DESIGN OF RADIAL GATE AND ITS PIERS

By

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DESIGN OF RADIAL GATE AND ITS PIERS

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

Jayesh B Patel
(05MCL009)

Guide
Shri. J. S. Thakur

DEPARTMENT OF CIVIL ENGINEERING
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May 2007

CERTIFICATE

This is to certify that the Major Project entitled “Design of radial gate and its piers” submitted by Mr. Jayeshkumar B Patel (05MCL009), 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 of Science and Technology, 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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Date of Examination

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ABSTRACT

One of the Prime necessity of a Water Storage Scheme is the amount of water it can store and its flexibility in handling the instantaneous fluctuation of water during high floods. Such schemes are, therefore, supplemented with Gates. Gates not only increase the water storing capacities of Dams but also help in releasing excess water in the times of high floods thus saving the total Dam structure from failure.

This Project deals with analysis, design, and detailing of radial gates subjected to water loads. The Programming is carried out in C^{++} language for the analysis and design of Radial gate. The design of radial gate mainly involves design of skin plate, horizontal girders, radial arms and trunnion. The design is carried out for FRL and then sections obtained are checked for HFL, Hydrodynamics Pressure etc. with increase in permissible stresses by 33% as per the provision of BIS.The Program gives output in terms of section sizes and stresses they carry. Greater care has to taken in deciding the location of horizontal girder so that they carry almost equal horizontal forces, thus resulting into economical design of girder. Radial Arms are designed as compression members and the bracing are designed for carrying 2.5% loads on the arms carry. This is also done in C^{++} language. The radial arm, in turn, transfers the loads to trunnion girder via trunnion bracket. It is basically a design of steel connection and same is carried out in Excel, using welding application.

The second part of design consists of design of Pier and its footing, that carry loads from Gates. Apart from this, Pier carries live loads, weight of bridge and hoisting platform. Pier is generally found critical in transverse direction, when one adjoining gate is closed and other is kept open. Pier is designed for biaxial eccentricity. Steel Reinforcement is designed along the periphery of the pier to take care of stresses at crest level. This is also done in MS Excel, in this project.

As Pier is tightly supported at crest level, till foundation level, by spillway, it is designed as footing subjected to uniaxial eccentricity along the length of the pier. The bearing capacity of the soil underneath is assumed as $30 t/m^2$. Various parametric studies, along with case study, are carried out to check the authencity of program prepared.

Results are also checked with results of STAAD PRO. Thus this program would be a helping hand in taking preliminary decisions and accordingly, it will help in carrying out design speedily, accurately and economically.

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NOTATIONS AND NOMENCLATURE

The symbols and notations given below apply

t	Thickness of plate, hub thickness
B	Width of the stiffeners flange in contact with skin plate
l	Effective length
r	Least radius of gyration
p	hydrodynamic pressure
c_s	Coefficient which varies with shape and depth
w	Unit weight of water
h	Depth of reservoir
I	A factor depending upon the importance of the structure
α_h	Design horizontal seismic coefficient
β	Coefficient depending upon the soil foundation system
α_0	Basic horizontal seismic coefficient
hw	height of wave from trough to crest
V	Wind velocity
F	‘Fetch’ or straight length of water
P	Wind pressure
Vh	Hydrodynamic shear
Mh	Moment due to hydrodynamic force
y	Depth
V_z	Design wind speed at any height
V_b	Basic wind speed
k_1	Probability factor
k_2	Terrain, height and structure size factor
k_3	Topography factor
P1	Self weight of pier
P2	Longitudinal hydrostatic pressure due to steady water
P3	Longitudinal water pressure due to flowing water
P4	Transeverse water pressure due to closed
P5	Transeverse water pressure due to gate open
P6	Hydrodynamic force due to earthquake
P7	Reaction due to opening of gate
P8	Weight of bridge

P9	Live load due to bridge
P10	Frictional resistance
P11	Braking force
P12	Pin reaction
P13	Longitudinal wind pressure
P14	Weight of hoist equipment
P15	Weight of trunnion equipment
P16	Uplift pressure

