ANALYSIS AND DESIGN OF CANAL STRUCTURES (CROSS DRAINAGE WORKS)

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ANALYSIS AND DESIGN OF CANAL STRUCTURES (CROSS DRAINAGE WORKS)

Major Project

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(Computer Aided Structural Analysis & Design)

By

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Declaration

This is to certify that

a. The thesis comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

b. Due acknowledgement has been made in the text to all other material used.

Bhargav K. Kothia

Certificate

This is to certify that the Major Project entitled "ANALYSIS AND DESIGN OF CANAL STRUCTURES (CROSS DRAINAGE WORKS) " submitted by Bhargav K. Kothia (10MCLC22), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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Abstract

On it's alignment, a canal meets a number of natural drains, rivulets, streams and other obstructions such as roads, railways, valleys etc. When canal crosses any obstruction out of these, a suitable structure has to be provided to enable the canal to pass over or below the obstruction. This is referred as cross drainage works. Syphon and aqueduct are two different sub categories of cross drain age works.

When the canal is made to pass below the drain the structure so built is called canal syphon. Canal syphon is underground type structure, which consists of upstream transition walls, inclined barrel, horizontal barrel, downstream transition walls, breast walls.

When the canal is made to pass over the drain the structure so built is called aqueduct. Aqueduct is a bridge like structure. Aqueduct can use for both purpose, to pass water and vehicle traffic across an interception. Aqueduct consists of mainly two components like superstructure and substructure. Super structure consists of pipe, trough or box shaped barrel through which water flows depending upon the discharge. Sub structure consists of pier and foundation. Pier may be column type, wall type, hollow cellular type etc. The foundation can be open foundation, pile foundation, well foundation etc. depending upon the soil condition.

Focus of the present study is to perform analysis and design of Canal syphon and Aqueduct. Two shapes such as Rectangular box shape and Circular shape are considered for canal syphon. In ordered to understand economy of both shape, Comparison is done in terms of cost and head loss.

Aqueduct is design for both canal water and vehicle traffic load by considering rectangular barrel.Parametric study for economical design of aqueduct is performed. Span of aqueduct is varied to find the economical span.

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Contents

Declaration **iii**

Certificate **iv**

Abstract **v**

Acknowledgements **vi**

List of Tables **ix**

List of Figures **x**

1 Introduction **1**
- 1.1 General . . . 1
- 1.2 Types of canal structure . . . 1
- 1.3 Scope of work . . . 3
- 1.4 Organisation of report . . . 3

2 Literature Survey **5**
- 2.1 General . . . 5
- 2.2 Types of Cross- Drainage Works . . . 5
- 2.3 Selection of a Suitable Type of Cross- Drainage Works . . . 16
- 2.4 Selection of site for Cross- drainage work:- . . . 18
- 2.5 Possible causes of failure of cross- drainage works:- . . . 19
- 2.6 Codal provisions . . . 22

3 Analysis and Design of Canal Syphon **27**
- 3.1 General . . . 27
- 3.2 Analysis and Design of Box type Canal syphon . . . 29
  - 3.2.1 Hydraulic Design . . . 31
  - 3.2.2 Structural design of the Barrel . . . 35
- 3.3 Analysis and Design Circular type Canal syphon . . . 52
  - 3.3.1 Hydraulic Design of circular syphon . . . 52
  - 3.3.2 Structural Design of circular syphon . . . 55

4 Analysis and design of Aqueduct **57**
4.1 General . . . 57
4.2 Analysis and design of Super structure . . . 61
4.2.1 Determination of size of barrel and head loss calculation . . . 61
4.2.2 Transverse analysis and design of super structure . . . 63
4.2.3 Longitudinal analysis and design of superstructure . . . 71
4.3 Design of substructure . . . 82
4.3.1 Design of pier cap using limit state method . . . 82
4.3.2 Design of Pier . . . 86
4.3.3 Design of foundation . . . 95

5 Parametric Study **107**

6 Conclusion and further scope of study **109**
6.1 Conclusion . . . 109
6.1.1 Syphon . . . 109
6.1.2 Aqueduct . . . 110
6.2 Further scope of study . . . 110
6.2.1 Canal syphon . . . 110
6.2.2 Aqueduct . . . 110
References112

List of Tables

3.1 Main reinforcement summary 46
3.2 Shear reinforcement summary 48
3.3 Distribution reinforcement summary 49
3.4 Bar bending schedule for Barrel of syphon 51

4.1 Main reinforcement summary table (Normal condition) 68
4.2 Main reinforcement summary table (Seismic condition) 69
4.3 Distribution reinforcement summary 70
4.4 Barbending schedule 81
4.5 Ast calculation summary table 94
4.6 Summary : Load on Single pile 98

5.1 Variation in cost of one span of super structure and one unit of sub structure 107
5.2 Variation in total cost of Structure 108

6.1 Cost comparison 109
6.2 Headloss comparison 110

List of Figures

2.1 Aqueduct . . . 6
2.2 Barrel of aqueduct . . . 8
2.3 Syphon Aqueduct . . . 8
2.4 Super passage . . . 9
2.5 Canal Syphon . . . 10
2.6 Water supply canal alignment and canal syphon across Sarda river . . 13
2.7 Level crossing . . . 14
2.8 Inlet . . . 15
2.9 Inlet and Outlets . . . 15

3.1 Intersection of Canal and Drain . . . 28
3.2 Load acts on Barrel of syphon . . . 29
3.3 3D - STAAD model . . . 30
3.4 Centre line - STAAD model . . . 31
3.5 Cross section of Barrel . . . 32
3.6 Load on top slab . . . 37
3.7 Load on Botto slab . . . 38
3.8 Resultant load on Whole barrel . . . 39
3.9 Resultant load on Whole barrel . . . 40
3.10 Load diagram . . . 41
3.11 STAAD Bending Moment diagram . . . 42
3.12 STAAD Shear force diagram . . . 43
3.13 Reinforcement detail for Barrel of syphon . . . 50
3.14 Cross section of Circular syphon . . . 52
3.15 Reinforcement detail of circular syphon . . . 56

4.1 Intersection of Canal and Drain . . . 58
4.2 Longitudinal section of Aqueduct . . . 59
4.3 Cross section of Aqueduct . . . 60
4.4 Cross section of barrel of aqueduct . . . 61
4.5 Barrel (Trough) . . . 71
4.6 Reaction factor as per Courbon's theory . . . 73
4.7 Staad load and Bending Moment diagram . . . 75
4.8 Staad load and Shear force diagram . . . 76
4.9 Reinforcement detail for Barrel . . . 79

4.10 Reinforcement detail for Side wall of barrel 80
4.11 Pier cap 82
4.12 Combined effective width 84
4.13 Reinforcement detail for Pier cap 85
4.14 Reinforcement detail for pile cap 106

5.1 Variation in total cost of structure. 108

Chapter 1

Introduction

1.1 General

To fulfill a project purpose of producing crops or increasing crop production, water delivery to the land must be provided by a reliable and efficient irrigation system.

A canal is frequently used to convey water for farmland irrigation. In addition to transporting irrigation water, a canal may also transport water to meet requirements for municipal, industrial, and outdoor recreational uses.
Many different types of canal structures are required in an irrigation system to effectively and efficiently convey, regulate, and measure the canal discharge and also to protect the canal from storm runoff damage.

1.2 Types of canal structure

(a) Conveyance Structures

In addition to the canal itself, it is usually necessary, because of topography or existing manmade features, to use inline canal structures to convey water along the canal route. Such structures include:
(1) inverted siphons to convey canal water under natural channels

(2) road crossings to carry canal water under roadways

(3) bench flumes to conduct the water along a steep hillside

(4) drop or chute structures to safely lower the canal water down a hillside.

(5) Tunnels to convey canal water Tunnels to convey canal water through a ridge or hill than to: (i) pump water over the obstruction, (ii) convey the water along the hillside or around the ridge, or (iii) construct a canal section requiring a very deep cut.

(b) Regulating Structures

Regulation of canal discharge begins at the source of water supply. This may be a canal headworks structure adjacent to a diversion dam on a stream or river, a turnout from a larger canal,or a pumping plant located on a reservoir or large canal. Downstream from the source of water supply, regulation of canal discharge is primarily controlled by outflow through turnout structures. Where canal flow is to be divided and directed in several directions,division structures are used to regulate the discharge in each direction. Wasteway structures also are used to control flow in a canal

(c) Cross drainage works

On it's alignment, a canal meets a number of natural drains, rivulets, streams and other obstruction, such as roads, railways, valleys etc,

Cross drainage works are structures that help the canal to bypass such obstruction. Sometime, a cross- drainage work is required to dispose of the natural run- off from such areas intercepted by the canal, so that the canal supply remains uninterrupted.

(d) Water Measurement Structures

Efficient management of an irrigation system insists that measurement of the rate-of-flow and volume delivered be made.Equitable water distribution to the users is a primary consideration. Water measurement also tends to prevent unnecessary wasteful water management practices.

(e)Protective structures

Protective structures protect the canal system and adjacent property from damage which would result from uncontrolled storm runoff or drainage water, or an uncontrolled excess of flow within the canal. Strom or drainage water must have either : (i)controlled entrance into the canal through a drain inlet (ii) controlled conveyance over the canal in an overchute (iii)controlled conveyance under the canal through a culvert; or (iv) the canal must be routed under the cross-drainage channel in a siphon.

(f) Structure Components and Appurtenances

Nearly all canal structures are made of several different structural parts which together make up the complete structure.Components and appurtenances includes Pipe, Pipe appurtenances, Transitions, Energy dissipators, Safety features etc.

1.3 Scope of work

- Analysis and Design of Canal Syphon using box section.
- Analysis and Design of Canal Syphon using Circular Section.
- To understand economy of Box section and Circular section in terms of cost and head loss.
- Analysis and Design of Aqueduct.
- Parametric study of Aqueduct.

1.4 Organisation of report

Chapter 1 Covers the introduction of canal structures,scope of work and organisation of report.

Chapter 2 Deals with literature review. It includes the details of various literature covered in journals, paper and books.the literature divided in two parts, part I covers the general information regarding cross drainage works and part II covers the general codal provisions for design of cross drainage work.

Chapter 3 Consists on analysis and design of canal syphon. Which covers problem formulation, various forces acting the canal syphon, load combinations as per relevant IS code. Analysis and design of canal syphon divided into two parts, part I covers analysis and design of box type canal syphon and part II covers analysis and design of circular type canal syphon.

Chapter 4 Deals with analysis and design of aqueduct which covers analysis and design of super structure, pier cap, pier, pile cap, pile foundation.

Chapter 5 Covers the parametric study of aqueduct. Which includes quantity and cost of aqueduct having various span length.

Chapter 6 Covers conclusion and further scope of study.

Chapter 2

Literature Survey

2.1 General

For the objectives of major project discussed in Chapter 1, Literature review related to cross drainage is presented in this chapter.

Various books of Irrigation Engineering, authored by Arora [3], Varshney [1], Modi [6], Sharma [2], Garg [5] and Asawa [4] have given information about Cross Drainage Works.Technical publications by Aiseney,A.J. Hasney,R.B.[28] [29] and technical paper by Sastry [11],Raichur [13] has also given information and design aspects of CDW.

2.2 Types of Cross- Drainage Works

Depending upon the relative positions of the canal and drainage, the cross- drainage works may be classified in to 3 categories.

(1) Canal over the drainage.

I. Aqueduct

II. Syphon aqueduct

(2) Canal below the drainage.

I. Super Passage

II. Canal Syphon

(3) Canal at the same level as drainage.

I. Level crossing

II. Inlet

III. Inlet & Outlet

The brief description of above mention categories are as below.

(1)Canal over the drainage.

(I)Aqueduct

Aqueduct is structure in which the canal flows over the drainage and the flow of the drainage in the barrel is open channel flow,as shown in Fig.2.1 .An aqueduct is similar to an ordinary road bridge over a drain but in aqueduct the canal is taken over the drainage instead of a road.

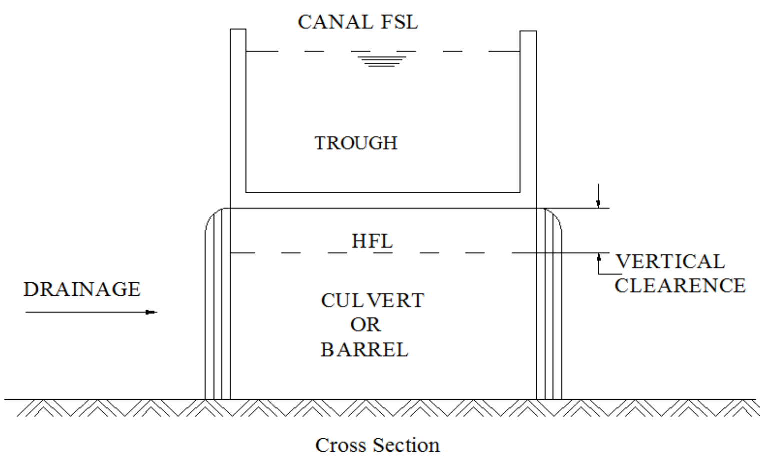


Figure 2.1: Aqueduct

The canal is taken over the drainage in trough ported over the piers constructed on drainage bed. An aqueduct is provided when the canal bed level is higher than the H.F.L. of the drain.

Analysis and structural design of Aqueduct

M.G.Raichur [13]has given introduction of various type of Cross drainage work and some design aspects of aqueduct.

a) Types of aqueducts.

i) Culvert type type ii) Box or trough type In the culvert type aqueduct the canal section is carried fully over the drain by providing a slab or an arch over the drain. Here the section of the canal is not flumed. In the trough or the box type the canal section is flumed and the water is carried in a masonry or R.C.C. structure laid over the piers and abutments.

b) Fluming

The advantage of fluming the canal would naturally be to narrow down the size of box or trough with the result the cost of structure gets reduced. But there is a limit to the extent of such fluming from consideration of (i) Head loss in the structure and (ii) bearable velocity in the trough.

Normally the area of flumed section is kept half the area of canal water way. However maximum fluming to the extent of 67 per cent is adopted in rare cases.

c) Design of the box (barrel)

Sides AD and BC are designed as simply supported beams resting on the piers or a abutments. The beam is designed for self load , dead load reaction of the top and bottom slab and that of water load. Also the live load reaction from the top slab is accounted for in the design of the beams.

The moment due to dead load and live load are added to get the design moment for the beam. The beam is again designed for maximum shear. The shear reinforcement is then worked out at various location of the beam. There will be the effect of direct tension on the side beams caused by the reaction of water load and slab load. Steel for this direct tension is worked out and added to the shear reinforcement required

Figure 2.2: Barrel of aqueduct

for shear.

The box A B C D is also analysed as a closed structure with load from top and bottom slabs, side walls and water load from within. The dead load and live load moments are added and combined moment diagram is worked out. For this combined moment steel at various corners and centres on the outer face and for inner face is worked out.

(II)Syphon Aqueduct

In a syphon aqueduct also the canal is taken over the drainage, but the flow in the barrel of the drainage is pipe flow.as shown in Fig.2.3 A syphon aqueduct is con-

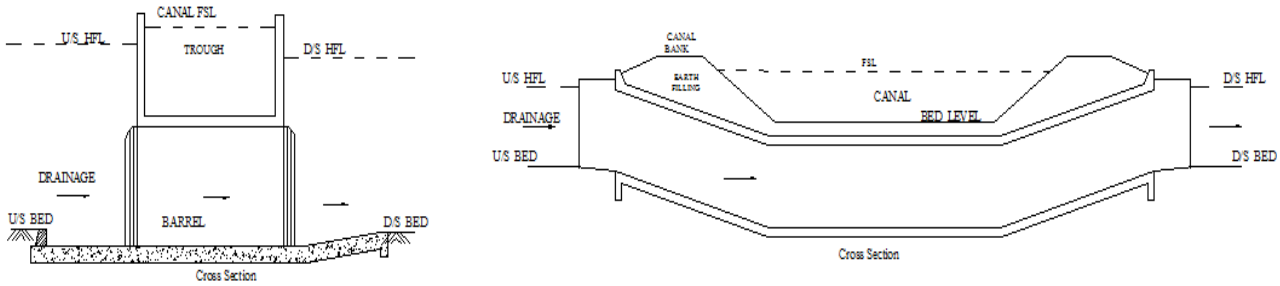


Figure 2.3: Syphon Aqueduct

structed when the H.F.L. of the drainage is higher than the canal bed level.

(2)Canal below the drainage.

(I)Super passage

In a super passage, the canal is taken below the drainage and flow in the canal is open channel flow.As shown in Fig.2.4 A super passage is constructed when the canal

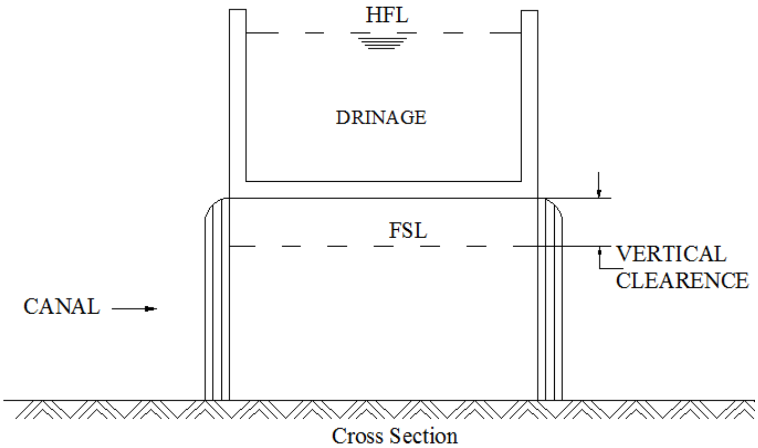


Figure 2.4: Super passage

F.S.L. is below the drainage bed level.

(II)Canal Syphon

A canal syphon is a structure in which the canal is taken below the drainage and the flow of the canal in the barrel is pipe flow. A canal syphon is constructed when the F.S.L. of the canal is above the drainage bed level.As shown in Fig.2.5 Syphon used to convey other structures, various types of drainage channels, and depressions. A siphon is a closed conduit to run full and under pressure. Closed conduits with straight profiles under road ways and rail track may also function as siphons with internal pressure.

(A)Structure Components of canal syphon:-

(a)Closed conduit:- It may in form of pipe or reinforced cement concrete barrel of circular , rectangular of horse shoe shape. Precast reinforced concrete pressure pipe, asbestos-cement pressure pie, reinforced plastic mortar pressure pipe may use as pipe conduit. The profile of conduit is determined based on certain requirements of cover,

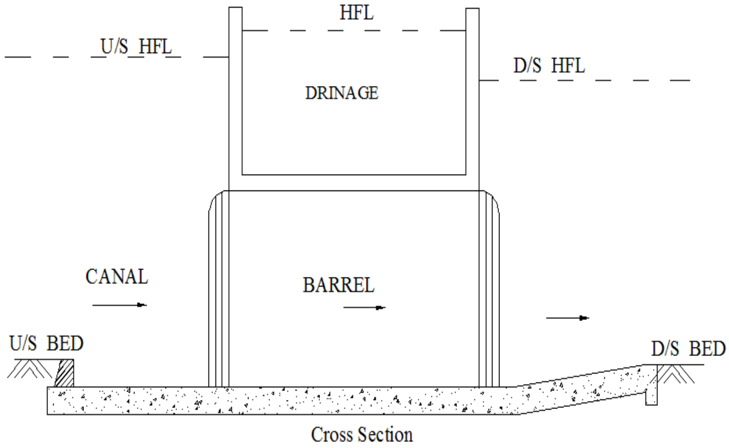


Figure 2.5: Canal Syphon

conduit slopes, bend angles, and submergence of inlet and outlet.

(b)Transition walls:- transition walls provided at the inlet and outlet of a syphon provides smooth entry and exit of water at upstream and downstream of syphon. It reduces head loss and prevents canal erosion in unlined canals by causing the velocity change between the canal and syphon.

(c)Pipe collars:- Pipe collars are transverse fins that extend from the pipe into the surrounding earth and prevents leakage of water. Collar often provided to reduce the velocity of water moving along the outside of pipe thereby prevents piping effect.

(d)Blowoff Structures:- Blowoff structures are provided at or near the low point of relatively long inverted siphons to permit draining the pipe for inspection and maintenance.It may also be used in an emergency in conjunction with wasteways for evacuating water from canals.A manhole may include with a blowoff on long siphons 36 inches and larger in diameter to provide an intermediate access point for inspection and maintenance.

(e)Wasteways:- Wasteways are often placed upstream from a syphon for the purpose of diverting the canal flow in case of emergency.

(B)Advantages and Disadvantages of canal syphon:-

a. **Advantages**

1 Inverted siphons are economical, easily built, and have proven a reliable means of water conveyance.

2 Normally, canal erosion at the ends of the siphon is inconsequential if the structures in earth waterways have properly designed and constructed transitions and erosion protection.

3 Costs of design, construction,& maintenance are factors that may make an inverted siphon more feasible than another structure.

b. **Disadvantages**

1 Syphon produce more head loss which turns in reduction in command area of and to be irrigated.

Sastry and **Bheemiah**[11] dealt with the canal syphon on water supply canal across river Sarda located near Anakapalle, Visakhapatnam, Dist. Andhra Pradesh.fig.2.6 Due to following reason, canal syphon was selected.
For an Aqueduct to be proposed for crossing, it was aligned on the right side of Railway Line and Highway and it was crossing the commercial township Anakapalle. The proposal worked out was costly.
Due to this constraint it was proposed to cross Sarda River by means of a syphon.
It was found that the cross sectional shape of the inverted syphon depends on the conditions of its static loading and functional requirements. When internal pressure was small and external load considerably higher, the inverted syphon was made to form a box when multiple barrels were found necessary. So rectangular shape was adopted for easy construction.
Analysis of structure was made by using moment distribution method for following conditions.
a) Canal flowing full and Drain flowing full.
b) Canal empty and Drain flowing full.

c) Canal flowing full and drain empty.

d) Drain empty and one barrel full.

e) Drain flowing full and one barrel full.

