base of the well below the maximum scour level is kept not less than 2 m for pier and abutments with arches and 1.2 m for pier and abutments supporting other types of structure.

Size of well:

length = 15 m
width = 7 m
height = 11 m
thickness of steining = 1.5 m

XV. DESIGN OF WELLCAP

let us provide 75 cm thick well cap

self weight of well cap = 15 X 17 X 0.75 X 24 = 1890KN
moment of inertia (I_B) = 372.75 m^4
horizaontal force/ Q acting in the transverse direction.= 119.33 KN
M_B = -4767.29 KN-m
qmax = 81.46 KN/m2
qmin = 222 KN/m2

The maximum pressure should not be more than allowable soil pressure. the minimum pressure should not be negative.

hence safe.

Elastic analysis

H > M/ ϒ (1 + μμ^1) - μw

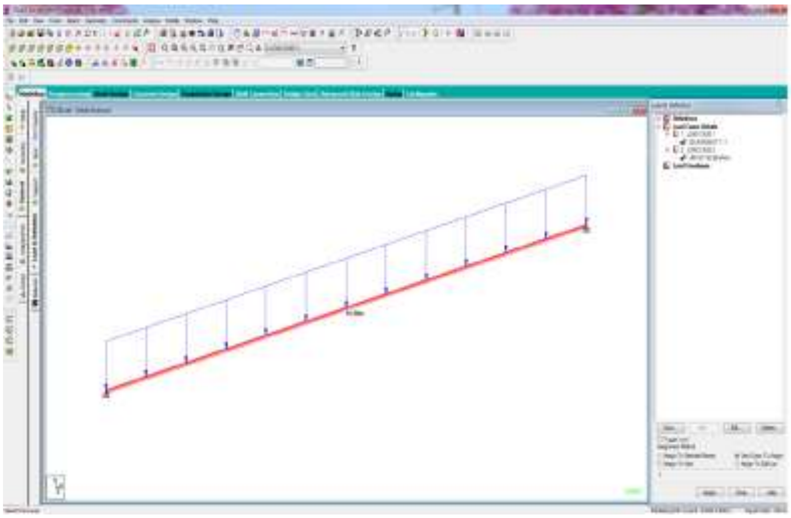
= 16821/19.76 (1 + tan 30^0.tan 20^0) - tan 30^0 X 26979

H > -14546

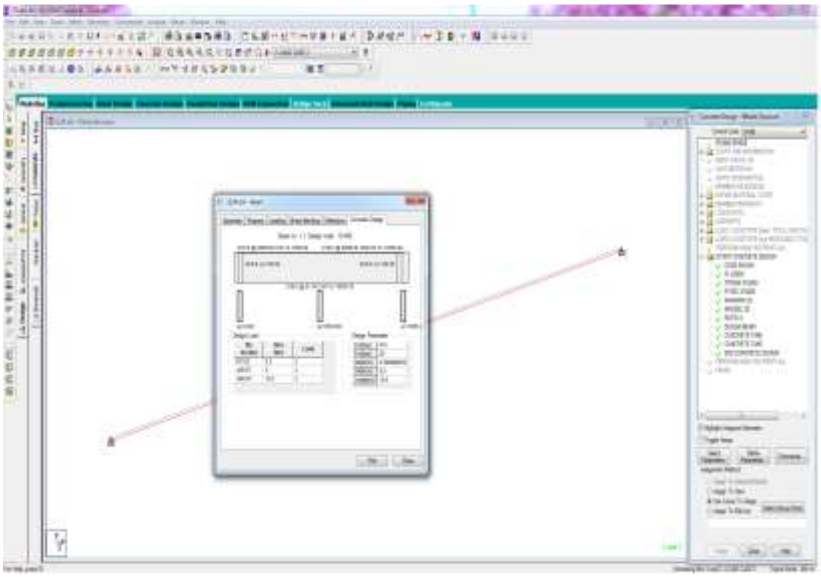
as H of 3738 is greater than -14546

Hence safe.

XVI. STADDPRO ANALYSIS & DESIGN OF SIDE WALLS



Fig(4) Side wall section with a length of 10.5m



Fig(5) Reinforcement Details of Side wall section with a length of 10.5m

B E A M N O. 1 D E S I G N R E S U L T S

M20 Fe415 (Main) Fe415 (Sec.)
LENGTH: 10500.0 mm SIZE: 500.0 mm X 2300.0 mm COVER: 25.0 mm
SUMMARY OF REINF. AREA (Sq.mm)

SECTION 0.0 mm 2625.0 mm 5250.0 mm 7875.0 mm 10500.0 mm

TOP 2355.42 0.00 0.00 0.00 2355.42
REINF. (Sq. mm) (Sq. mm) (Sq. mm) (Sq. mm) (Sq. mm)
BOTTOM 0.00 2311.82 2311.82 2311.82 0.00
REINF. (Sq. mm) (Sq. mm) (Sq. mm) (Sq. mm) (Sq. mm)

SUMMARY OF PROVIDED REINF. AREA

SECTION 0.0 mm 2625.0 mm 5250.0 mm 7875.0 mm 10500.0 mm

XVII REFERENCES

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