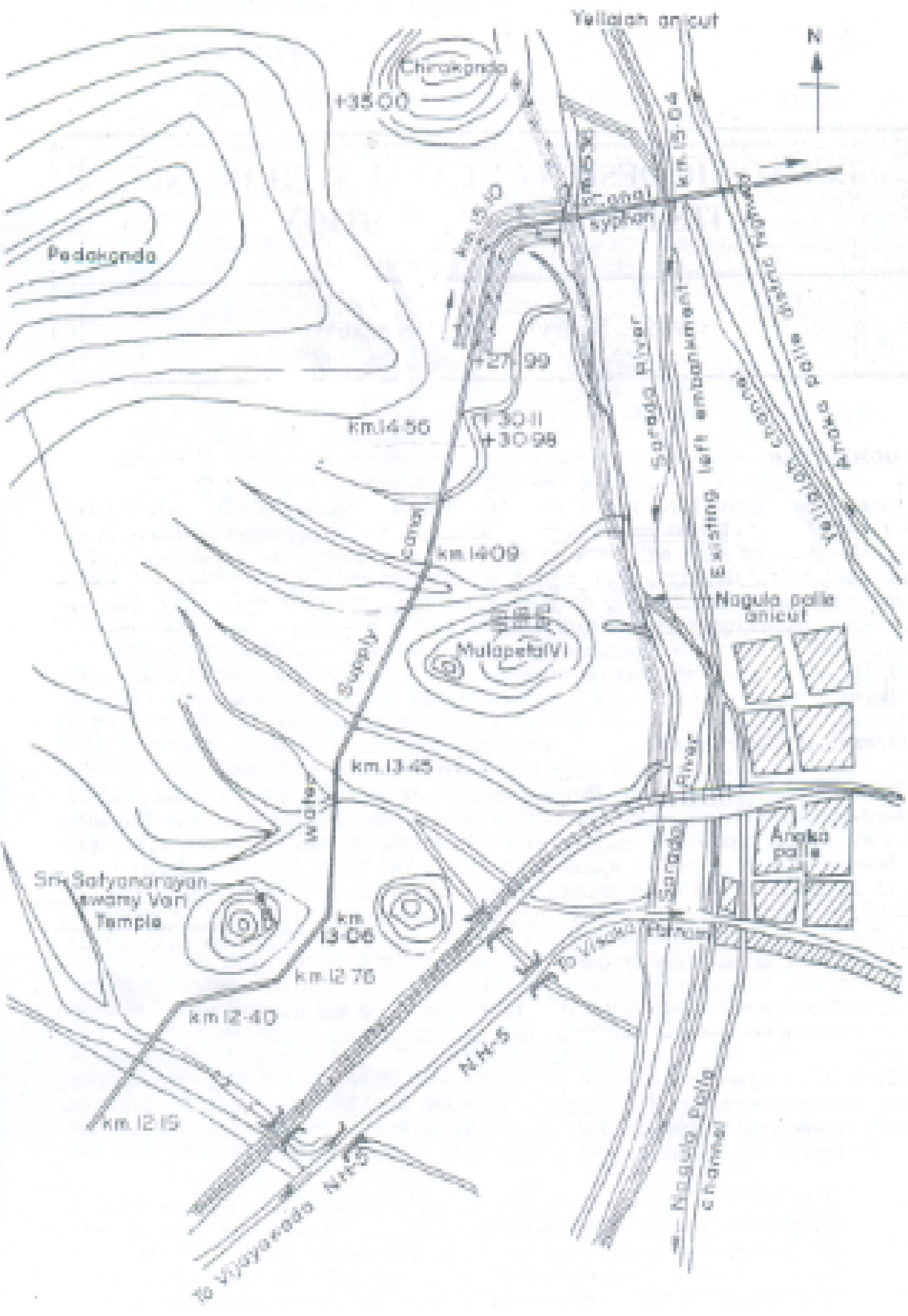


Figure 2.6: Water supply canal alignment and canal syphon across Sarda river

(3)Canal at the same level as drainage.

(I)Level crossing

Level crossing is provided when the canal and the drainage are at the same level. In a level crossing , the drainage water is admitted into the canal at one bank and is taken out at the opposite bank as shown in Fig.2.7 A level crossing usually consist

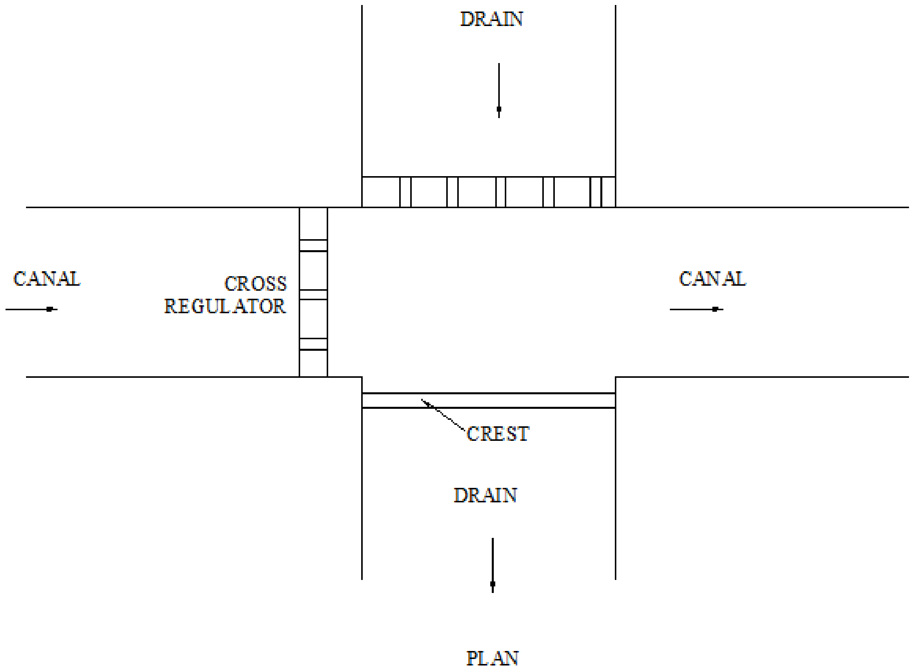


Figure 2.7: Level crossing

of a crest wall provided across the drainage on the upstream of the junction with its crest level at the F.S.L. of the canal. The drainage water passes over the crest and enters the canal whenever the water level in the drainage rises above the F.S.L. of the canal. There is a drainage regulator on the drainage at the down stream of the junction and a cross regulator on the canal at the down stream of the junction for regulating the outflows. Level crossing is provided on the canal when it is more or less at the same level the drainage and is a large discharge in the drainage for a short duration. The main disadvantage of a level crossing is that an operator is required to regulate the discharge.

(II)Inlet

An inlet alone is sometimes provided when the discharge is very small with a very low discharge and it does bring heavy silt load.Fig.2.8 Of course, it increases the

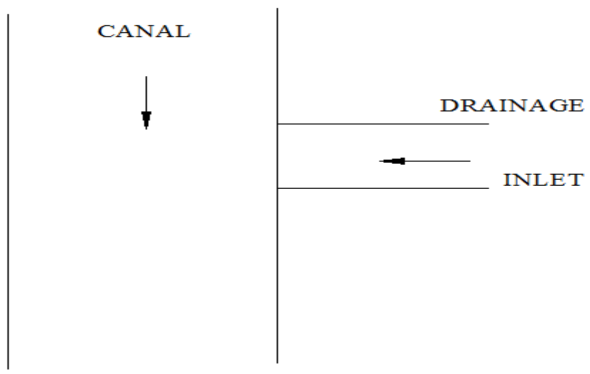


Figure 2.8: Inlet

discharge in the canal, which is absorbed in the space provided as the free board above the F.S.L.

(III)Inlet and Outlets An inlet outlet structure is provided when the drainage and the canal are almost at the same level, and the discharge in the drainage is small. The drainage water is transmitted into the canal at a suitable site where the drainage bed is at the F.S.L of the canal. The excess water is discharged out the canal through an outlet provided on the canal at some distance downstream of the junction as shown in Fig.2.9

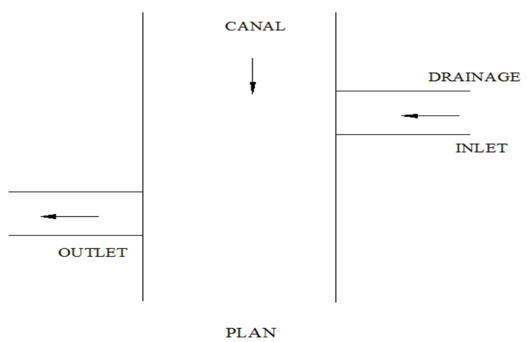


Figure 2.9: Inlet and Outlets

An outlet is usually combined with some other masonry work where an arrangement for removing the excess water is even otherwise required.

2.3 Selection of a Suitable Type of Cross- Drainage Works

(1) Relative bed levels, water levels, and discharge of the canal and the drainage. This parameter mainly affects the type of Cross-drainage work. The following are the broad outlines:-

i If the canal bed level is sufficiently above the H.F.L. of the drainage, an aqueduct is preferable.

ii If the bed level of the drainage is sufficiently above the F.S.L. of the canal, a supper passage is suitable.

iii If the canal bed level is only slightly below the H.F.L. of the drainage, and the drainage is small, a Syphon aqueduct is provided.If necessary, the drainage bed is depressed below the canal

iv If the bed level of the drainage is slightly below the F.S.L. of the canal and the canal is of small size, a canal Syphon is suitable.

v If the canal bed and the drainage bed are almost at the same level.

- A level crossing is provided when the discharge in the drainage is large.

- An inlet-outlet structure is provided when the discharge in the drainage is small.However, the relative levels of the canal and the drainage can be altered and manipulated by suitably changing the canal alignment, so that the point of crossing is shifted upstream or down stream of the drainage.
In that case, the suitable type of the cross- drainage work will be selected depending upon the levels at the changed crossing.

(2) Performance:-

- In the case of a canal Syphon and a Syphon aqueduct silting problems usually occur at the crossing. As well as, in the case of canal Syphon loss of head results in considerable loss of command area.
  Therefore, As far as possible, the structure having an open channel flow is preferable compare to structure having pipe flow.

- The performance of inlet- outlet structures is not good and should be avoided.

(3) Size of drainage:-

- When the drainage is of small size, a Syphon aqueduct will be preferred to an aqueduct as the letter involves high banks and long approaches. However, if the drainage is of large size, an aqueduct is preferred.

(4) Provision of road:-

- A aqueduct is better than a supper passage because in the former a road bridge can easily be provided along with the canal through at a small extra cost, whereas in super passage a separate road bridge is required.

(5) Cost of earth works:-

- The type of cross drainage work in which large quantity of earthwork required, should not preferable.

- In high embankment problem of stability also arises.

(6) Cost of construction:-

- The cost of construction of cross drainage work should not be excessive.

(7) Foundation:-

- The type of cross drainage work should be selected depending upon the foundation strata available at the site of work.

(8) Permissible head loss:-

- Sometimes, the type of Cross-drainage is selected considering the permissible loss of head.

- ie.If the head loss cannot be permitted in a Cross- drainage work like canal Syphon, in such case canal Syphon should avoid.

(9) Material of construction:-

- Suitable types of materials of construction in sufficient quantity should be available near the site for the type of Cross- drainage work selected.

(10) Overall cost:-

- The overall cost of the canal banks and cross- drainage work, including maintenance cost, should be a minimum.

(11) Subsoil water table:-

- In the subsoil water table is high the types of cross- drainage which requires excessive excavation should be avoided.

2.4 Selection of site for Cross- drainage work:-

The following points should consider for select the site.

(1) The drainage should cross the canal alignment at right angle. Such a site provides good flow conditions and also the cost of the structure is usually a minimum.

(2) For economic purpose a firm and strong sub -stratum for foundation should exist at a reasonable depth.

(3) The stream at the site should be stable and should have stable banks.

(4) The length and height of the marginal banks and guide banks for the drainage should be small.

(5) The site should be such that long and high approaches of the canal are not required.

(6) The water table at the site should not be high, because it will create dewatering problems for foundation.

(7) In the possibility of diverting one stream in to another stream upstream of the canal crossing should also be considered and adopted, if found economical and feasible.

(8) The possibility of diverting one stream in to another stream upstream of the canal crossing should also be economical and feasible.

(9) As far as possible the site should be selected of the confluence of two stream to avoid the necessity of construction of two cross- drainage works.

(10) A cross- drainage work should be combined with a bridge, if required. If necessary, the bridge site can be shifted to the cross- drainage work or vice versa.

2.5 Possible causes of failure of cross- drainage works:-

One or more factors described below may lead failure of the structure.

(1) Weak structures

In the case of an aqueduct, the over head concrete flume, supported on piers and abutments should be structurally sound so that there is no failure owing to bending, sheave or bond.

Side walls should be designed against the lateral thrust and the bottom floor should be designed for water and traffic loads in case a bridge is combined with the aqueduct.

The barrels in the case of a Syphon aqueduct will be subjected to internal pressure, when the drain is empty, the barrel will be subjected to vertical pressure due to overburdening and surcharge. It will also be subjected to lateral earth pressure.
The barrels should be designed consider ding all such critical loading condition.

(2) Weak foundation

The foundation of the abutments and piers in an aqueduct should be strong and extended beyond the maximum possible depth of scour.
In the design of abutments, the uplift pressure arising out of seepage from the canal should be considered, beside the usual earth pressure and hydrostatic thrust.
The foundation of an inverted Syphon aqueduct is subjected to uplift due to rise of water table above the foundation which can blow out the foundation The barrel should either be anchored to the foundation or sufficient foundation thickness should be provided.

(3) Inadequate water way for the drain

To economies the construction sometimes the drainage water way is made smaller than water is necessary for passing the highest probable flood discharge. Inadequate water way will cause afflux to such an extent that the water level in the drain (upstream) may overtop the canal banks by out.

(4) Defective Transition

If the transitions are not designed properly, there can be severe erosion, resulting in the failure of the Cross- drainage works.
An abrupt transition is responsible for high head loss and consequent high afflux, separation and consequent concentration of flow on one side, resulting in the erosion of the bed and banks.

(5) Leakage and piping Usually the wing-walls are separated from the abutments and the concrete trough of the aqueduct is provided with adjoining structure and for free movements.

All such joints must be adequately sealed against leakage by use of water sealing compounds. A mixture of bitumen, sand and cement with jute or some other fibrous materials provides good water sealant.

Water seepage through the interface between the abutments and the soil in contact as well as wings and soil in contact can cause piping.

The through path of creep flow or seepage flow should be broken by the provision of ribs cut-off etc.

(6) Scouring of Bed and Banks

The drainage bed and bank and scour if the flood is high, Drainage Syphon must be protected with a cut-off and pitching of the floor, both at the upstream and down stream ends to prevent undermining of the barrels.

The apron and cut- off should be design to resist the uplift due to seepage from the canal in to the drain.

A similar protection is required for the canal siphon.

In the case of aqueducts, the apron and exit ends of the trough are in heavy filling and should be protected against scouring by paving the bed with masonry blocks and stone pitching along with cut- off at the end of the apron.

(7) Negligence in Construction

Strict adherence to specification for filling, concreting, curing, jointing, etc is essential for class I work. Negligence in supervision even for apparently minor items ie.back filling can lead to failure, resulting in a colossal loss of water and loss of property.

All such works have an economic life extending up to about **100years**. Any negligence in the planning, design and construction of a structure can reduce it life.

2.6 Codal provisions

General features for design of cross drainage work as IS 7784 (Part 1) : 1993 are listed below.

A **Data requirement**

a) **An Index map**

An index map suitable scale showing the recommended location of the cross- drainage structure, the alternative sites, general topography and the important habitation in the vicinity.

b) **Catchment area map** Map with contour marking at suitable intervals showing the main drainage channel from its sources together with all tributaries.

Existing under construction or proposed embankments and flood management measures should also be shown.

c) A detailed survey plan of the drainage channel to suitable scale showing important topographical features extending considerable distances, downstream and upstream, of the proposed site of crossing and either of its banks.

d) **Site plan** showing details of

- Site selected,cadestral survey plot numbers important topographical features like depressions near the proposed alignment of canal, general sub-soil water levels , etc.
- Bench-mark used as datum with its full description and reduced level
- The locations of the various trial pits and/or borings with their identification numbers;
- The lines and identification numbers of the cross sections and longitudinal sections of drainage channel taken within the scope of site plan

* The contour of the drainage channel,direction of flow of water,the angle of direction of crossing

e) **Cross section of drainage channel**

* Cross section covering the bed and banks of the channel portion and the ground levels beyond the banks covering the entire flood plane,
* Nature of the soil in bed, bank and approaches
* Low water level and Maximum flood level.
* Longitudinal section of the drainage channel showing the location of the cross drainage work, with levels of the observed flood, the low water and the bed levels at suitably spaced intervals along the line of the deep water channel.

B **Hydraulic design**

a) **Water way**

I) Water way fixed based on following factors

· Design flood

· Topography of the site

· Existing and proposed selection and slope of the drainage channel in the vicinity of the crossing

· Permissible afflux

· Construction and maintenances aspects

II) In plains, the water way usually provided in works without rigid floor is about 60 to 80% of perimeter given by Lacey's formula

$P_w = C[Q]^{1/2}$

where, p = wetted perimeter in m

C = 4.5 to 6.3 according to local condition, the usual value adopted being 4.8 for regime channel

Q = designed flood in m^3/s

· In the construction with rigid floors, water way can flumed within the permissible limit of velocity. Velocity should be limited to the values given in below table.

Note : when the flow carries abrasive materials with it, the permissible values may be further reduced by 25%

· The minimum dimension of openings should be such as to permit, manual clearing of deposits.

b) **Clearance for aqueduct**

I) **Clearance for Rectangular openings**

Minimum clearance for rectangular openings are suggested in table below. If the minimum clearances specified in above table are not available; safety of the super structure should be ensured against likely repercussions.

II) **Clearance for Arch openings**

Minimum clearance measured to the crown of the arch should normally as specified for rectangular openings.

III) In case the of drainage channels, where a bed rise due to progressive silting is anticipated, the permissible clearance specified in above table should be increased to allow for such aggradations depending upon the extent of silting.

IV) **Free board**

On aqueduct structure the free board depends on

* High flood level including afflux in drainage channel and
* Full supply level of canal the free board should not be less than 900mm

c) **Clearance for super passage**

- **Clearance**

  Clearance of about 50 % of those specified for Clearance for Aqueducts with rectangular openings and aqueducts with arch openings with required changes may be provided in case of super passages.

- **Free board**

  Free board specified for aqueduct may be provided

C **Loss of head**

When water flows through any structure there are head losses due to various factor such Inlet and outlet,Elbows or Bends in barrel,Skin friction etc.

The total head loss H = h_1 + h_2 + h_3 +h_4

where

h_1 = losses at the inlet and outlet (for syphon),

h_2 = losses at elbows or bends (for barrel),

h_3 = losses due to transitions (other than syphon),

h_4 = losses due to skin friction (for barrel and trough).

I Lass of Head at the Inlet and at the Outlet of Syphons

$$h_1 = [1 + f_i] \times \frac{v^2}{2g} \tag{2.1}$$

where

f_i = coefficient

f_i = 0.08 for a bell mouth entrance

f_i = 0.505 for sharp edge

II h_3 generally applicable for normal design and installation condition.this is not applicable to syphons.

III Loss of head due to skin friction in Barrels and Troughs

$h_4 = \frac{v^2 * n^2}{R^{\frac{4}{3}}} \times$ Length of barrel

D **Transition walls**

Transition walls at ends of cross drainage work,turn nearly right angles to flow in the channel and should extend for a minimum length of 0.6m into the earth bank.

Chapter 3

Analysis and Design of Canal Syphon

3.1 General

Problem formulation

As shown in fig.3.1 Canal and natural drain intersects each other at 90^0.Canal having bed width of 5m and carries 12.5cumecs discharge. Width of Drain at bank level is 104 m and average width is 98m and carries 500 cumecs discharge.

As shown in figure canal bed level(C.B.L.) is higher than the drainage bed level (D.B.L.) but lower than the highest flood level (H.F.L.) of he drain. Hence, clearance between C.B.L. and D.B.L. is not available. So Aqueduct type cross drainage work (CDW) is not possible. In this type of site situation there are two possibilities for CDW : (i) Canal syphon and (ii) Drainage syphon. Here, discharge of drain is very large compare to discharge of canal, so drainage syphon is very expansive in this situation. Due to less discharge, out of two Possibilities of canal syphon and drainage syphon, canal syphon is advisable.

Design of Canal syphon consists of two parts:

(1)Hydraulic design

It covers determination of size of barrel, uplift check for barrel and head loss calcula-

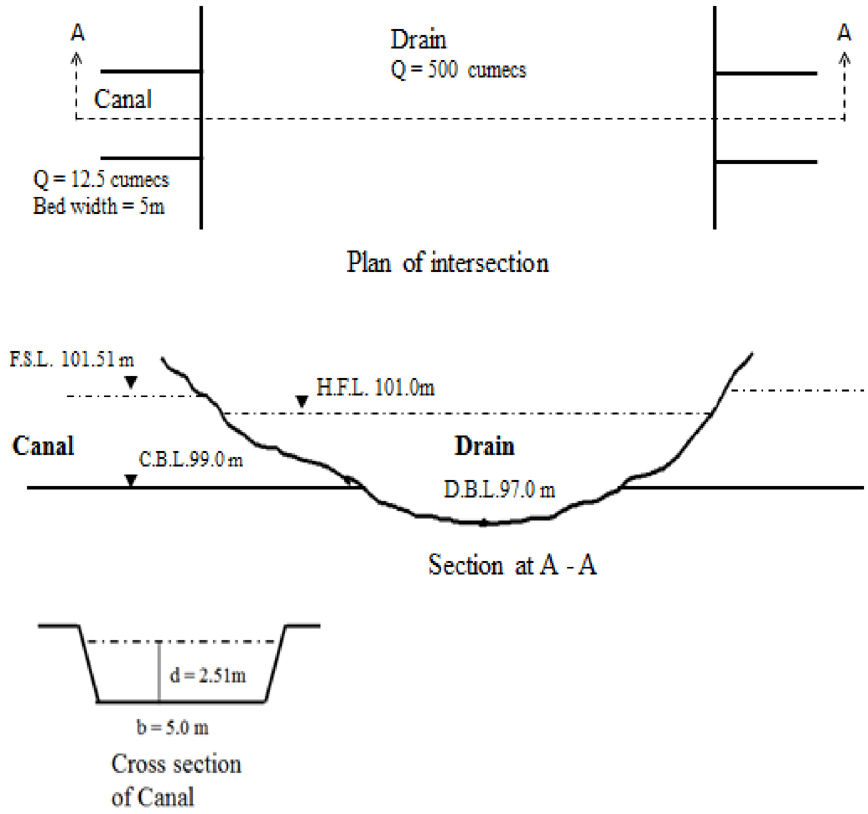


Figure 3.1: Intersection of Canal and Drain

tion, for this excel sheets are prepared.

(2)Structural design

In this section calculation of all the forces acting on the barrel of syphon is carried out,for this calculation excel is sheet is prepared.

3.2 Analysis and Design of Box type Canal syphon

Loads acts on barrel of syphon shown in fig.3.2 (cl 7.1,IS 7784 part2/sec3)

a) Self weight of the structure

b) Super impose loads :Weight of water in drain

c) Surcharge : Consists of weight of buoyancy or weight of soil or both

d) Bursting pressure on Whole periphery : From inside to out side bursting pressure due to head difference.

e) Soil reaction : Base pressure at bottom slab of barrel.

f) Earth pressure on side walls: Due to earth filling near the side wall.

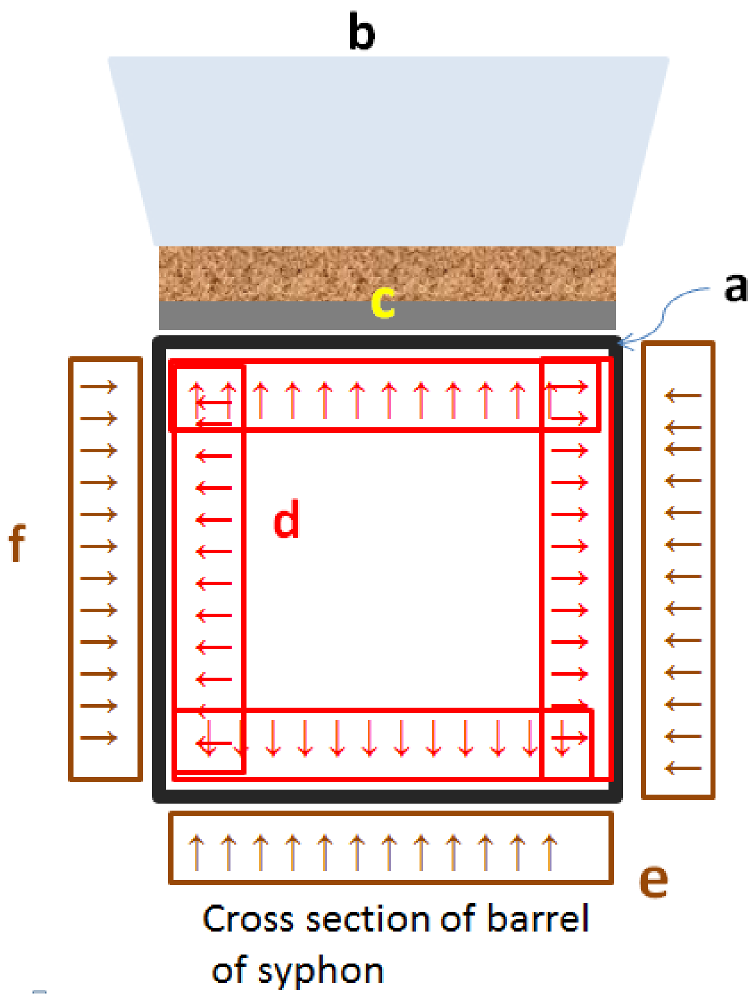


Figure 3.2: Load acts on Barrel of syphon

Load Cases (cl 7.1,IS 7784 part2/sec3)

1) Syphon full and Drain dry

2) Syphon dry and Drain full

After calculation of all loads, to determine bending moment,shear force and axial force in all components of barrel STAAD model fig.3.3 is prepared.

In staad model results are taken at face of support and midpoint,as shown in fig.3.4

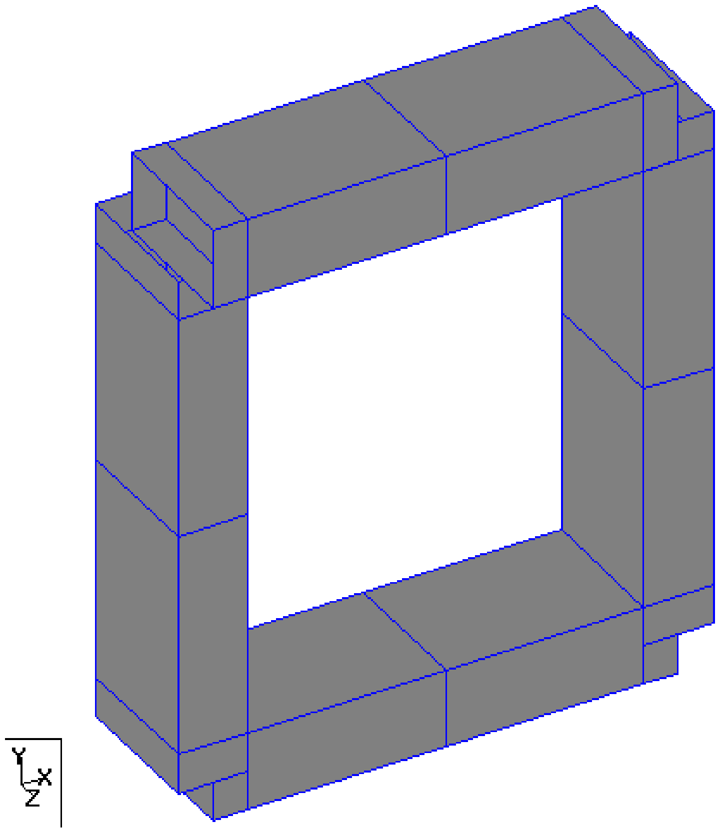


Figure 3.3: 3D - STAAD model

For reinforcement calculation excel sheet is prepared.

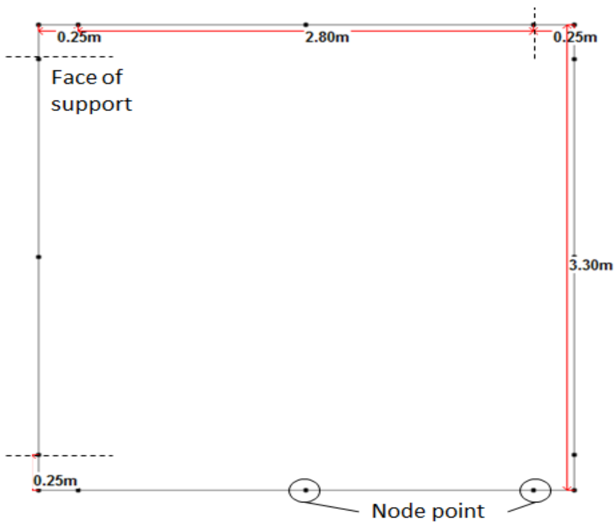


Figure 3.4: Centre line - STAAD model

3.2.1 Hydraulic Design

A Fixing the size of barrel

DATA

Q_d=12.5 cumecs

Design velocity,V_d=1.36 m/sec

Try, $2.8m * 2.8m$ sizer Barrel as shown in fig.3.5

So,$Areaofwaterway = [2.8 * 2.8 - (4 * 0.15 * 0.15/2)] = 780m^2$

Perimeter = 10.85m

Now,$R = \frac{A}{P} = \frac{7.8}{10.85} = 0.718$

Then Velocity, $V = \frac{Q_d}{A} = \frac{12.5}{7.8} = 1.604m/sec$

Velocity > Design velocity

Hence OK.

So,provide $2.8m * 2.8m$ sizer Barrel as shown in fig.3.5

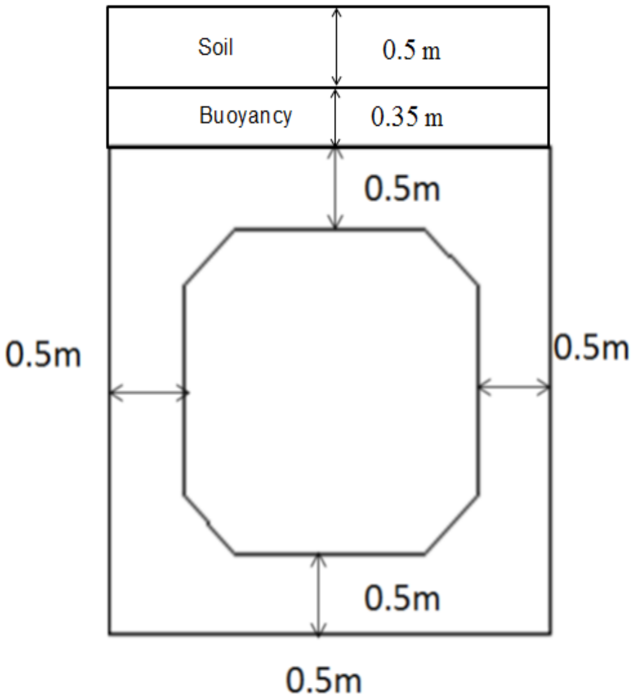


Figure 3.5: Cross section of Barrel

B Check for uplift

DATA

Depth of soil surcharge above barrel = 0.5m

Density of soil = 18 kN/m^3

Thickness of buoyancy concrete above barrel = 0.35m

$U/SFSL$ = 101.51m

Drainage bed level=97m

HFL=101m

Density of water = $\S_w$ = 10 kN/m^3

Case 1. Canal at FSL and drain dry

(1.)Self wt. of barrel

Weight of top slab = 47.5 kN/m

Weight of bottom slab = 47.5 kN/m

Weight of side wall = 70 kN/m

Weight of haunch = 1.13 kN/m

(2.)Weight of the water in barrel(W2) = 77.95 kN/m

(3.)Weight of the soil surcharge(W3) = 3.8*0.5*18=34.2 kN/m

(4.)Weight of the buoyancy(W4) = 0.35*3.8*24 = 31.92 kN/m

Total downward force,W = 165+77.95+34.2+31.92 = 309.07kN/m

RL of bottom of barrel=97-0.5-0.35-2.8-0.5=92.85m

Uplift force=(HFL-RL of bottom of barrel)*b*$\S_w$

. =(101-92.85)*3.7*10*0.75=246.81 kN/m

F.S.= $\frac{309.07}{246.81} = 1.25$

Hence OK.

Case 2. Canal dry and Drain full

Total Downward force,$W = W_1 + W_2 + W_4 + W_6$

1.Self weight of the Barrel(W_1) = 165 kN/m

2.Weight of the buoyancy concrete(W_4) = 31.92 kN/m

3.Weight of water above Barrel(W_6) = 179.45 kN/m

Total Downward force,W = 381.22 kN/m

Uplift force=(HFL-RL of the barrel)*b*$\S_w$

. =(101-92.85)*3.8*10=309.7 kN/m

F.S.=$\frac{381.22}{309.7}$= 1.23

Hence OK.

C Head loss calculation

a. Loss due to Entry and Exit(HL_1).

F_1=0.505

$HL_1 = (1 + F_1) * \frac{V_2^2}{2g} = (1 + 0.505) * \frac{1.604^2}{2*9.81} = 0.2m$

b. Loss due to friction in syphon Barrel(HL_2).

Size of Barrel=$2.8 * 2.8m^2$

Consider, n=0.018 for concrete

S_f=Friction slope in Barrel

$S_f = \frac{V^2n^2}{R^{\frac{4}{3}}}$

Where, V=Velocity of water in Barrel

R=Hydraulic mean radius

Now, $R = \frac{A}{P} = \frac{7.8}{10.85} = 0.718$

$V = \frac{Q}{A} = \frac{12.5}{7.8} = 1.604m/sec$

$S_f = \frac{1.604^2 * 0.018^2}{0.718^{\frac{4}{3}}} = 0.0013$

$HL_2 = S_f * L_b = 0.163m$

. Where, L_b=Width of drain=125.63m

c. Loss due to bend of syphon(HL_3).

$\tan\theta = 0.333$

So,θ=18.43'

For inlet bend at 18'43'

$HL_{3(a)} = K * \frac{V^2}{2g}$

Where,K is taken from the graph from IS:2951(Part-III)1965

$HL_{3(a)} = \frac{0.03*1.604^2}{2*9.81}$=0.003932m

d. Head loss for second bend.

$HL_{3(b)} = K * \frac{V^2}{2g}$

where,θ=14.03'

K=0.02

$HL_{3(b)} = \frac{0.02*1.604^2}{2*9.81}$=0.002621m

$HL_3 = HL_{3(a)} + HL_{3(b)} = 0.003932 + 0.002621 = 0.006553m$

Total head loss

HL_1=19.7254 cm

HL_2=16.3 cm

HL_3=0.6553 cm

Total=36.6807 cm

Consider 10% higher (cl.6.3, IS 7784(part2/Sec3):1996

Design head loss=40.3488 cm<42.0cm is permissible.

Hence OK.

3.2.2 Structural design of the Barrel

A Load calculation for Barrel

DATA

Nos. of Barrel = 1

Width of Barrel = 2.8 m

Depth of barrel = 2.8 m

Thickness of Top Slab = 0.5 m

Thickness of Bottom Slab = 0.5m

Thickness of Outer Side wall =0.5 m

Total width of barrel = 3.8m

Top RL of Top Slab below Bed = 96.15m

TOP RL of Top Slab Below Bank = 99.65m

Drain Bank RL = 102.41m

Drain HFL = 101m

Drainage Bed Level (DBL) = 97m

FSL = 101.51m

Thickness of Buoyancy Concrete = 0.35m

Soil Surcharge Depth Below Bed = 0.5m

Soil Surcharge Depth below Drain Bank = 3.65m

Density of concrete for Barrel = $25kN/m^3$

Density of buoyancy concrete =24 kN/m^3

Saturated Density of soil = 21 kN/m^3

Density of water = 10 kN/m^3

Lateral earth pressure coefficient = Ka = 0.297

Case (1) Drain dry and syphon full

a. **Load on top slab**

(1.)Weight of the buoyancy

=Width of barrel*thickness*density of concrete

=$3.8 * 0.35 * 24$

= 31.92 kN

(2.)Self weight of top slab

=3.8*0.5*25

=47.5 kN

(3.)Cushion load

=Width of barrel*soil surcharge below bed*density of soil

=$3.8 * 0.5 * 21$

= 39.9 kN

(4.)Bursting force

=(HWSL-soffit RL of barrel)*clear width of barrel*density of water

=(101.51-95.65)*2.8*10

=164.08 kN

Total load acting on top slab

=(31.92+47.5+39.9-164.08)

=-44.76 kN

Total downward pressure on top slab

= - 15.99 kN/m(Upward) as shown in fig.3.6

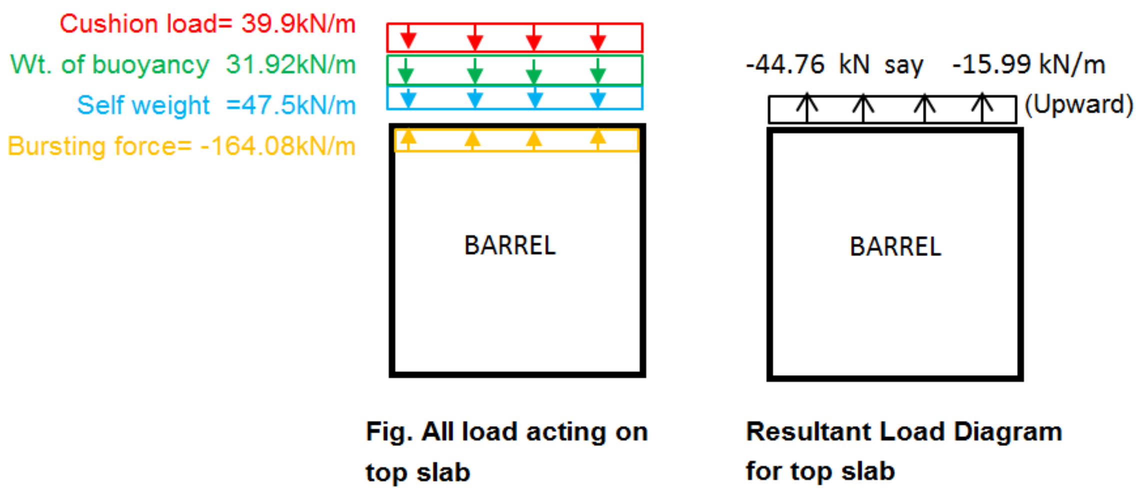


Figure 3.6: Load on top slab

b. **Load on bottom slab**

(1.)Load from top slab

=-44.76 kN

(2.)Load due to water=Weight of water in barrel

=2.8*2.8*10

= 78.4 kN

(3.)Self weight of bottom slab

=3.8*0.5*25*2

=70 kN

(4.)Weight of vertical wall

= 2.8*0.5*25*2

=70 kN

(5.)Bursting force

=(HWSL-soffit RL of barrel)*clear width of barrel*density of water

=$(101.51 - 95.65) * 2.8 * 10$

=164.08 kN

Total load acting on bottom slab

= - 44.76+78.4+47.5+70+164.08

=315.22 kN

Net soil pressure

$=\frac{(315.22-47.5-78.4-164.08)}{3.8}$

=6.64 kN/m As shown in fig.3.7

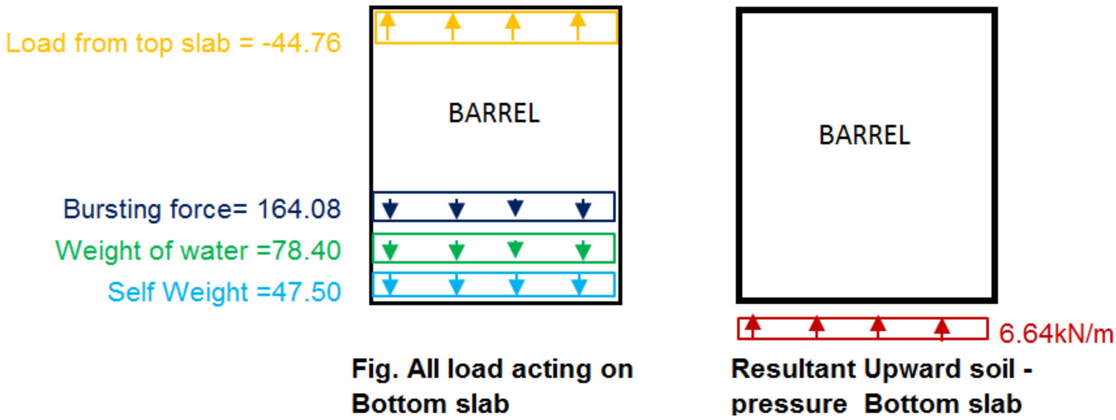


Figure 3.7: Load on Botto slab

c. **Load on side wall**

Assuming that there is no contact between outer face of barrel and earth fill.

(1.)Water pressure at bottm of top slab

=(HWSL- bottom RL of top slab)*density of water

=(101.51-95.65)*10

=58.6 kN/m(Inner to outlet)

(2.)Water pressure at top of bottom slab

=(HWSL-top of bottom slab)*density of water

=(101.51-92.85)*10

=86.6 kN/m(Inner to outlet)fig.3.8

Case (2) Drain full and Syphon dry

a. **Load on Top slab**

(1.)Weight due to buoyancy concrete

=31.92 kN

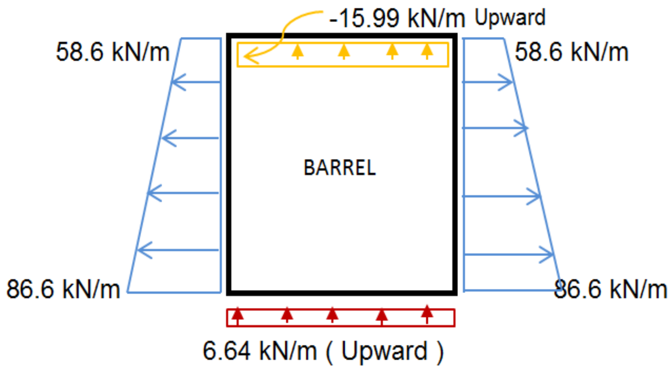


Figure 3.8: Resultant load on Whole barrel

(2.)Self weight of top slab

=47.5 kN

(3.)Soil cushion load

=39.9 kN

(4.)Weight of water in drain

=(HFL-DBL)*barrel width*Density of water

=(101-97)*3.8*10=152 kN

Total load acting on top slab

=31.92+47.5+39.9+152

=271.32 kN

Total downward pressure on top slab

=71.4 kN/m(downward)

b. **Load on bottom slab**

(1.)Load from top slab

=271.32 kN

(2.)Self weight of bottom slab

=47.5 kN

(3.)Weight of vertical load

=70 kN

Total downward pressure on top slab

=271.32+47.5+70

=388.82 kN

Net soil pressure

$=\frac{(388.82-47.5)}{3.8}$

=89.82 kN/m(upward)

c. **Load on side wall**

Assuming earth filling as under saturated condition.

(1.)Pressure at center of the top slab

Lateral earth pressure below DBL

$=K_a * \chi * h$ where h = DBL-Centre RL of bottom slab

=0.297*21*(97-95.9)

=6.86 kN/m

(2.)Lateral earth Pressure at center of the bottom slab

$=K_a * \chi * h$

=0.297*21*(97-92.6)

=27.44 kN/m

Load diagram for whole Barrel fig.3.9

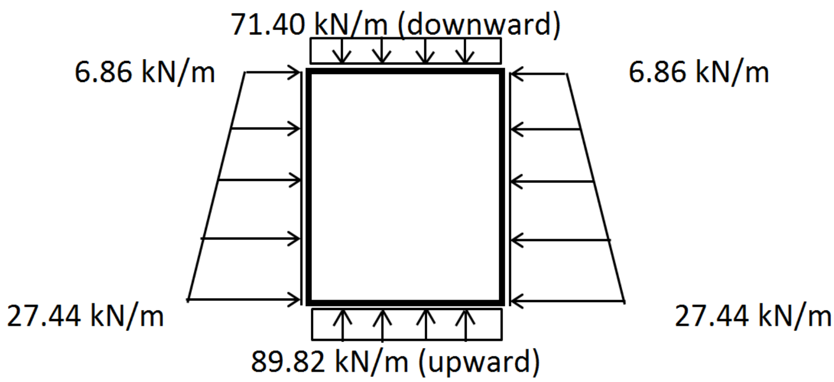


Figure 3.9: Resultant load on Whole barrel

STAAD load and results diagram

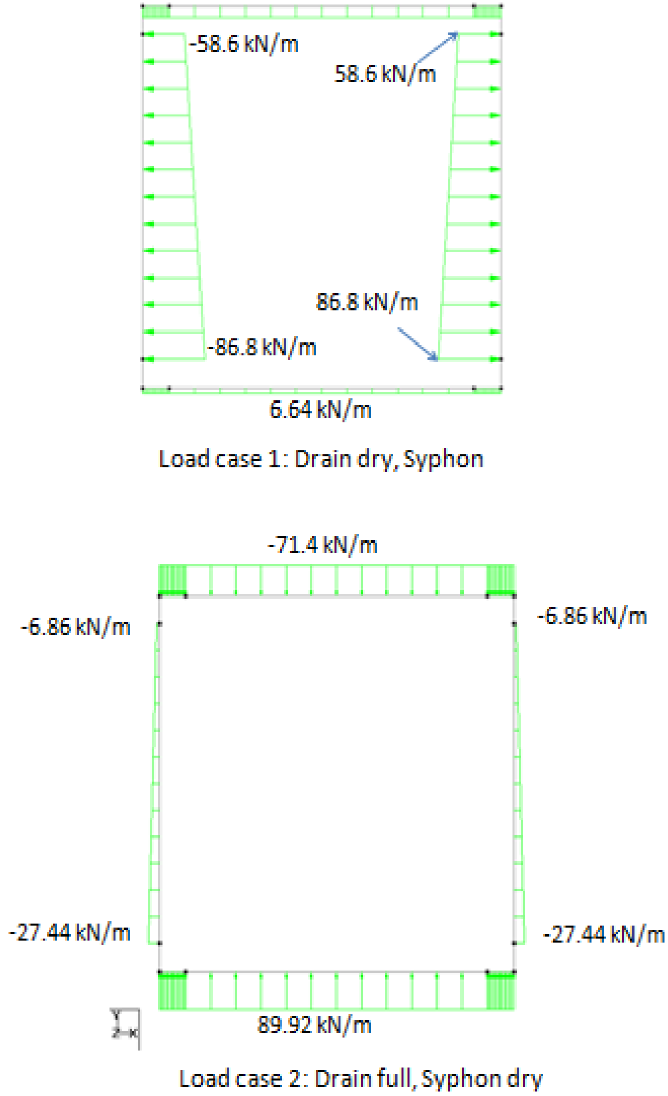


Figure 3.10: Load diagram

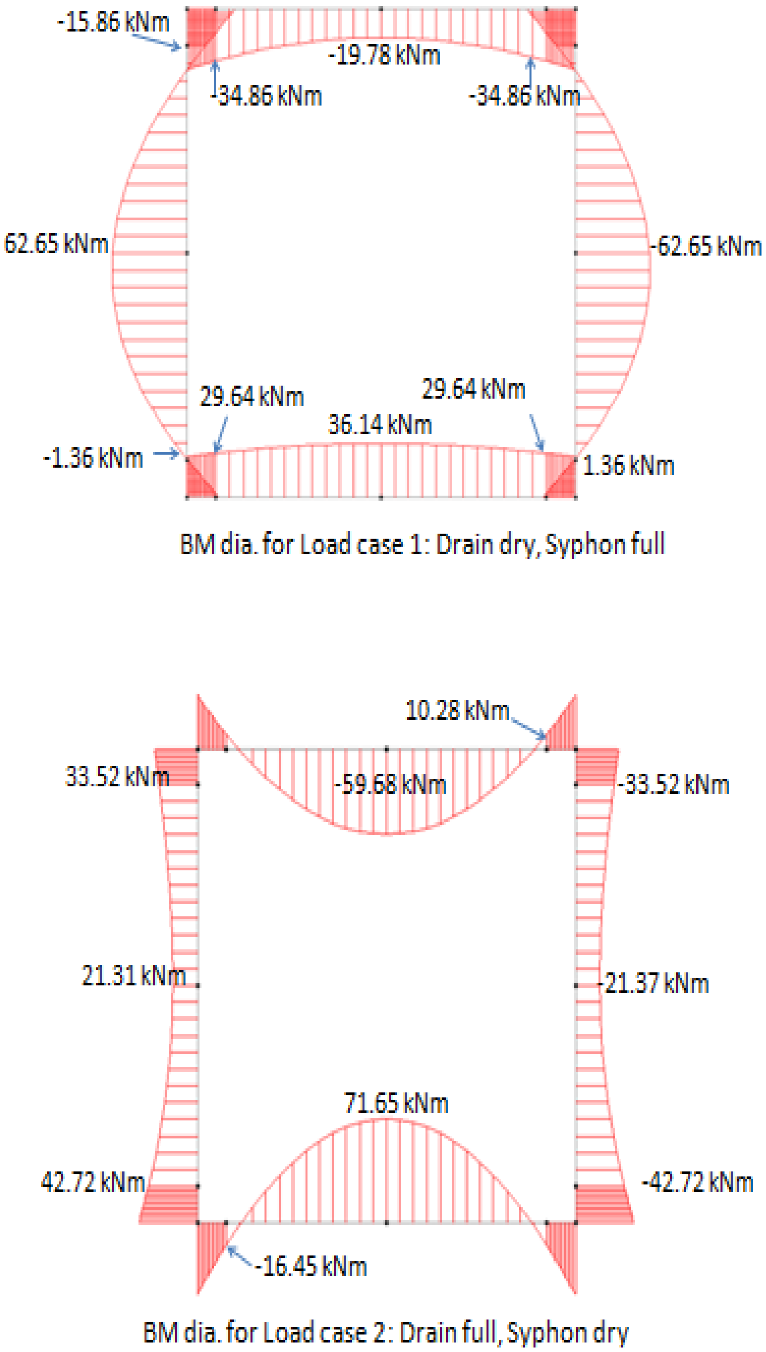


Figure 3.11: STAAD Bending Moment diagram

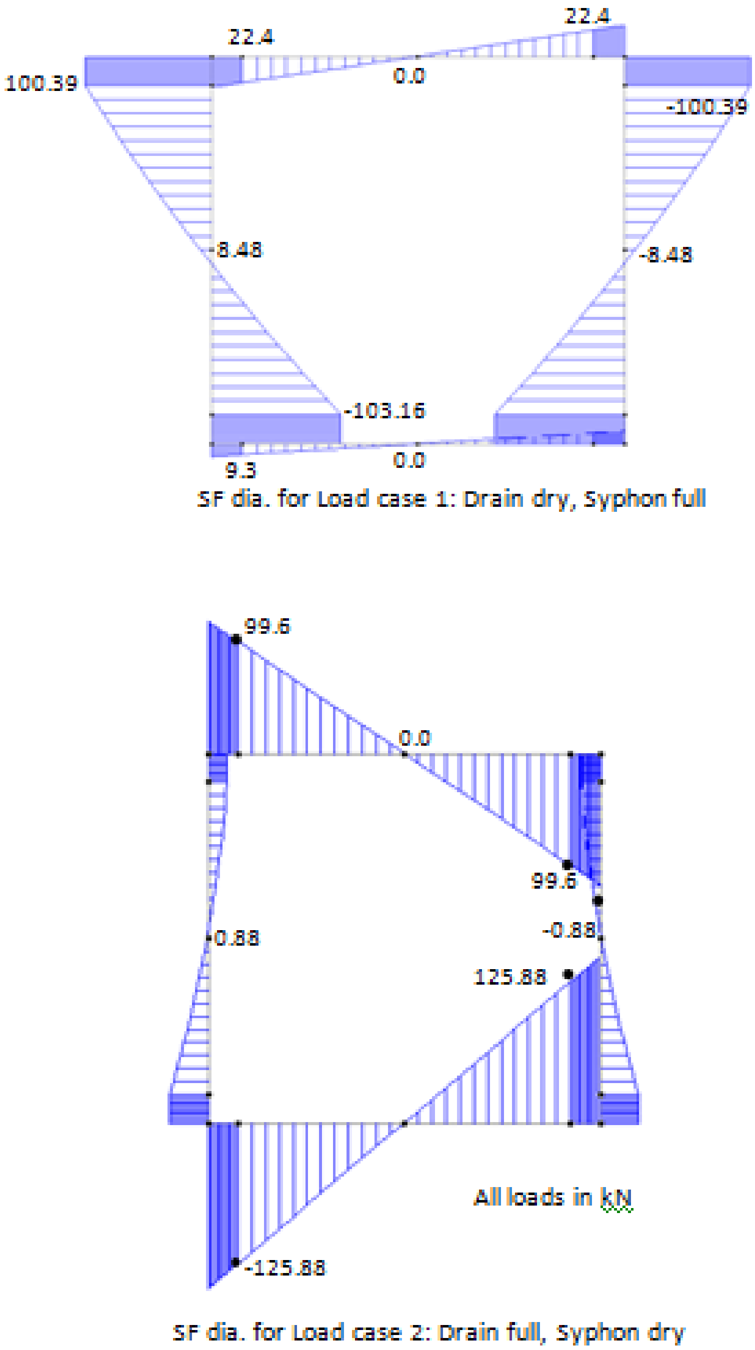


Figure 3.12: STAAD Shear force diagram

B Reinforcement calculation

Reinforcement calculation is carried out based on Combine effect of Axial tension and Bending

M = Bm ,

D = over all depth ,

d = effective depth

T = Axial tension

Eccentricity=e= M/T

x=d-D/2

(1.) If x≥e(i.e. Eccentricity inside the section)

$A_{s,@tensionface} = \frac{T}{2\sigma_{st}} + \frac{M}{2x\sigma_{st}}$

$A_{s,@oppositeface} = \frac{T}{2\sigma_{st}} - \frac{M}{2x\sigma_{st}}$

(2.) If x<e(i.e. Eccentricity out side the section)

$A_{st} = \frac{T}{\sigma_{st}} + \frac{M-Tx}{\sigma_{st}jd}$

DATA

Thickness of Top Slab=0.500 m

Thickness of Bottom Slab=0.500 m

Thickness of Outer Side wall=0.500 m

Material property

Concrete Mix Grade=M30

Main Steel Grade=Fe415

Design Constants

Permissible stress in steel (σ_{sst})=130.0 N/mm^2

Permissible stress in concrete (σ_{cbc})=10.0 N/mm^2

Modular ratio (m)=9.3333

Factor for neutral axis depth n=1 / (1 + (σ_{st}/(m*σ_{cbc})))

=0.4179

Factor of lever arm (j)=1-n/3

=0.8607

Q =1/2*σ_{cbc}*j*n

=1.8 N/mm^2

Table 3.1: Main reinforcement summary

Component	Design BM	Axial Tension	Depth Required	Depth Provided	Cover Provided	Ast required	Diameter of Bar	Spacing required	Spacing Provided	Ast Provided	Bar notation In Reinforcement schedule
	kN.m	kN	mm	mm	mm	mm²	mm	mm	mm	mm²	
Top Slab @ **Support** (Top face)	10.28	-	75.6	444.00	50	206.93	12	546.55	160	706.86	(1)
							12	2409.92	320	353.43	(2)
									Total	1060.29	
Top Slab @ **Support** (Bottom face)	34.86	100.39	139.22	442.00	50	1087.36	16	184.91	160	1256.64	(3)
									Total	1256.64	
Top Slab @ midspan (Top face)	-	-	-	444.00	50	-	12	5654867	160	706.86	(1)
									Total	706.86	
Top Slab @ Mid span (Bottom face)	59.68	-	182.16	442.00	50	1206.74	16	166.62	160	1256.64	(3)
									Total	1256.64	

Note : Details Shown in fig.3.13

. Continued..

Component	Design BM	Axial Tension	Depth Required	Depth Provided	Cover Provided	Ast required	Diameter of Bar	Spacing required	Spacing Provided	Ast Provided	Bar no. In Reinforcement schedule
	kN.m	kN	mm	mm	mm	mm²	mm	mm	mm	mm²	
Bottom Slab @ Support (Top face)	29.64	103.16	128.38	440.00	50	997.46	20	314.96	200	1570.8	(4)
									Total	1570.8	
Bottom Slab @ Support (Bottom face)	16.46	-	95.67	442.00	50	-	16	-	200	1005.31	(5)
									Total	1005.31	
Bottom Slab @ mid span (Top face)	71.65	-	199.6	440.00	50	-	20	-	200	1570.8	(4)
									Total	1570.8	
Bottom Slab @ Mid span (Bottom face)	-	-	-	442.00	50	-	16	1005310	200	1005.31	(5)
									Total	1005.31	

. Continued..

Component	Design BM	Axial Tension	Depth Required	Depth Provided	Cover Provided	Ast required	Diameter of Bar	Spacing required	Spacing Provided	Ast Provided	Bar no. In Reinforcement schedule
	KN.m	kN	mm	mm	mm	mm²	mm	mm	mm	mm²	
Side Wall @ Top end (Outer face)	33.52	-	136.52	440.00	50	680.86	20	461.42	220	1428	(6)
									Total	1428	
Side Wall @ Top end (Inner face)	15.85	26.4	93.88	442.00	50	421.07	16	477.5	220	913.92	(7)
							12	-229.48	220	514.08	(8)
									Total	1428	
Side Wall @ Mid span (Outer face)	62.65	26.4	186.64	440.00	50	1373.74	20	228.69	220	1428	(6)
									Total	1428	
Side Wall @ Mid span (Inner face)	-		-	442.00	50	-	16	-	220	913.92	(7)
									Total	913.92	
Side Wall @ Bottom end (Outer face)	42.72	-	154.12	440.00	50	-	20	362.05	220	1428	(6)
									Total	1428	
Side Wall @ Bottom end (Inner face)	1.31	26.4	26.99	442.00	50	127.78	16	1573.5	220	913.92	(7)
							12	-1226.39	220	514.08	(8)
									Total	1428	

Table 3.2: Shear reinforcement summary

Component	Design SF	Nominal Shear Stress (t_v)	Permissible Shear Stress (t_c)	Design Shear	No. of Legs	Diameter of Steel	Spacing required	Spacing Provided
	KN	(N/mm²)	(N/mm²)	KN		mm	mm	mm
Top Slab @ Support	99.96	0.2251	0.2266	0.00	-	-	N.A.	-
Top Slab @ Mid Point	0.00	0.0000	0.2410	0.00	-	-	N.A.	-
Bottom Slab @ Support	125.88	0.2835	0.2232	27.66	2	10	324.80	300
Bottom Slab @ Mid Point	0.00	0.0000	0.2642	0.00	-	-	N.A.	-
Side Wall @ Top end	100.00	0.2273	0.2534	0.00	-	-	N.A.	-
Side Wall @ Mid Point	8.48	0.0193	0.2538	0.00	-	-	N.A.	-
Side Wall @ Bottom end	103.16	0.2345	0.2534	0.00	-	-	N.A.	-

Note : Details Shown in fig.3.13

Table 3.3: Distribution reinforcement summary

Distribution Reinforcement Summary Table (At water side)

Component	Thickness of strip	Ast required	Diameter of Steel	Spacing required	Spacing Provided	Ast Provided	Bar No.
	m	mm²	mm	mm	mm	mm²	
TOP SLAB	0.250 m	875	12	129.25	130	869.98	(10)
BOTTOM SLAB	0.250 m	875	12	129.25	130	869.98	(10)
SIDE WALL	0.250 m	875	12	129.25	130	869.98	(10)

Distribution Reinforcement Summary Table (At Earth side)

Component	Thickness of Strip	Ast required	Diameter of Steel	Spacing required	Spacing Provided	Ast Provided	Bar No.
	m	mm²	mm	mm	mm	mm²	
TOP SLAB	0.100 m	350	10	224.4	200	392.70	(9)
BOTTOM SLAB	0.100 m	350	10	224.4	200	392.70	(9)
OUTERSIDE WALL	0.100 m	350	10	224.4	200	392.70	(9)

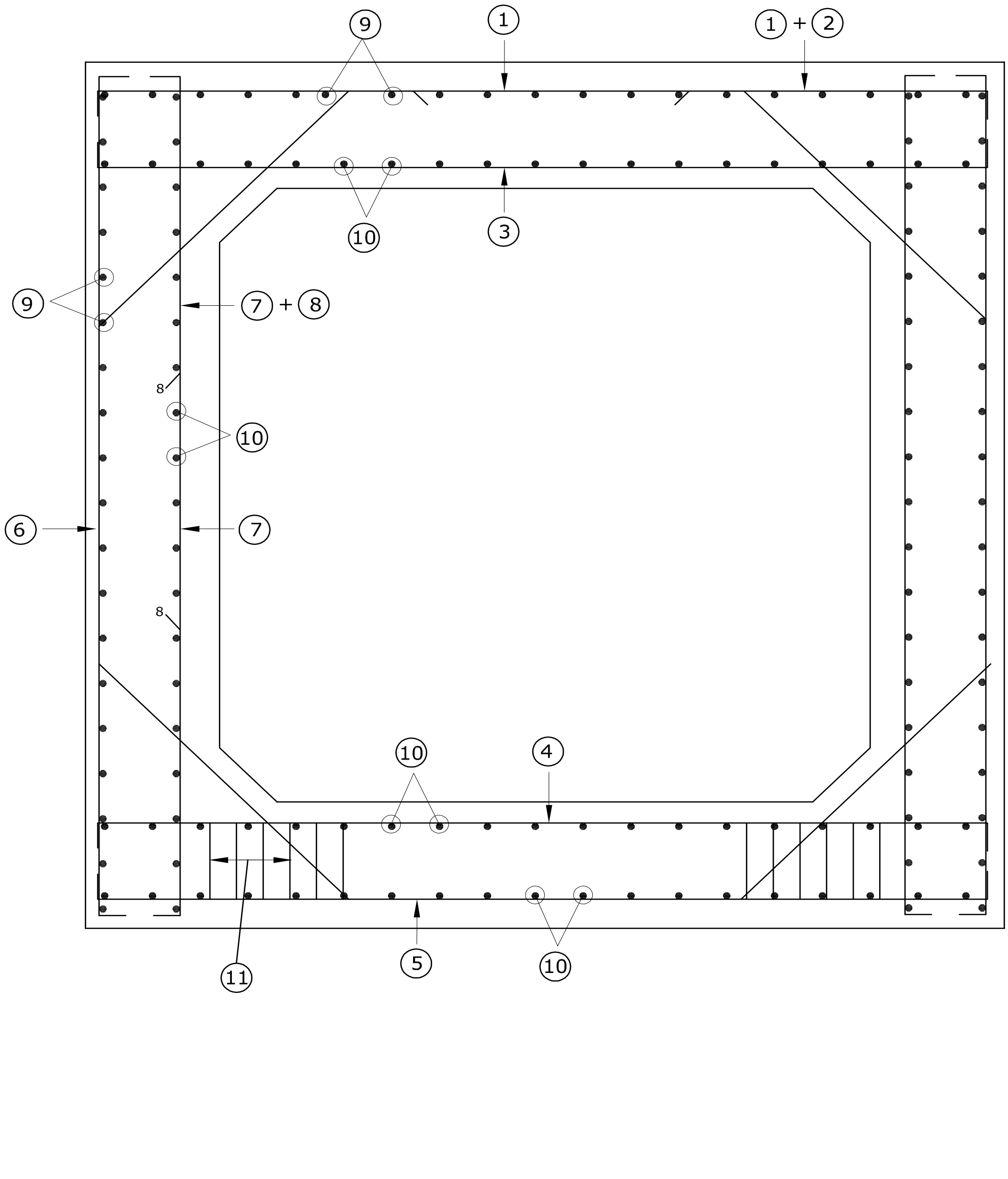


Figure 3.13: Reinforcement detail for Barrel of syphon

Table 3.4: Bar bending schedule for Barrel of syphon

Bar Notation	Shape of Bar	Dia of bar (mm)	Spacing of bar (mm)	No.bars for 1 m length	Weight for 1 m Length kg
1	0.2 3.70m 0.2	12	160	6	22.73
2	0.2 3.6 m 0.2	12	320	3	9.86
3	0.2 3.7m 0.2	16	160	6	40.42
4	0.3 3.7 m **	20	200	5	51.76
5	0.2 3.6m 0.2	16	200	5	31.55
6	0.4m 3.7m 0.4m	20	220	5	50.42
7	0.3m 3.6m 0.3m	16	220	5	30.12
8	0.15m 0.9m	12	220	4	4.24
9	15	10	220	69	42.58
10		12	130	86	76.45
11	0.4m	10 mm 2 leg @300 mm c/c		10	7.4
Total weight of steel for one span					367

Note : Above value shows quantity for 1m length of syphon.

3.3 Analysis and Design Circular type Canal syphon

3.3.1 Hydraulic Design of circular syphon

[A.]Fixing the size of barrel

Q_d=12.5cumecs

Design Velocity V_d=1.36m/sec

V=1.605 m/sec

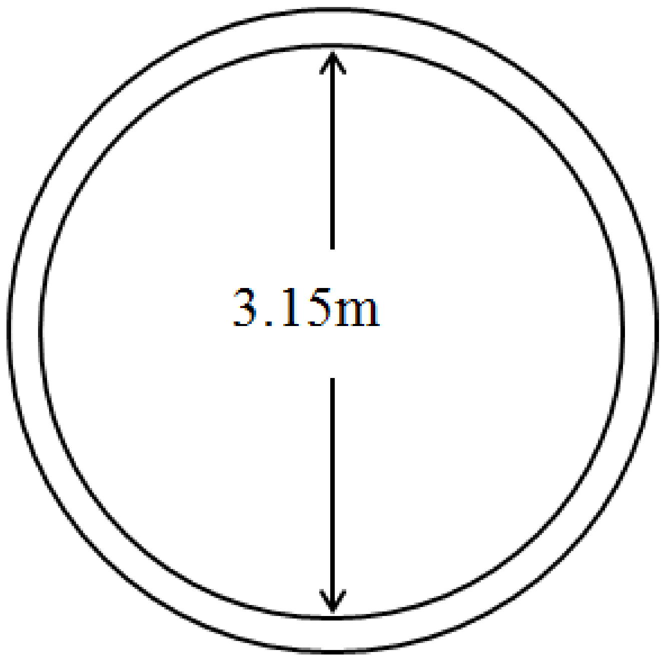


Figure 3.14: Cross section of Circular syphon

Now Area of water way,A=[3.14*9.92/4]

. =7.79 m^2

Perimeter,P=9.89 m

Now,R=$\frac{A}{P}$

R=$\frac{7.79}{9.89}$

=0.788

Velocity in syphon,

V=$\frac{Q_d}{A}$

V=$\frac{12.50}{7.79}$

=1.605 m/sec

>Design velocity

Hence O.K.

[B.]Head Loss Calculation for canal syphon

(1)Loss due to Entry and Exit

$HL_1 = (1 + F_1)\frac{V_2^2}{2g}$

F_1=0.505

=(1+ 0.505)$\frac{1.605^2}{2*9.81}$

=0.19m

(2)Loss due to Friction in Syphon Barrel

Considering n=0.018 for concrete

S_f=Friction slope in Barrel

$S_f = \frac{V^2 * n^2}{R^{\frac{4}{3}}}$

where V=Velocity in Barrel

R=Hydraulic man radius

Now R=$\frac{A}{P}$

R=$\frac{7.79}{9.89}$

=0.788

V=$\frac{Q}{A} = \frac{12.5}{7.79}$=1.605 m/sec

$S_f = \frac{1.605^2 * 0.018^2}{0.788^{\frac{4}{3}}}$

=0.00114

$HL_2 = S_f * L_b$

L_b=width of drain=125.63

=0.144m

(3)Loss due to of bends of syphon

(a) For inlet bend, $\tan\theta$=0.333, θ=18.43

$HL_{3(a)} = K * \frac{V^2}{2g}$

where K is taken from the graph from

IS:2951(Part-II)1965

K=0.028

=0.028*$\frac{1.605^2}{2*9.81}$

=0.0036 m

(b) Head Loss for second bend:-

$HL_{3(b)} = K * \frac{V^2}{2g}$

θ=14.03 K = 0.017

=0.017*$\frac{1.605^2}{2*9.81}$

=0.00223m

$HL_3 = HL_{3(a)} + HL_{3(b)}$

=0.00367 + 0.00223

=0.0059 m

TOTAL HEAD LOSS:-

HL_1=19.75cm

HL_2=14.40cm

HL_3=0.59cm

Total=34.74cm

Consider10% higher

Design Head loss=38.22cm<42.00cm permissible

Hence O.K.

3.3.2 Structural Design of circular syphon

FSL=101.51 m

Center RL of syphon=94.275 m

H=101.51-94.275=7.235 m

Hoop tension=$\frac{\gamma * wHD}{2} = \frac{10*7.235*3.15}{2}$

= 113.95 kN

Area of hoop steel = T/σ_{st}=113.95*1000/130

= 876.53 mm^2

Provide 12 tor @ 120 mm c/c

A_{st} provided =942 mm^2

Thickness of barrel(t)

Tensile stress σ_{ct}=T/(1000t+(m-1)A_{st})

1.5 = 113.95*1000/(1000t+(9.33-1)942)

Hence, t=67 mm

Provide 200 mm thick ness of barrel and 12tor @ 120 mm c/c

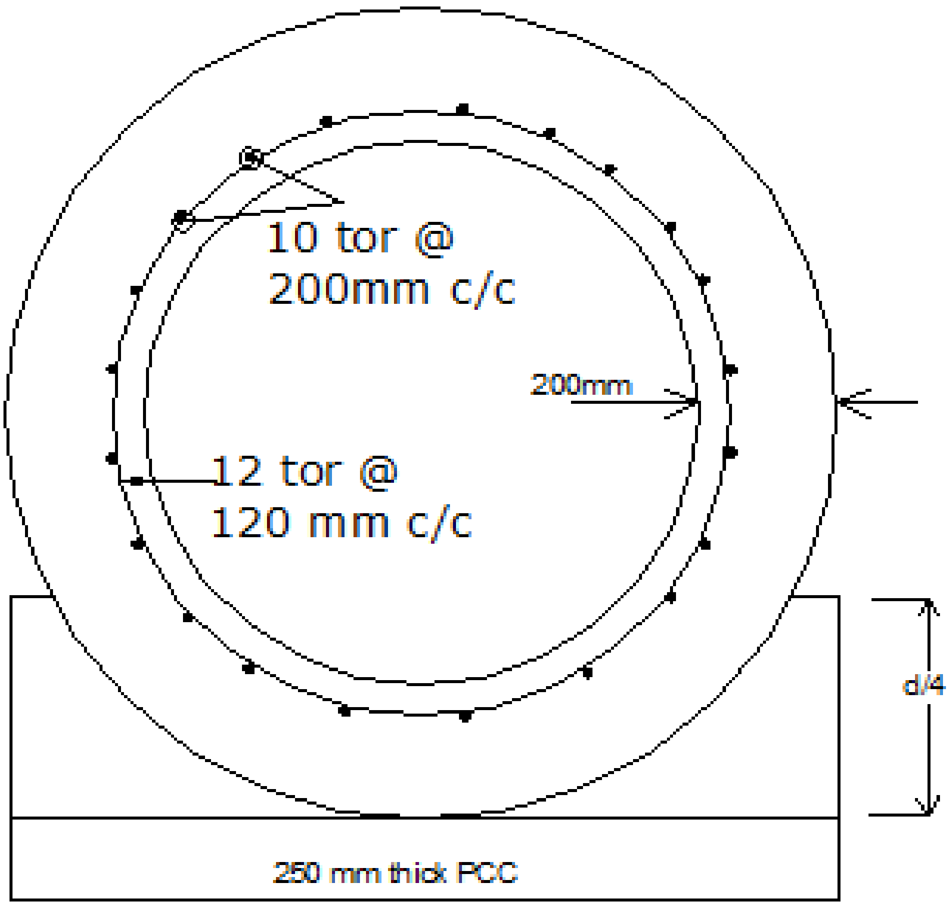


Figure 3.15: Reinforcement detail of circular syphon

Chapter 4

Analysis and design of Aqueduct

4.1 General

As shown in fig.4.1 Canal and natural drain intersect each other at 90'.Canal having bed -width of 6m and carries 20 cumecs discharge. Width of Drain is 90 m and is 98m and carries 400 cumecs discharge.

As shown in figure canal bed level(C.B.L.) is higher than the highest food level (H.F.L.) of the drain. Since, sufficient vertical clearance between C.B.L. and D.B.L.available, Aqueduct type Cross drainage work is feasible.

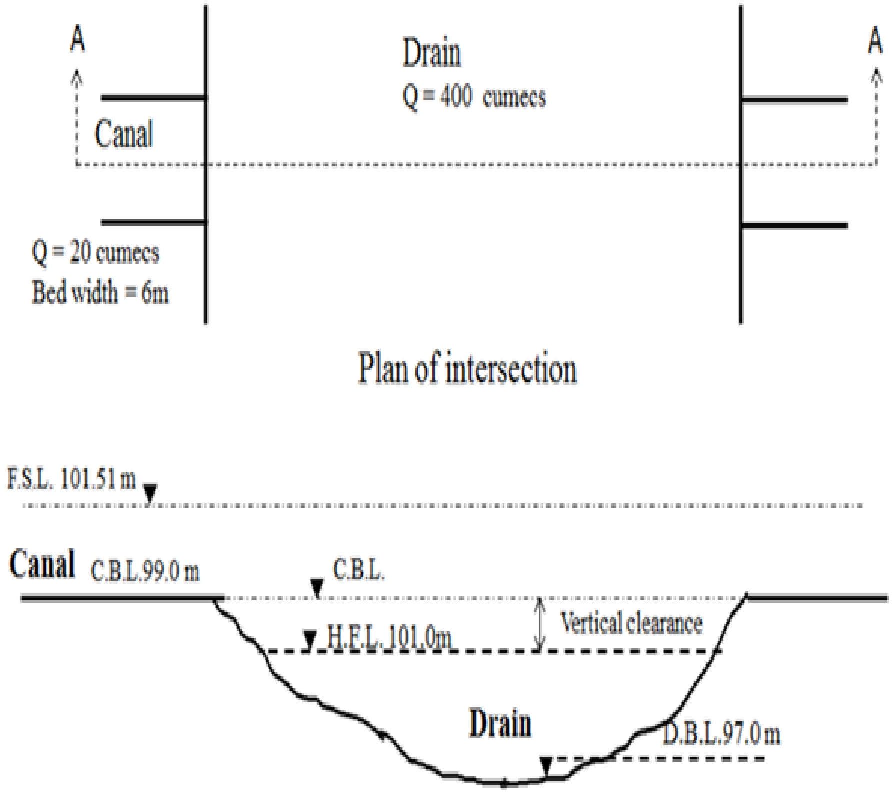


Figure 4.1: Intersection of Canal and Drain

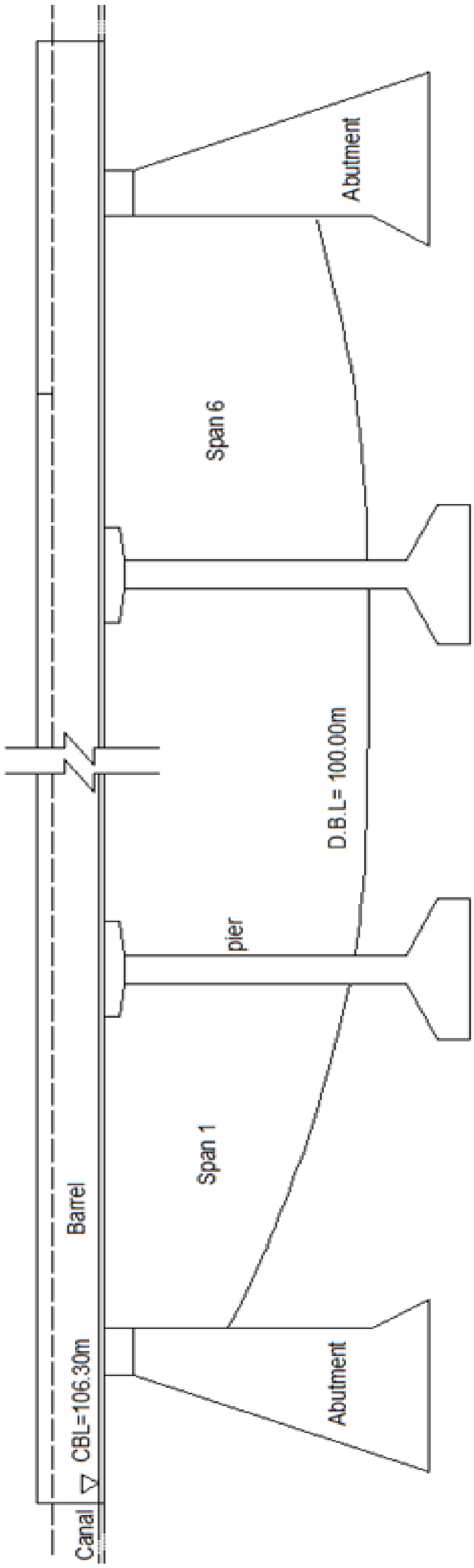


Figure 4.2: Longitudinal section of Aqueduct

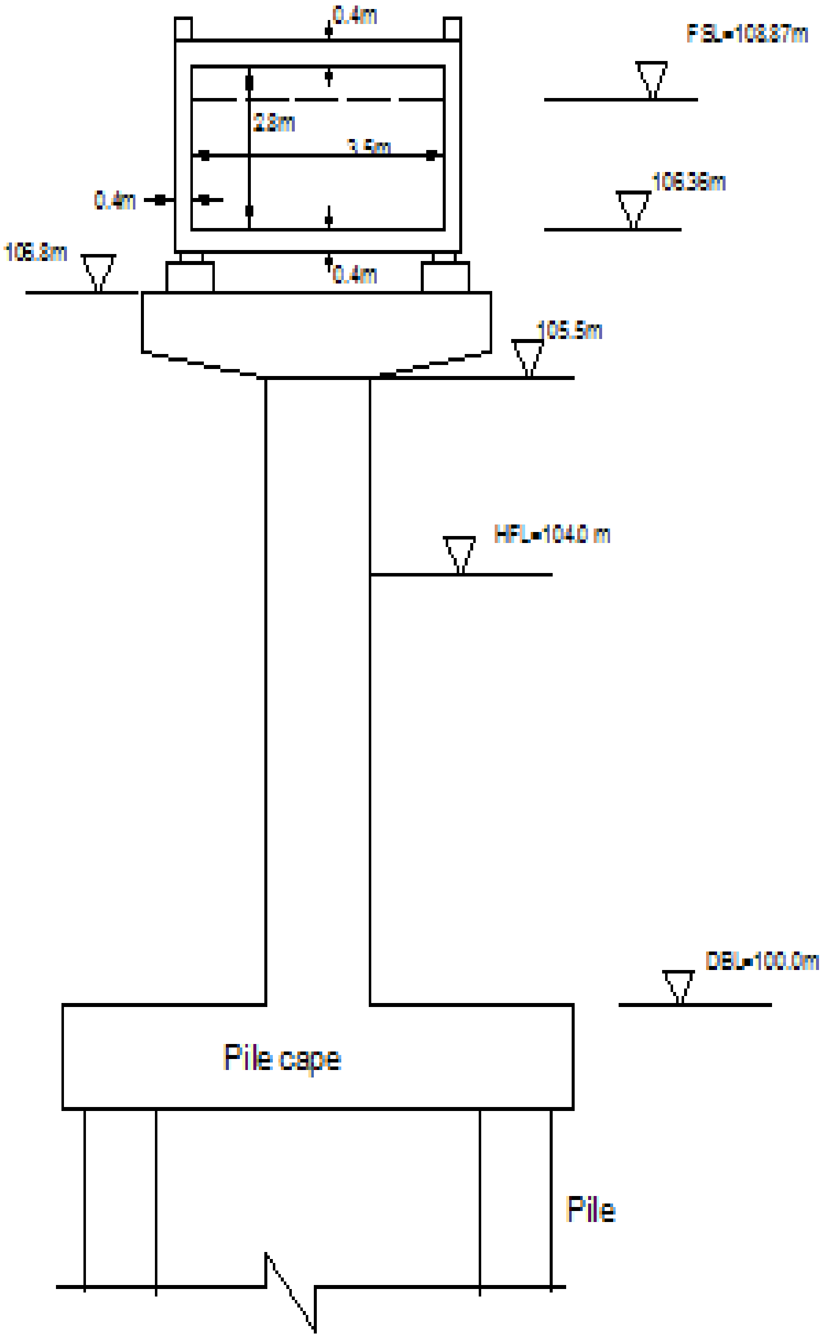


Figure 4.3: Cross section of Aqueduct

4.2 Analysis and design of Super structure

4.2.1 Determination of size of barrel and head loss calculation

Data

Discharge of canal=20m^3/s

Velocity in canal =1.2m/s

Depth of water in canal =2.5m

n =0.018

Length of trough =90m

Permissible Head loss = 0.3m

(A) Determination of size of barrel

Depth of water in trough = Depth of water in canal = 2.5m

Let provide trough width =3.5m

So, velocity in trough = $Q/A = 2.28m/s$

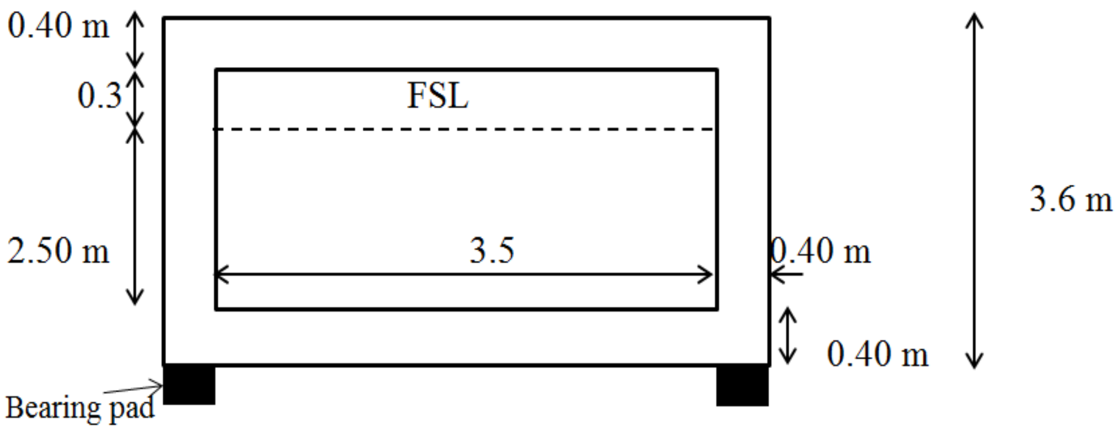


Figure 4.4: Cross section of barrel of aqueduct

(B) Head loss calcalation

(1)Head loss due to inlet transition(h_1)

$$\begin{aligned} h1 &= 0.3 * \frac{(V_1^2 - V_2^2)}{2g} \\ &= 0.3 * \frac{(2.286^2 - 1.2^2)}{2g} \\ &= 0.057m \end{aligned}$$

(2)Head loss due to Skin friction (h_2)

Manning's formula

$$\begin{aligned} v &= \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \\ Hence, 2.28 &= \frac{1.029^2/3 * S^1/2}{0.018} \\ S &= 0.016 \end{aligned}$$

Where,

S=slope

R=hydraulic mean radius in m,

$R = \frac{3.5*2.5}{3.5+(2*2.5)}$

R=1.029

v=2.28

Loss of head in the through=Length of trough*slope

h_2= 0.146m

(3)Head loss due to outlet transition(h_3)

$$h3 = 0.4\frac{(V_1^2 - V_2^2)}{2g}$$

$$= 0.4\frac{(2.286^2 - 1.200^2)}{2g}$$
$$= 0.077$$

$$TotalHeadLoss = h_1 + h_2 + h_3$$
$$= 0.28m < 0.3m$$
$$Hence, OK$$

4.2.2 Transverse analysis and design of super structure

(1.)Loads acting on through(Barrel)

-Weight of water in through

-Vehicle load(Live load)on top slab

-Self weight of trough

(2.)Load combinations

-Barrel empty and Live load

-Barrel full and live load

(A) Load Calculation

(a)Weight of water

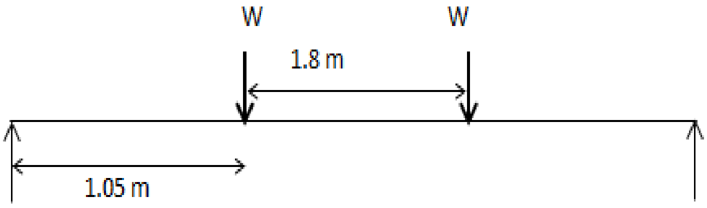
$$W_w = Depthofwater * 1m * 1m * \delta_w$$
$$= 2.5 * 1 * 1 * 10$$
$$= 25kN/m$$

(b)Live load calculation

IRC:Class A vehicle load considered as live load.For maximum bending moment vehicle placed at center span of top slab.

The effective width of dispersion in transverse direction (i.e.,In direction perpendicular to direction of movement of vehicle.)

$$b_{eff} = k_x * (1 - \frac{x}{l_{eff}}) + C_l$$



Where,

k from Table, k=2.48

x=Distance of concentrated load from nearer support

=1.05m

l_{eff}=Effective span=3.9m

C_l=Width of concentrated load parallel to the supported edge

=Width of tyre+(2*thickness of wearing force)

=250mm+(2*80mm)

=0.41m

$b_{eff} = 2.48 * 1.05 * (1 - \frac{1.05}{3.9}) + 0.41$

=2.31m

Which is effective width due to one wheel.

Combined effective width for both wheels

Total combined effective width=1.05+1.8+1.05=3.9m

Dispersion in longitudinal direction

(i.e.,In direction parallel to movement of vehicle.)

Dispersion through single wheel=Width of wheel+2*(thickness of slab+thickness of wearing coat)

=0.25+2*(0.4+0.08)

=1.21m

So,Combine dispersion length.

Total dispersion length=$\frac{1.21}{2} + 1.2 + \frac{1.21}{2}$

=2.41m

Impact factor for class-A loading

IF=$\frac{4.5}{6+L}$ =$\frac{4.5}{6+3.9}$

=0.45

Maximum Axial load for Class-A load=114 kN

UDL in transverse direction

=$\frac{(1+0.45)*114}{3.9*2.41}$

=17.58 kN/m

(B) Reinforcement Calculation

DATA

Thickness of Top Slab=0.4 m

Thickness of Bottom Slab=0.4 m

Thickness of Outer Side wall=0.4 m

Material property

Concrete Mix Grade=M30

Main Steel Grade=Fe415

Design Constants - Normal condition

Permissible stress in steel (σ_{sst})=130.0 N/mm^2

Permissible stress in concrete (σ_{cbc})=10.0 N/mm^2

Modular ratio (m)=9.3333

Factor for neutral axis depth n=1 / (1 + (σ_{st}/(m*σ_{cbc})))

=0.418

Factor of lever arm (j)=1-n/3

=0.861

Q =1/2*σ_{cbc}*j*n

=1.8 N/mm^2

Design Constants - Seismic condition

Permissible stress in steel (σ_{sst})=179.2 N/mm^2

Permissible stress in concrete (σ_{cbc})=13.3 N/mm^2

Modular ratio (m)=7.017

Factor for neutral axis depth n=1 / (1 + (σ_{st}/(m*σ_{cbc})))

=0.351

Factor of lever arm (j)=1-n/3

=0.883

Q =1/2*σ_{cbc}*j*n

=2.1 N/mm^2

$d_{req} = \sqrt{\frac{M}{Q*b}}$

$A_{st} = \frac{BM}{\sigma_{sst}jd}$

Table 4.1: Main reinforcement summary table (Normal condition)

Component	Design BM	Depth Required	Depth Provided	Cover Provided	Ast required	Diameter of Steel	Spacing required	Spacing Provided	Ast Provided	Nomenclature In Reinforcement schedule
	KN.m	mm	mm	mm	mm^2	mm	mm	mm	mm^2	
Top Slab @ Support (Top face)	48.7	164.56	342.00	50	1272.65	16	157.99	280	718.08	1 + 2
						16	362.55	280	718.08	
								Total	1436.16	
Top Slab @ **Mid span** (Bottom face)	54.13	173.49	340.00	50	1422.87	20	220.79	220	1428	3
								Total	1428	
Bottom Slab @ Support (top face)	37.42	144.24	344.00	50	972.19	12	116.33	160	706.86	4 + 6
						12	426.25	320	353.43	
								Total	1060.29	
Bottom Slab @ mid span (bottom face)	29.12	127.25	342.00	50	760.98	16	264.21	260	773.32	7
								Total	773.32	
Side wall @ Top end	48.7	164.56	342.00	50	1272.65	16	157.99	280	718.08	1 + 2
						16	362.55	280	718.08	
								Total	1436.16	
Side wall @ Mid span	14.75	90.56	342.00	50	385.45	16	521.63	280	718.08	2
								Total	718.08	
Side wall @ Bottom end	37.42	144.24	344.00	50	972.19	12	116.33	160	706.86	5 + 6
						12	426.25	320	353.43	
								Total	1060.29	

Note : Details are Shown in fig.4.9

Table 4.2: Main reinforcement summary table (Seismic condition)

Component	Design BM	Depth Required	Depth Provided	Cover Provided	Ast required	Diameter of Steel	Spacing required	Spacing Provided	Ast Provided	Bar no. In Reiforcement schedule
	KN.m	mm	mm	mm	mm^2	mm	mm	mm	mm^2	
Top Slab @ Support (Top face)	48.7	164.56	342.00	50	932.56	16	215.6	140	1436.16	1 + 2
								Total	1436.16	
Top Slab @ **Mid span** (Bottom face)	50.9	168.23	340.00	50	980.42	20	320.43	220	1428	3
								Total	1428	
Bottom Slab @ Support (top face)	37.42	144.24	342.00	50	716.56	16	280.59	200	1005.31	4 + 6
								Total	1005.31	
Bottom Slab @ mid span (bottom face)	29.12	127.25	342.00	50	557.62	16	360.57	260	773.32	7
								Total	773.32	
Side wall @ Top end	48.7	164.56	342.00	50	932.56	16	215.6	280	718.08	1 + 2
						16	937.44	280	718.08	
								Total	1436.16	
Side wall @ Mid span	7.02	62.48	342.00	50	134.43	16	1495.66	280	718.08	2
								Total	718.08	
Side wall @ Bottom end	37.42	144.24	342.00	50	716.56	16	280.59	200	1005.31	5 + 6
								Total	1005.31	

Note : Details are Shown in fig.4.9

Table 4.3: Distribution reinforcement summary

Component	Thickness of Member	Thickness of strip	A_{st} required	Diameter of Steel	Spacing required	Spacing Provided	A_{st} Provided	Bar no.
			mm^2	mm	mm			
TOP SLAB	0.400 m	0.200 m	700	12	161.57	160	706.86	8
BOTTOM	0.400 m	0.200 m	700	12	161.57	160	706.86	8
OUTERSIDE	0.400 m	0.200 m	700	12	161.57	160	706.86	8

4.2.3 Longitudinal analysis and design of superstructure

DATA

Depth of water in trough=2.5m

Thickness of top slab=0.4m

Thickness of bottom slab=0.4m

Thickness of side wall=0.4m

Width of side wall=0.4m

Horizontal seismic co-efficient,α_h=0.12

Span of beam=15m

f_y=415 N/mm^2

f_{ck}=30 MPa

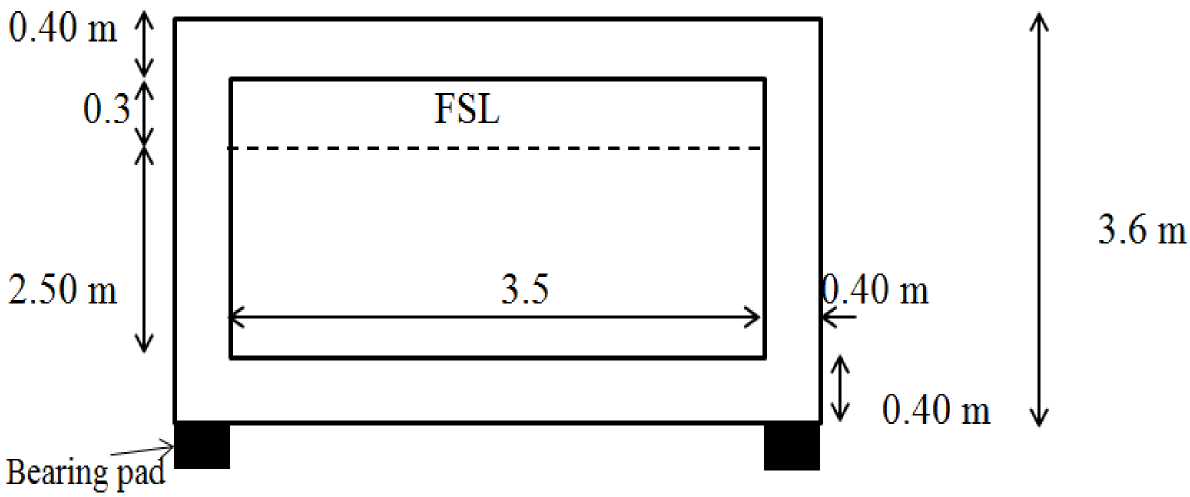


Figure 4.5: Barrel (Trough)

(A) Dead load calculation

Self weight of trough per meter length

(a) Self weight of top slab

=3.5*0.4*25

=35.00 kN

(b) Self weight of bottom slab

=3.5*0.4*25

=35.00 kN

(c) Self weight of side wall

=2*(3.6*0.4*25)

=72.00 kN

(d)Weight of water in trough

=3.5*2.5*10

=87.50 kN

Total weight per meter length=229.50 kN

Design of side wall

Side walls are designed as a beam.

UDL on beam due to daed load $=\frac{229.5}{2}$

. =114.75 kN/m

Moment due dead load(M_D)

$=\frac{w*l^2}{8}$

=3227.34 kNm

(B) Live load calculation

Live load calculation is carried out by using STAAD Pro.

Longitudinal design

To get maximum reaction on beam due to live load, Live load is placed 0.15m away from kerb.

When LL is positioned nearer to the kerb, it gives maximum bending moment for beam.

When LL is positioned nearer to the kerb due to eccentricity, the loads shared by each girder is different.This is calculated by Courbon's theory by a reaction factor given by R_x as shown in fig.4.6

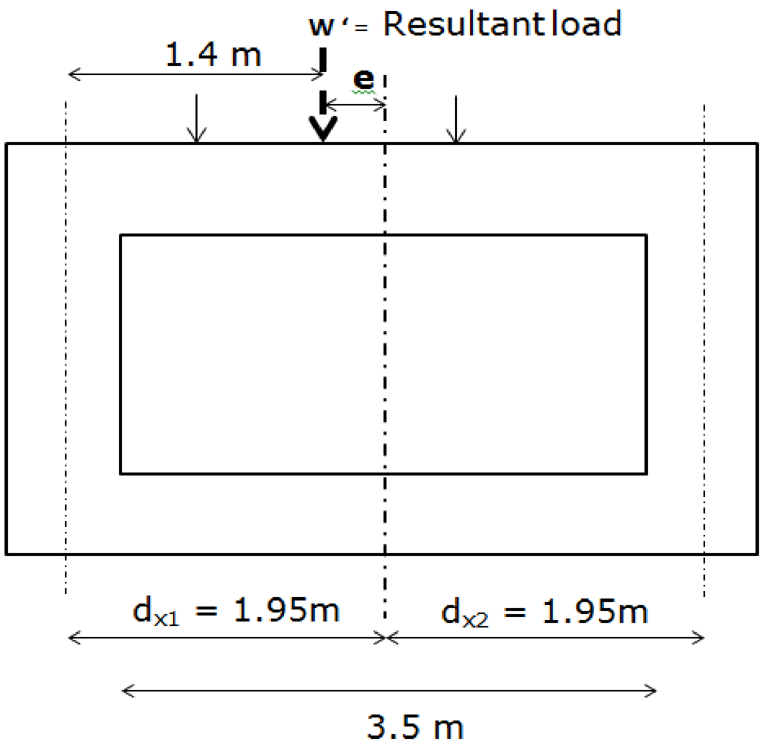


Figure 4.6: Reaction factor as per Courbon's theory

$$R_x = \frac{\sum W}{n}[1 + (\frac{\sum I}{\sum d_x^2} * d_x e)]$$

Where,

R_x=Reaction factor for the girder under consideration

I=moment of inertia of each longitudinal girder

$=\frac{1}{12} * 0.4 * 3.6^3$

$=1.55\ m^4$

d_x=Distance of the girder under consideration from the central axis of the bridge

d_{x1}=d_{x2}=1.95m

W=Total concentrated live load

n=numbers of girders=2

e=Eccentricity of Live load=0.55m

Reaction factor for girder '1',

$$R_1 = \frac{\sum W}{2}[1 + (\frac{1.55 * 2}{2 * 1.95^2 * 1.55} * 1.95 * 0.55)]$$
$$R_1 = 0.64 * \sum W$$

$\sum W$=Total load of one axel.

Impact factor IRC-Class A-load

$$\begin{aligned} IF &= \frac{4.5}{6 + L} \\ &= \frac{4.5}{6 + 15} \\ &= 0.21 \end{aligned}$$

Total BM due to live load=971 kNm as shown in fig.4.7

Bending moment due to live load on girder (1),with reaction factor and Impact factor

=971*0.64*1.21

=752 kNm

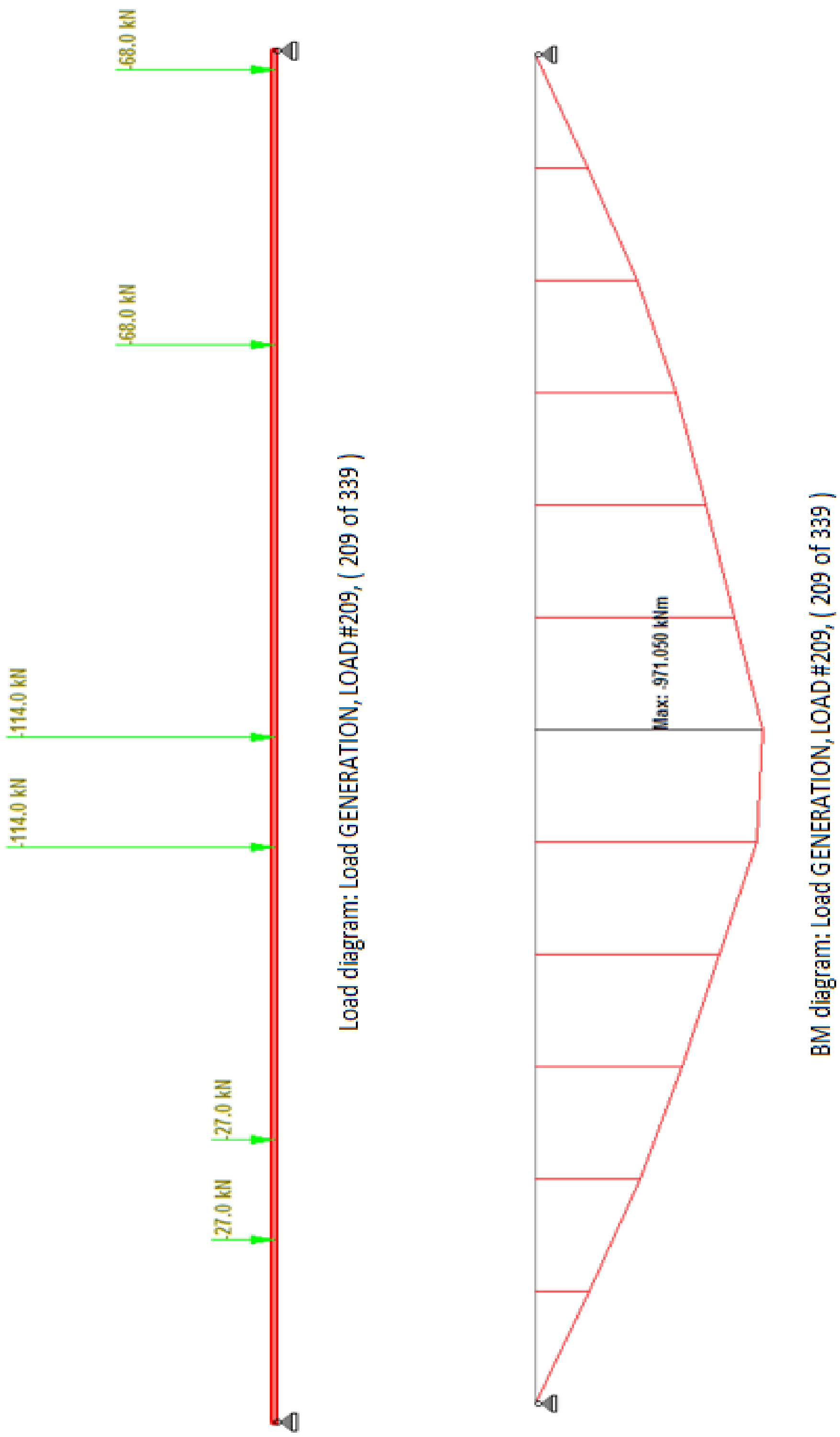


Figure 4.7: Staad load and Bending Moment diagram

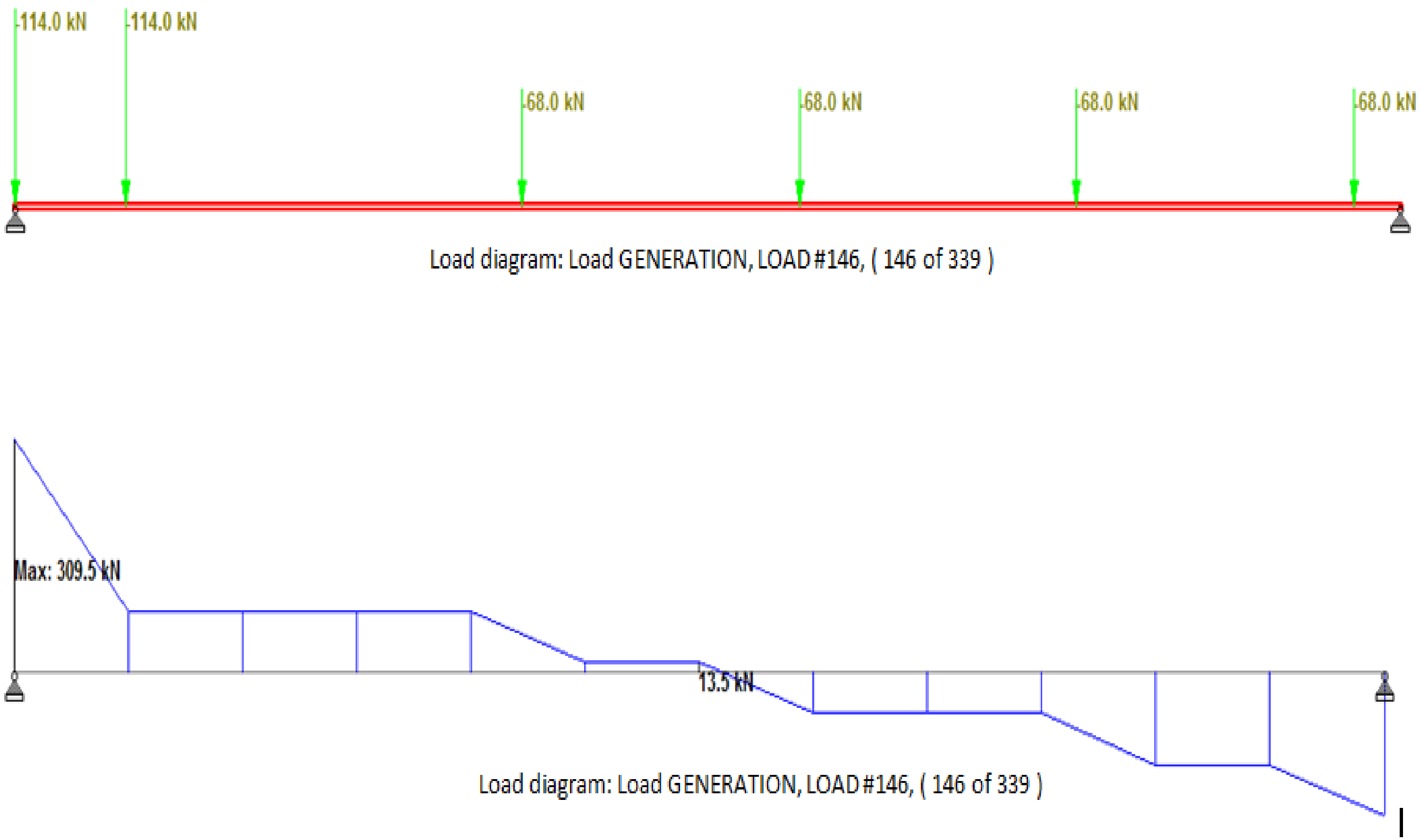


Figure 4.8: Staad load and Shear force diagram

$$\begin{aligned} M_{total} &= M_D + M_L \\ &= 3227.34 + 752 \\ &= 3979.34kNm \\ A_{st} &= \frac{M_{total}}{\sigma_{st} jd} \\ &= \frac{3979.34 * 10^6}{230 * 0.90 * 3440} \\ &= 5563.6mm^2 \end{aligned}$$

Provide 7 No.s of 32mm diameter bars.

$A_{stprovided}$=5626.9 mm^2

$p_{t,provided}$=0.41

Out of 7 bars, 3 bars are curtailed at $L/4$ distance from each support,

So,$A_{stprovided}$ at support=2813.44 mm^2

$p_{t,provided}$=0.2

Design for shear

Shear due to dead load

=860.63 kN

Shear due to live load

=239.7kN

Total shear

=1100.53 kN

$\tau_u = \frac{V_u}{bd}$

$=\frac{1100.325*1000}{0.4*3.44}$

=799.7 kN/m^2

=0.8 N/mm^2

For,p_t=0.2 ,τ_c=0.33 MPa

$\tau_v > \tau_c$

Hence, Design of shear reinforcement is required.

$$S_v = \frac{0.87 * f_y A_{sv} d}{(\tau_v - \tau_c) b d}$$

Assume, 10 mm dia. bar with 2-legged stirrups.

$$S_v = \frac{361.05 * 157 * 3440}{(0.8 - 0.33) * 400 * 3440} = 301.5mm$$

Minimum shear reinforcement for beam

$$S_v = \frac{0.87 * f_y A_{sv}}{0.4b}$$

Assume, 10 mm dia. bar with 2-legged stirrups.

$$S_v = \frac{361.05 * 157}{0.4 * 400} = 354.3mm$$

Provide, 10mm bar-2-legged stirrups with 300mm c/c

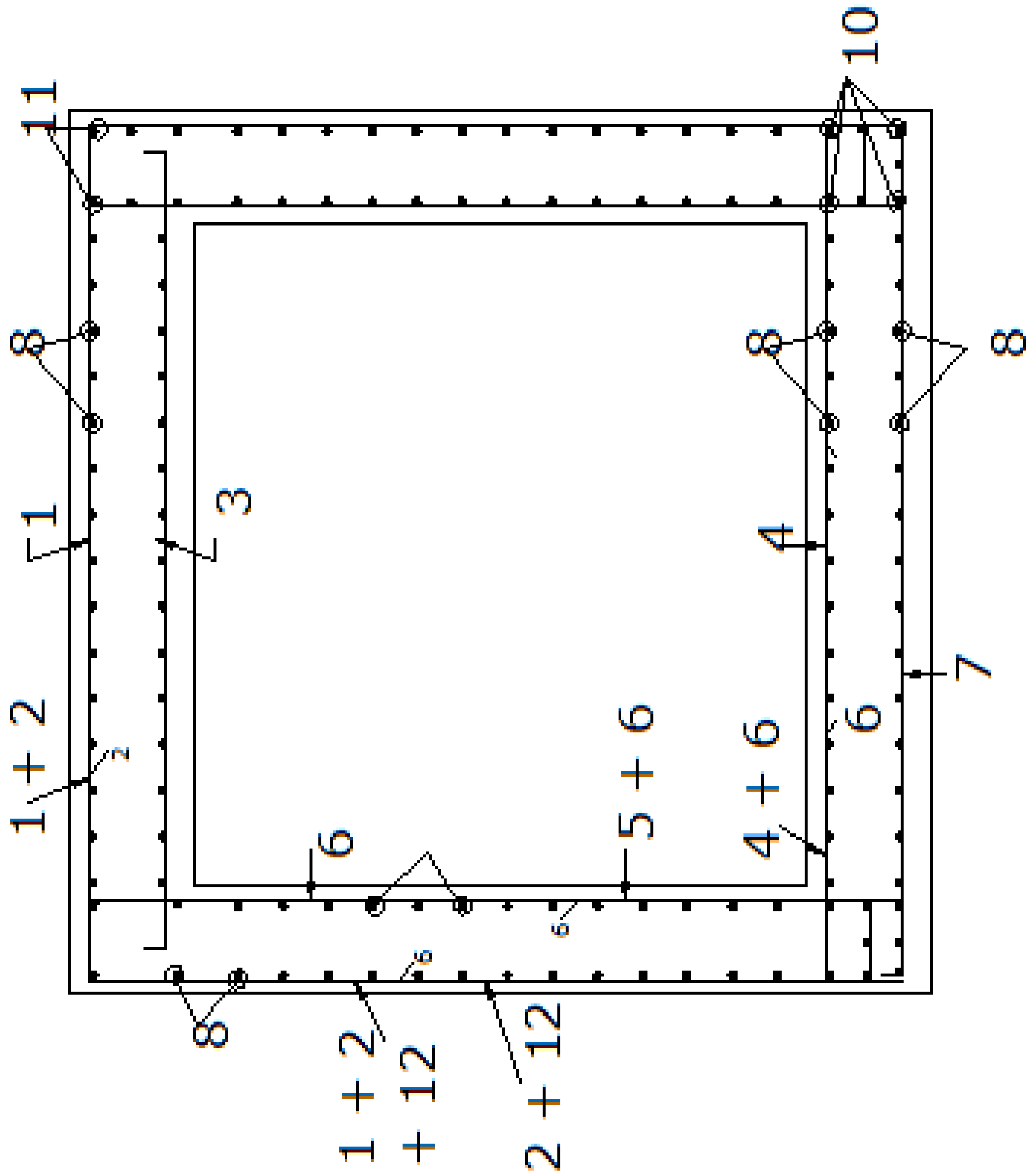


Figure 4.9: Reinforcement detail for Barrel

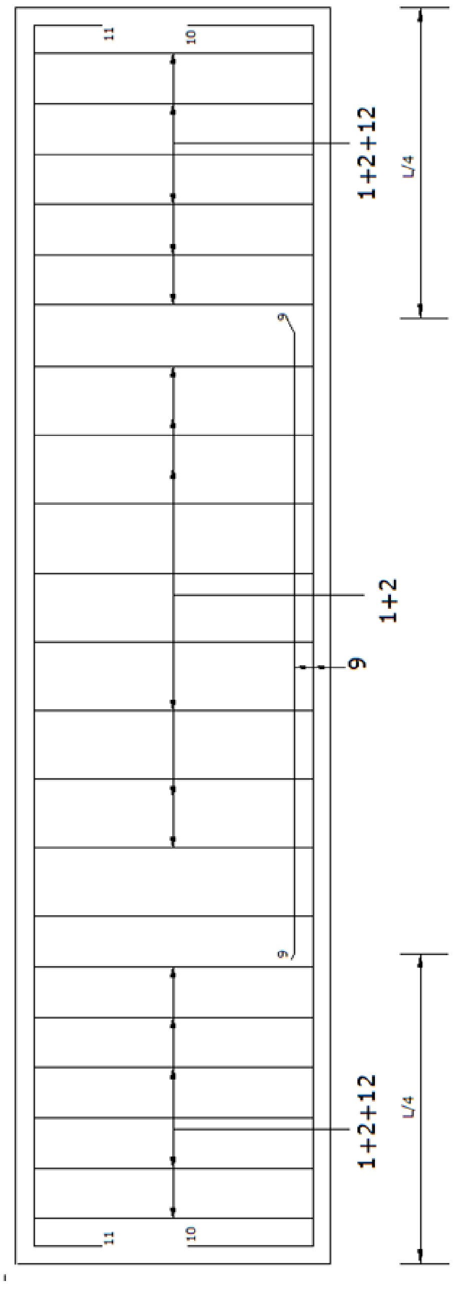


Figure 4.10: Reinforcement detail for Side wall of barrel

Table 4.4: Barbending schedule

Bar Notation	Shape of Bar	Dia of bar (mm	Spacing of bar (mm)	No.bars for single span	Weight for single
1	1.1 4.10m 1.1	16	280	54	532.42
2	1.20m 3.20m 0.40m	16	280	54	405.65
3	0.2 4.10m 0.2	20	200	75	813.42
4	0.1 4.00m 0.1	12	160	94	341.08
5	0.05 3.20m 0.05	12	160	94	274.53
6	1 1.20m	12	320	47	91.51
7	0.2 4.10m 0.2	16	260	58	400.45
8	15	12	160	84	1114.75
9	7.5	32		3 nos	283.96
10	1.3 15m 1.34	32		4 nos	842.71
11	0.5 15 0.5	16		4 nos	201.92
12	0.3 3.5	10 mm 2 leg ########		50	234.17
Total weight of steel for one span					55#7

Note : Above value shows quantity for 1 Span of super structure.

4.3 Design of substructure

4.3.1 Design of pier cap using limit state method

Data

Width of side wall = 400mm

Clear width of trough (Barrel) = 3500mm

c/c spacing of bearing in X-direction=3900mm

Width of cap=2500mm

a_v=1418.4

Load on one bearing=1053.6 kN

f_{ck}=30MPa

f_y=500MPa

M_u=(2*1053.65*1.4184)*1.5

=4483.49 kNm

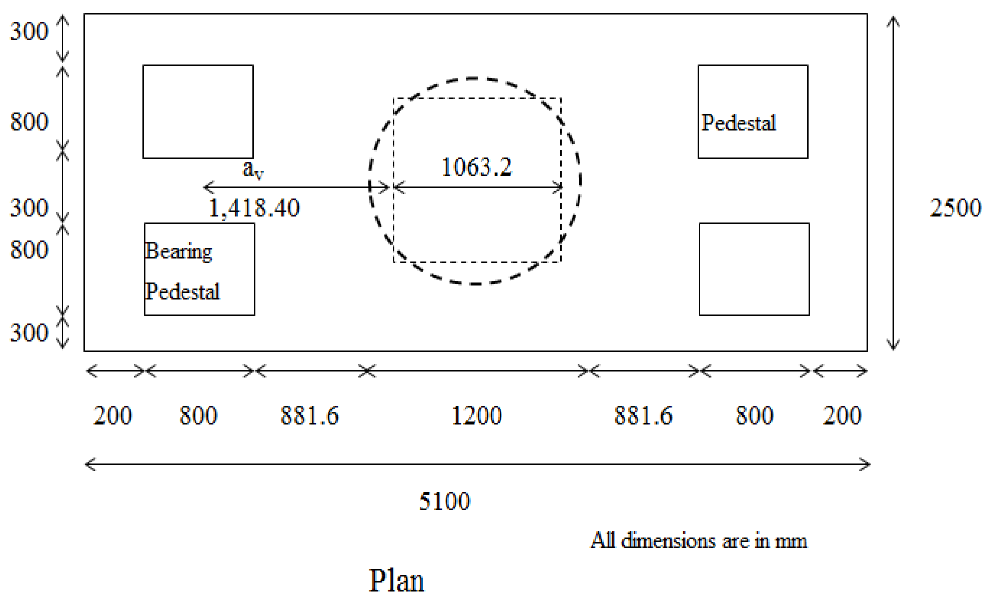


Figure 4.11: Pier cap

(A) d_{eff} required

(i.)As per IS-456(2000),cl.23.2.1,pg.37,

For cantilever,

span/d_{eff}=7

So,d_{eff}=$\frac{1418.4}{7}$

=202.63mm

Provide,d=950mm

Cover=50mm

So,D=1000mm

$\frac{a_v}{d}$

=$\frac{1418}{950}$

=1.5>1

So,Flexural criteria governs.

(ii.)

$$\begin{aligned} M_{u,lim} &= 0.133 * f_{ck} * bd^2 \\ So, d = \sqrt{\frac{4483.49 * 10^6}{0.133 * 30 * 2500}} & \\ &= 670.42mm \end{aligned}$$

(B) Effective width of pier cap as per IS 456(Cl.24.3.2.1(d))

Effective width below one bearing,

b_{eff}=1.2*a_1+a

Where,

a_1=a_v

a=width of contact area of concentrated load measured parallel to the supporting edge

=800mm

So, b_{eff}=2502.08 mm

So, b_{eff} = 2502.8 /2 + 1100 + 2502.8/2 as shown in fig.4.12

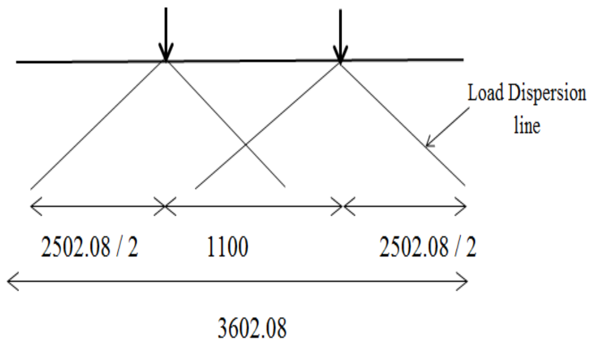


Figure 4.12: Combined effective width

Combined effective width

Minimum of two : (i)3602.08mm

. (ii)2500mm

So, combined b_{eff}=2500mm

(C)Design for flexure

$$\begin{aligned} A_{st} &= \frac{0.5*f_{ck}}{f_y}[1-\sqrt{1-\frac{4.6*M_u}{f_{ck}bd^2}}]bd \\ &= \frac{0.5*30}{500}[1-\sqrt{1-\frac{4.6*4483.5*10^6}{30*2500*950^2}}]2500*950 \\ &= 11827.5mm^2 \end{aligned}$$

Provide,15 numbers of 32mm dia. bars, Fig.4.13

Design for shear

V_u=3160.95 kN

$\tau_v=\frac{V_u}{bd}$

=1.3 MPa

$p_{t,provided}$=0.5%

τ_c=0.5MPa (From, IS:456,2000,Table-19)

$$S_v = \frac{0.87 * f_y A_{sv} d}{(\tau_v - \tau_c)bd}$$

Assume 12mm dia bars 8-legged stirrups

$$S_v = \frac{0.87 * 415 * 904.32 * 950}{(1.3 - 0.5)2500 * 950}$$

$So, S_v = 157.35mm$

Provide, 12 mm dia. bars with 2- legged stirrups with 150 mm c/c .fig.4.13

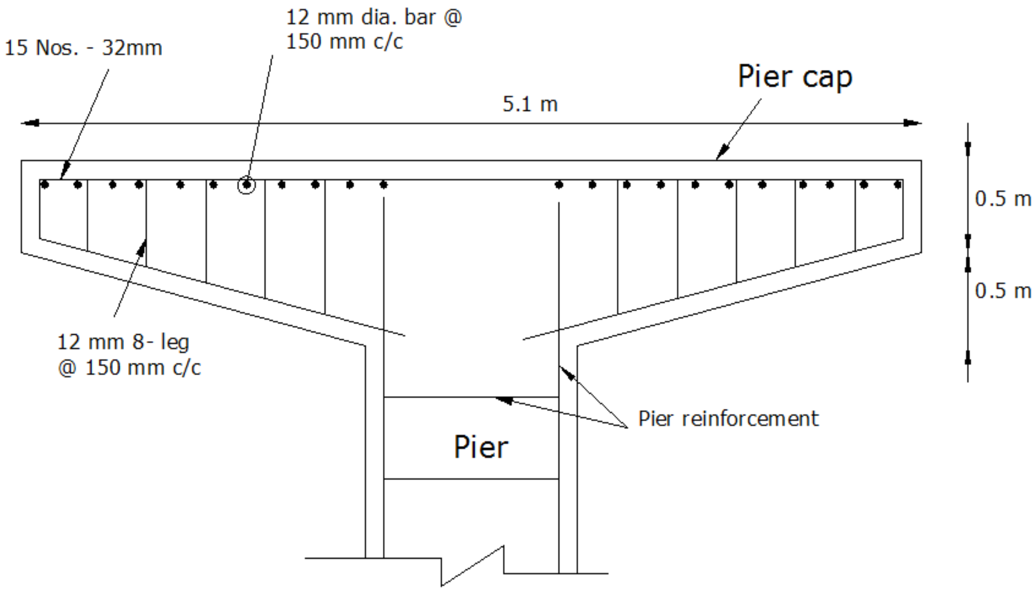


Figure 4.13: Reinforcement detail for Pier cap

4.3.2 Design of Pier

DATA

Diameter of pier =1.2m

Density of concrete= 25 kN/m^3

Height of pier= 5.5m

Maximum depth of water in drain= 4 m

Mean velocity of water in drain(v) = 2 m/s

K for pier = 0.66(cl210.2, IRC : 6)

Height of bearing pedestal = 0.3m

eccentricity between C.G. of column and C.G. of bearing in longitudinal direction(e) = 0.55m

(A) Calculation of loads acting on pier

(1) Load due to super structure

Self weight of trough = 2130 kN

Weight of water in Barrel = 1313kN

Max. Vertical reaction on single bearing due to live load = 239.7kN

Min. Vertical reaction on single bearing due to live load= 146.3 kN

Total axial load on column due to Vehicle = 2*(239.7+146.3)=772 kN

Braking force(cl.211.2, pg33, IRC:6-2010)

Breaking force = 20% of first train load + 10 % of the load of succeeding train

So, braking force =(0.2*228) + (0.1*272)

= 72.8 kN (in longitudinal direction)

(2) Pier cap

Width of pier cap = 2.5 m

Length of pier cap = 5.1 m

height of Rectangular portion of cap = 0.5 m

height of triangular portion of cap = 0.5 m

Weight of pier cap = 159.38 + 72.84

= 232.22 kN

(3) Weight of pier

Weight of pier = 3.14*(1.2^2)/4)*5.5*25

= 155.43 kN

(4) Water current force on pier

Intensity of pressure = 52 Kv^2

. where K = 0.66 , v =2

=52*0.66*2^2 = 145.2 kN

Force due to water current=Obstructed area * Intensity of pressure

= 7.536*145.2 = 1094.23 kN

(5) Bouncy force (cl.213, IRC: 6-2010)

Reduction in down ward load = Weight of displaced water

= Submerged volume of column x Density of water

= 3.14*1.2*4*10

= 150.72 kN

(6) Earth quake load (cl no.219.5, IRC : 6)

A_h = Z/2*(I/R)*Sa/g

Where Z = 0.16 (zone 3)

I = 1, Sa/g = 1, R = 1.5

So,A_h =0.13

(B) Load calculation for different Load combinations

Load combination (1)Trough full , Live load, Stream at H.F.L

Axial load = Self weight of Barrel + Weight of water in barrel + Live load

. + Weight of pier cap + Weight of pier - Bouncy force

= 2130 + 1312.5 + 772 + 232.22 + 155.43 - 150.72

= **4451.43 kN**

Horizontal force

Minimum eccentricity =l/500+D/30

In longitudinal Direction (kN)	Lever Arm (m)	Moment at Base of Pier (kNm)
Breaking force = 72.8	6.85	$72.8 \times 6.85 = 498.68$
Water current = 1094.23	2	$1094.23 \times 2 = 2188.46$
Total = 1167.03		2687.14

= 0.053 m

= 53mm > 20mm

BM due to minimum eccentricity = Axial load on pier * Min. eccentricity

. = 4451.43*0.053= 235.93 kNm

Which is less than total moment.

Load combination (2)Trough full , Live load, Stream dry

Axial load = Self weight of Barrel + Weight of water in barrel + Weight of pier cap + Weight of pier + Live load

= 2130 + 1312.5 + 772 + 232.22 + 155.43

= 4602.15 kN

Horizontal force

In longitudinal Direction (kN)	Lever Arm (m)	Moment at Base of Pier (kNm)
Breaking force = 72.8	6.85	$72.8 \times 6.85 = 498.68$

Load combination (3) Trough empty, no Live load, Stream at H.F.L.

Axial load = Self weight of Barrel + Weight of pier cap + Weight of pier - bouncy force

= 2130 + 232.22 + 155.43 - 150.72

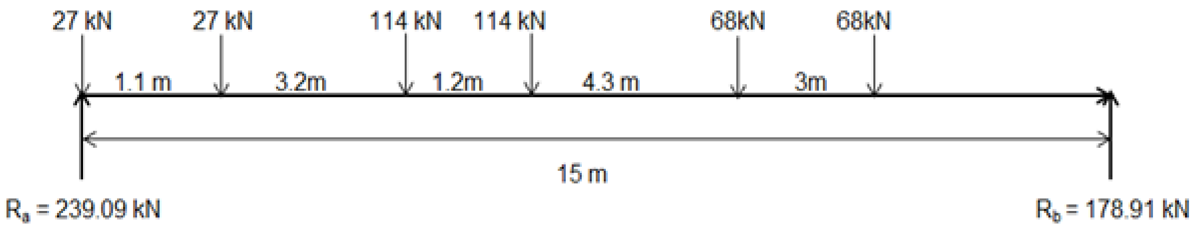
= 2366.93 kN

Horizontal force

In longitudinal Direction (kN)	Lever Arm (m)	Moment at Base of Pier (kNm)
Water current = 1094.23	2	2188.46

Load combination (4) Trough full, Live load on one span, Stream at H.F.L.

In One span loaded condition, following vehicle arrangement will arise



Impact Factor (IF) = 0.21

Ra with impact = 1.21 * 239.9 = 289.3 kN

Axial load = Self weight of Barrel + Weight of water in barrel + Weight of

. pier cap + Weight of pier + Live load due to one span- Bouncy force

. = 2130 + 1312.5 + 289.3 + 232.22 + 155.43 - 150.72

. = 3968.73 kN

Horizontal force

In longitudinal Direction (kN)	Lever Arm (m)	Moment at Base of Pier (kNm)
Breaking force = 72.8	6.85	72.8 × 6.85 = 498.68
Water current = 1094.23	2	1094.23 × 2 = 2188.46

BM due to one span loaded condition

BM = Live load due to one loaded span * eccentricity

= 298.3 * 0.55 = 159.12 kNm

Total moment = 498.68 + 2188.46 + 159.12

= 2846.26 kNm

Total horizontal force = 1167.03 kN

Load combination (5)

Super structure is constructed on one side of a pier and Stream at HFL

Axial load = Half weight of Barrel + Weight of pier cap + Weight of pier - Bouncy force

= 2130/2 + 232.22 + 155.43 - 150.72

= 1301.93 kNm

Horizontal force

In longitudinal Direction (kN)	Lever Arm (m)	Moment at Base of Pier (kNm)
Water current = 1094.23	2	2188.46

Eccentricity (e) = 0.55m

BM = 2130/2 * 0.55

= 585.75 kNm

Total BM = 2188.46 + 585.75

= 2774.21 kNm

Load combination (6)

Pier is constructed and Super structure is not constructed and Stream at H.F.L

Axial load = Weight of pier cap + Weight of pier - Bouncy force

= 232.22 + 155.43 -150.72

= 236.93 kN

Horizontal force

In longitudinal Direction (kN)	Lever Arm (m)	Moment at Base of Pier (kNm)
Water current = 1094.23	2	2188.46

Load combination (7)

Trough full,Vehicular load,Stream empty and Seismic

Axial load = Self weight of Barrel + Weight of water in barrel + Reduced Live load + Weight of pier cap + Weight of pier

= 2130 + 1312.5 + (772*0.25) + 232.22 + 155.43

= 4023.15 kN

Horizontal force

In longitudinal Direction (kN)	Lever Arm (m)	Moment at Base of Pier (kNm)
Breaking force = 18.2	6.85	124.67

Components	Vertical Load	Horizontal Seismic Force (kN)	Leaver arm (m)	Horizontal Horizontal Force (kNm)
Super structure	3635.5	0.13 × 3635.5 = 472.62	6.85	472.62 × 6.85 = 3237.45
Pier Cap	232.22	0.13 × 232.22 = 30.19	6.23	30.19 × 6.23 = 188.08
Pier	155.43	0.13 × 155.43 = 20.21	2.75	20.21 × 2.75 = 55.58
TOTAL		541.22	3605.78	

Load combination (8)

Trough empty + Vehicular load + Stream empty + Seismic in Longitudinal direction

Axial load = Self weight of Barrel + Reduced Live load + Weight of pier cap + Weight of pier

= 2130 + (772*0.25) + 232.22 + 155.43

= 2710.65 kN

Horizontal force

In longitudinal Direction (kN)	Lever Arm (m)	Moment at Base of Pier (kNm)
Breaking force = 18.2	6.85	124.67

Components	Vertical Load	Horizontal Seismic Force (kN)	Leaver arm (m)	Horizontal Horizontal Force (kNm)
Super structure	2323	$0.13 \times 2323 = 472.62$	6.85	$301.99 \times 6.85 = 2068.63$
Pier Cap	232.22	$0.13 \times 232.22 = 30.19$	6.23	$30.19 \times 6.23 = 188.08$
Pier	155.43	$0.13 \times 155.43 = 20.21$	2.75	$20.21 \times 2.75 = 55.58$
TOTAL		370.59		2436.96

Load Summary

Load Case	Axial Force (kN)	Moment (kNm)	Horizontal Force (kN)
Load Case 1	4451.4	2687.1	1167.03
Load Case 2	4602.2	498.7	72.8
Load Case 3	2366.9	2188.5	1094.23
Load Case 4	3968.7	2846.3	1167.03
Load Case 5	1301.9	2774.2	1094.23
Load Case 6	236.9	2188.5	1094.23
Load Case 7	4023.2	3605.8	541.22
Load Case 8	2710.7	2437.0	370.59

Reinforcement calculation for pier

Data

f_{ck} = 30 N/mm^2

f_y = 500 N/mm^2

slenderness ratio <12

So,design as short column

Assume = 32 mm bar with 50 mm clear cover

Dia of Pier = 1.2

d'/D = 0.0475

From SP - 16,

Table 4.5: Ast calculation summary table

Load Case	P_u/f_{ck} D^2	P_u/f_{ck} D^3	p/f_{ck}	p	$A_{st,req}$	Dia of Bar	No. of Bar req	No. of Bar provd	$A_{s,provd}$
Load Case 1	0.154	0.0777	0.05	1.5	16956	32	21.09	26	20899.84
Load Case 4	0.137	0.0823	0.06	1.8	20347.2	32	25.31	26	20899.84
Load Case 6	0.008	0.063	0.06	1.8	20347.2	32	25.31	26	20899.84

So, provide maximum,A_{st} 26 Nos, 32 mm dia bar

4.3.3 Design of foundation

Pile group is provided as foundation

(A)Bearing capacity calculation

Data

Assume, diameter of pile (D) = 1.3 m

(Min. diameter of pile = 1.2m (IRC 78))

Average cohesion at pile tip (C_p) = 90 kN/m^2

Average cohesion throught the length of pile(C) = 90 kN/m^2

Reduction factor α = 0.5

FOS = 2.5 (IRC 78)

Working axial force = 4602.2 kN (Load case 2)

Length of pile = 10 m

No. of piles = 4

Scour depth below DBL = 3.26 m

C/C Spacing of pile

As per IS 2911 min C/C Spacing = 3D = 3.9

Provide c/c spacing = 3.26

For,Cohesive soil, IS 2911 (Part 1 / Sec 2)

Qu = A_p N_c C_p + α c A_s

Bearing capecity of pile group

1) Single action

A_p = 1.33 m^2

N_c = 9 kN/m^2

A_s = 40.82

Qu = A_p N_c C_p + α c A_s

=(1.33*9*90) + (0.5*90*40.82)

= 2914.20

Total capecity = No.of pile x capecity of single pile

= 4 x 2,914.20

= 11656.80 kN

Safe bearing capecity due to single pile action = 11656.80/2.5 = 4662.72 kN

2) Group action

Perimeter = 20.8 m

A_p = 27.04 m^2

N_c = 9 kN/m^2

A_s = 208

Qu = A_p N_c C_p + α c A_s

= (27.04 x 9 x 90) + (0.5 x 90 x 208)

= 31,262.40

Safe bearing capecity due to group action = 7815.60 kN

Bearing capecity of pile group = Min. of following two

(i) Capecity due to Single pile action

(ii) Capecity due to group action

= 4662.72 kN

Total length of pile = Length required for bearing capecity + Scour depth form DBL

= 10 + 3.26

= 13.26 m

(B) Reaction calculation for individual pile

Data

Maximum Axial load on pile group (P) = 4023.2 kN (Load combination 7 of Pier design)

Self weight of pile cap = 1300 (From design of pile cap) kN

Total axial load = 5323.2 kN

Moment on pile group in X - direction (Mx) = 00 kNm

Moment on pile group in Y - direction (My) = 3605 kNm

Horizontal load on pile group = 541.2 kN

Diameter of pile = 1.3 m

c/c distance of pile in X - direction = 3.9 m

c/c distance of pile in Y - direction = 3.9 m

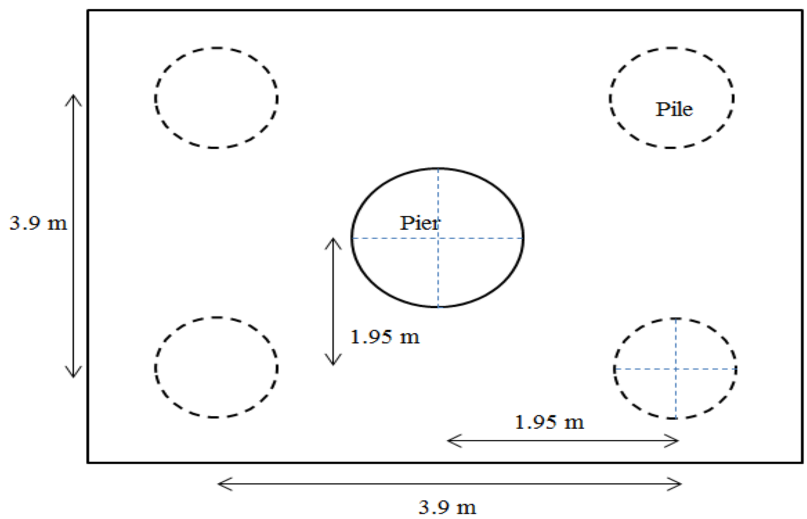
No. of pile in X - direction = 2

No. of pile in Y - direction = 2

Total no. of pile = 4

1) Axial force on each pile

When pile cap supports both axial load and moment, axial load on individual pile (P_p) can



$$P_p = \frac{p}{n} \pm \frac{M_y * X}{\sum X^2} \pm \frac{M_x * y}{\sum Y^2}$$

$= \frac{5323.2}{4} \pm \frac{3605.8*1.95}{4*1.95^2} \pm \frac{0}{4*1.95^2}$

Hence, $P_p = 1831.6$ kN or $P_p = 830$ kN

2) Moment on each pile

Moment on each pile in X - direction = $\frac{Total\ moment}{Number\ of\ pile}$

= 0 kNm

Moment on each pile in Y - direction = $\frac{Totalmoment}{Numberofpile}$

$= \frac{3605.8}{4}$

= 901.5 kNm

3) Lateral Load on each pile = $\frac{Total\ Load}{Number\ of\ pile}$

$= \frac{541.21}{4} = 135.303$

Loads on single pile due to load case 7

axial force = 1831.6kN

M_y = 901.5 kNm

F_y = 135.3 kN

Table 4.6: Summary : Load on Single pile

	Load Case 2	**Load Case 1**	Load Case 4	Load Case 7
Axial force(p) (kN)	1544.8	1811.1	1712.5	1831.6
M_y (kNm)	124.7	671.8	711.6	901.5
F_y (kN)	18.2	291.75	291.75	135.3

(C) Reinforcement calculation for individual pile

From summary, in normal condition governing case is : Case 1

Reactions of load case:7 is slight high than case:1, but since, case:7 is Seismic case permissible stresses of materials increases by 25 percent. So case 7 will not govern.

Working load on single pile (from load case 1) p = 1811.1 kNm

M_y = 671.8 kNm

F_y = 291.75 kN

Moment due to lateral load at top of plie

As per IS: 2911 (Part I/ Sec 2), equivalent cantilever length of pile can calculate as below

From table 2, K = 48

From figure 3. L/d = 4

. where, L = equivalent cantilever length of pile

. d = diameter of pile = 1.3m

So, L = 4 * 1.3 = 5.2m

The fixed end moment (M_F) of the equivalent cantilever is higher than the actual maximum moment (M)

Actual maximum moment (M) = $m * M_F$ (IS: 2911 (Part I/ Sec 2))

where m = 0.7 (fig.3, Ament no.3)

M_F = F_Y L /2 (As per C-2.1, Ament no.3)

= 291.75 x 5.2 / 2

= 758.55 kN-m

So,

M = 0.7 x 758.55 = 530.99 kN-m

Total moment = BM from pier + BM due to lateral load

= 671.8 + 530.985

= 1202.78 kN-m

Design load for single pile

F.O.S = 2.5 (IRC : 78)

P_u = 2.5 x 1811.1 = 4527.75 kN

M_{uy} = 2.5 x 1325.265 = 3006.95 kN-m

f_{ck} = 30 N/mm^2

f_y = 500 N/mm^2

$\frac{P_u}{f_ck*D^2} = 0.089$

$\frac{M_u}{f_ck*D^3} = 0.05$

Assume, 25 mm diameter bar

Cover = 80 mm (IRC : 78, Cl.709.4.4)

d' / D = 0.06

From SP-16

p / f_{ck} = 0.03

So, p = 0.9

A_s = 11939.85 mm^2

Numbers of bars required = 24.3

So, provide, 25 Nos, 25 mm diameter bar

Pitch and diameter of lateral ties

Pitch

i Least lateral dimension of member = 1.3m

ii Sixteen times the small diameter of the longitudinal reinforcement = 400

iii 300 mm

Diameter

i diameter > diameter of longitudinal bar/4 = 6.25

ii 6mm

Provide 10mm diameter ties at 300mm c/c distance

(C) Design of pile cap

Data

Working Axial load on pile group (P) = 4451.4 kN

Axial force on each pile = 1811.1 kN (Load case 1, reaction calculation, Design of pile)

Diameter of pile = 1.3 m

c/c distance of pile in X - direction = 3.9 m

c/c distance of pile in Y - direction = 3.9 m

No. of pile in X - direction = 2

No. of pile in Y - direction = 2

Total no. of pile = 4

Diameter of pier = 1.2

f_y = 500 N/mm^2

$f_c k$ = 25 N/mm^2

1) Length of pile cap

As per IRC:78, Min.offset of pile cap beyond the outer face of outer most pile = 150mm

Offset provided = 0.2 m

So,

Length of pile cap (in x - direction) = 5.6 m

Length of pile cap (in y - direction) = 5.6 m

2) Thickness of pile cap

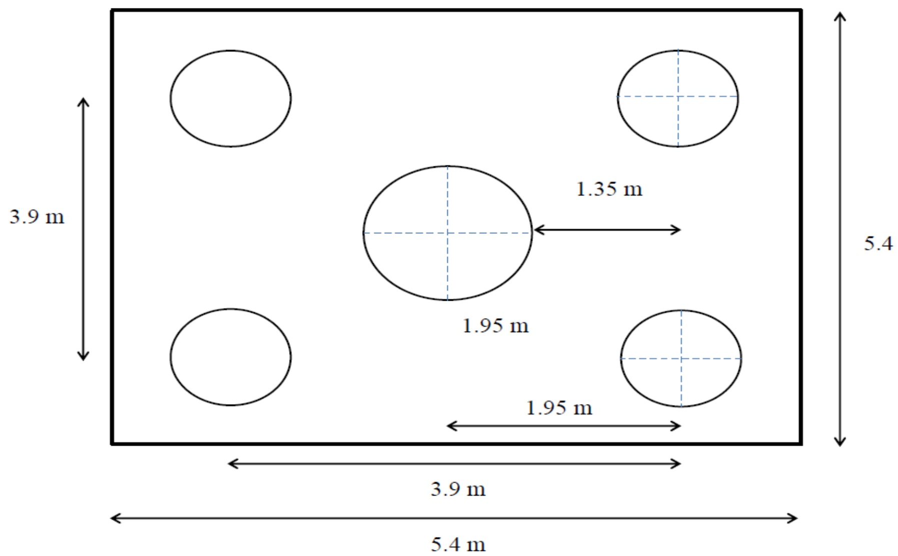
I) As per IRC -78, cl : 709.5.3 minimum thickness of cap = Max of two :

i) 0.6 m

ii)1.5 x Dia.of plie = 1.95 m

So ,

D = 1.95 m



Assume 25 mm bar, and provide 0.04 m nonimal cover to main reinforcement.

d = 1.95 - 0.04 - 0.025 - 0.025/2 = 1.87 m

II) For rigid pile cap span to thikness ratio should greate than 5 So, Min thickness required = span/5

= 3.9 / 5 = 0.8

III)Thickness based on Bending moment

Ultimate reaction from each pile on the pile cap (p')

p' = 1.5 * 1811.1 kN

p' = 2716.65

Moment at face of pier (M_u max) = 2 x 2716.65 x 1.35 = 7334.96

M_u max = M_u lim

7334.955 x 10^6 = 0.133 f_{ck} b d^2

d = $(\frac{7334.955*10^6}{0.133*25*5400})^{0.5}$

= 639.15 mm

d_{req} = Maximum of above three criteria = 1.87 m

Provide, D = 2 m, So d = 1.93 m

$A_s t = \frac{0.5*f_{ck}}{f_y}$ (1-$\sqrt{1 - \frac{4.6*M_u}{f_{ck}*}}$) b d

A_{st} = 0.025 * (1-0.966) * 5400 * 1930

A_{st} = 8858.7 mm^2

As the cap behaves as a wide beam ,

A_{st} Min = 0.85 * b*d* $/f_y$

A_{st} Min = 18373

Provide 25 mm bar

So,C/C Spacing required = 149.54 mm

Provide C/C Spacing = 150 mm

A_{st} provided = 18316.7 mm^2

p_t provided = 0.17

3) Design for shear

(a) One - way shrear

The max.shear force(Vu) at the critical section at distance ' d ' from the face of pier

V_u = No. of piles in a row x p' = 2*2716.65

= 5433.3 kN

for pt = 0.17, τ_c = 0.304 N/mm^2 (From, IS-456:2000)

Shear stress at critical section (τ_v) = $\frac{V_u}{b*d}$

(τ_v) = $\frac{5433.3}{5.4*1.93}$

= 521.33 kN/m^2

= 0.521 N/mm^2

$\tau_c < \tau_v$ (unsafe)

Shear reinforcement have to design

$S_v = \frac{0.87*f_y*A_{sv}*d}{(\tau_c - \tau_v)b*d}$

. Assume, 12 mm bar 8 leg stirrups

$S_v = \frac{0.87*415*904.32*1930}{(0.52-0.30)5600*1930}$

S_v = 293 mm

Provide ,12 mm dia bar 8 leg stirrups 290 at mm c/c

(b) Two - way shrear (Punching action)

Punching shear should check for both punching action of pier and punching action of pile at critical section (half the effective depth of pile cap) from the face of Pier and Pile.

Permissible shear stress in the pile cape (τ_c) = k_s*τ_p

where ks 0.5 + bc = 1

τ_p =0.25*$\sqrt{f_{ck}}$

hence,τ_c 1.25

i)Punching actoin of pier

Shear stress at critical section (tau_u) = $\frac{V_u}{pd}$

. p = periphery of the section = 11.96m

(tau_u)= $\frac{1.5*4451.4}{11.96*1.93}$ =289.27 kN/m^2

=0.28927 N/mm^2

$\tau_c > \tau_v$ Hence Safe

ii) Punching action of pile

Shear stress at critical section (τ_v) =$\frac{V_u}{pd}$

. p = periphery of the section = 12.32 m

= $\frac{2716.65}{12.32*1.93}$ =280.81 kN/m^2

. =0.28 N/mm^2

$\tau_c > \tau_v$ Hence Safe

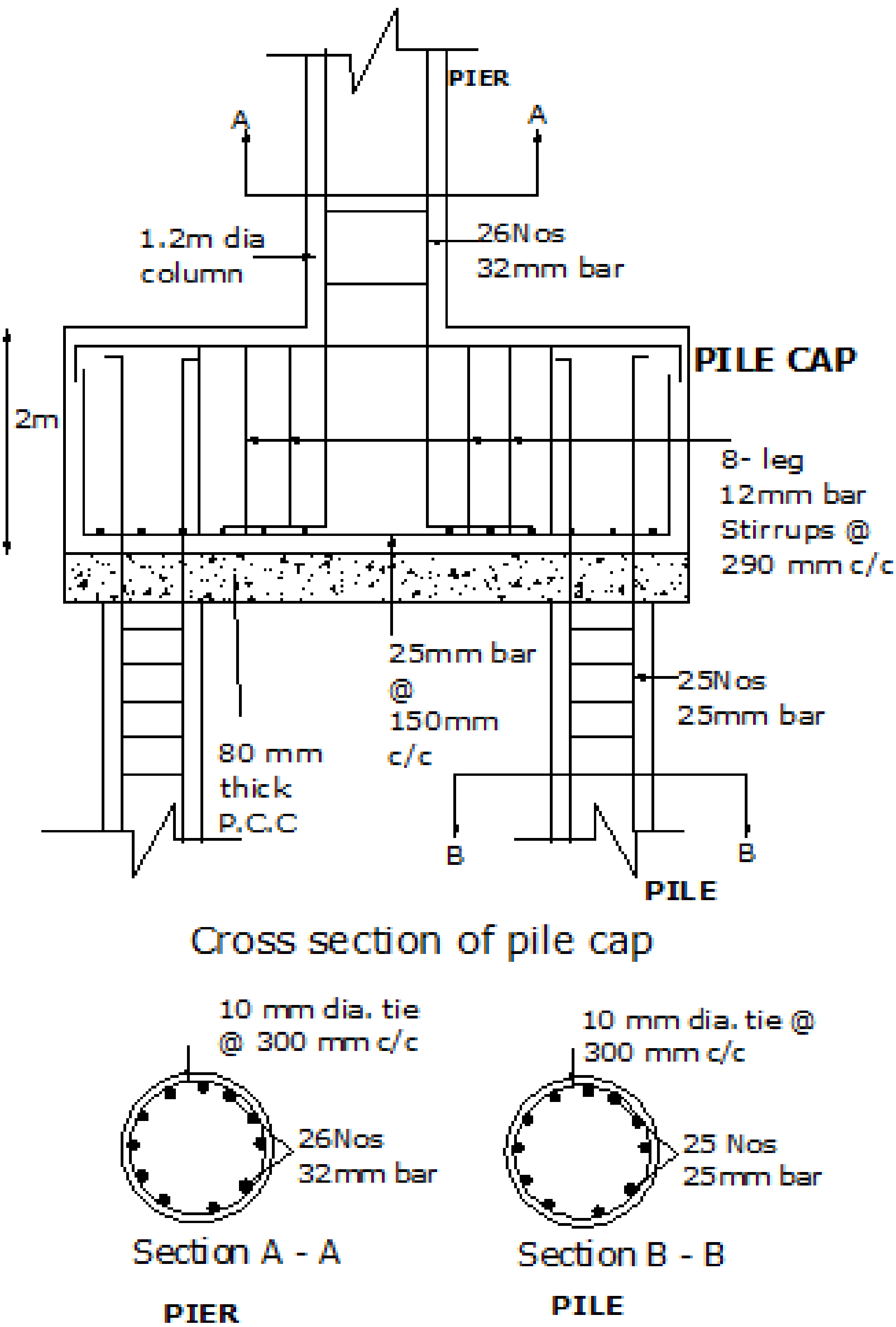


Figure 4.14: Reinforcement detail for pile cap

Chapter 5

Parametric Study

Economy of aqueduct depends on the cost of substructure and superstructure and therefore the aqueduct should be designed in such a manner that the cost of substructure and superstructure should be in well proportion. For this purpose parametric study is carried out. In order to carry out parametric study, the total discharge is kept same and span is varied as 10m, 15m, 20 and 25m. Following table shows the cost of superstructure for one span and cost of substructure for one unit for different spans.

Table 5.1: Variation in cost of one span of super structure and one unit of sub structure

Sr. No.	**Span m**	**Super structure cost Rs.**	**Sub structure cost Rs.**
1	10	396924	650936
2	15	615340	891990
3	20	863192	1137400
4	25	1164372	1480116

Note: Rate of concrete = 4400 Rs./m^3 , Rate of reinforcement = 4400 Rs./ tonne

Variation in total cost of structure is studied and produced in following table.

Table 5.2: Variation in total cost of Structure

Sr. No.	**Span m**	**Super structure cost Rs.**	**Sub structure cost Rs.**	**Total cost of structure Rs.**
1	10	3572316	5858424	9430740
2	15	3692040	5351940	9043980
3	20	3884364	5118300	9002664
4	25	4191739	5328418	9520157

The graphical representation of variation in cost of superstructure, cost of substructure and total cost of structure is shown below chart.

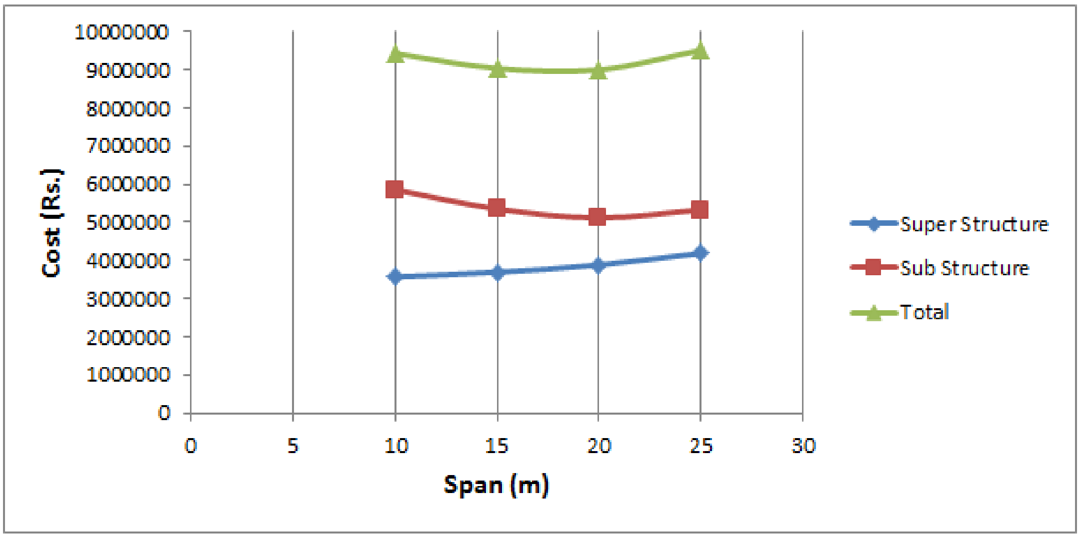


Figure 5.1: Variation in total cost of structure.

The graph shows that cost of structure decreases to some extent and then again increases with increase in span. The coordinate when graph dips, gives the economical cost of structure.From chart,18m span length is found to be economical span length.

Chapter 6

Conclusion and further scope of study

6.1 Conclusion

6.1.1 Syphon

In ordered to understand economy of both shape, Comparison is done in terms of cost and head loss.
Following tables shows the Cost comparison for unit meter length of syphon and Total head loss.

Table 6.1: Cost comparison

Cross section	**Volume** of **concrete** m^3	**Weight of reinforcement** **kg**	**Area of shutter** m^2	**Cost**/m **length** **Rs.**
Rectangular	6.6	367	16	48388
Circular	2.1	105	20	21860

Note: Rate of concrete = 4400 Rs./m^3

. Rate of reinforcement = 44000 Rs./$tonne$

. Rate of shuttering for box syphon = 200Rs./m^2

. Rate of shuttering for circular syphon = 400Rs./m^2

Table 6.2: Headloss comparison

Cross section	Total headloss cm
Rectangular	40.35
Circular	38.29

Conclusion In terms of cost, Circular section is 2 times cheaper than Rectangular section.
In terms of head loss Circular section reduce head loss by 10 Percent.

6.1.2 Aqueduct

From parametric study it can conclude that, 16m span length is most economical for entire length of aqueduct.

6.2 Further scope of study

6.2.1 Canal syphon

In present work, design of canal syphon singe barrel is considered. Multi barrel can be used.
In the present work, design of transition walls are not considered in design so in further work it can be added in the work.

6.2.2 Aqueduct

In present work, IRC class-A single lane load is considered, so in further work other classes of vehicular loading can be considered.

In the present work Circular pier is considered so in further work another section like Wall, Hollow, Frame, etc. can be used.

In the present work, clayey soil is considered, so further other types of soil conditions can be considered and the foundations can be designed accordingly.

In the present work bearing design is not considered so in further work bearing can be designed for.

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