1. INTRODUCTION

1.1 GENERAL

Gates are movable structures which are used to close the openings of hydraulics structures and to control discharges. Sometimes on large dams, regular gates may be installed over the permanent crest, to function like a movable additional crest. In such a case, the height of the permanent raised crest can be reduced and the balance provided by the movable crest (i.e. gate) as shown in fig. 1.1

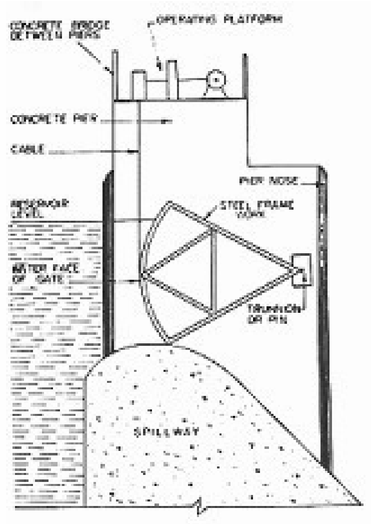


Figure 1.1 Arrangement of radial gate on dam

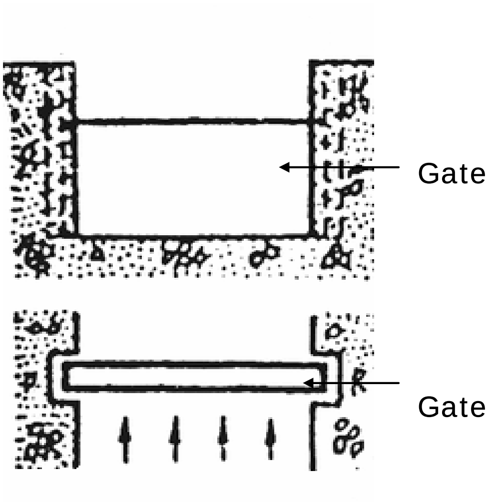
If there is a permanent raised crest up to the gate top, the storage, would be equal to that of a gated crest; but in times of floods, the rise in flood level would be higher as compared to what would have been in gated crest. This is because; the gates would be opened during floods so as to provide more head and, hence, larger discharge and consequent lesser rise in flood levels. Hence, the top level of the non-overflow section and the value of land acquisition for the reservoir which has to be determine by the maximum rise of flood above the spillways crest, can be reduced by providing gated crest or controlled crests. In other words, the dam height can be reduced for the same height by providing the dam spillway is controlled by gate etc. Gates can be provided on all types of spillways except syphon spillways. In syphon spillways, the gates not required as the rise in flood level is already small compared to other types spillways. The gates for earthen dams should be provided with caution, since the faulty operation or failure of their operation may lead to serious rise in flood levels, causing overtopping and failure of dam.

1.2 CLASSIFICATION OF GATES

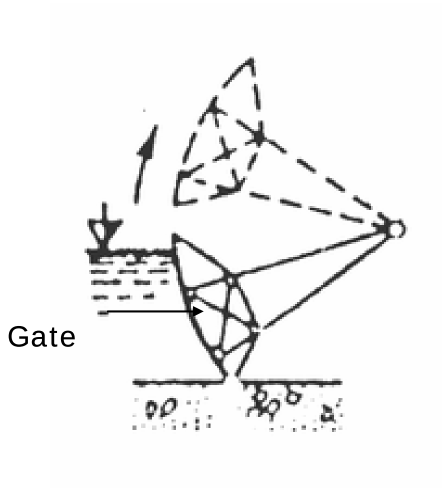
1.2.1 Classification on the basis of location The gates can be classified as:

(a) Crest type gates intended to close the overflow openings as shown in fig.1.2.

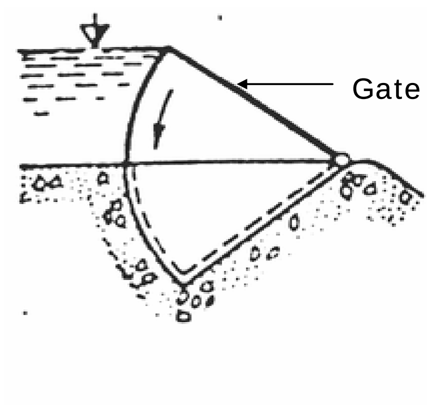
(b) Submerged type gates to close bottom orifices or conduits as shown in fig.1.3.



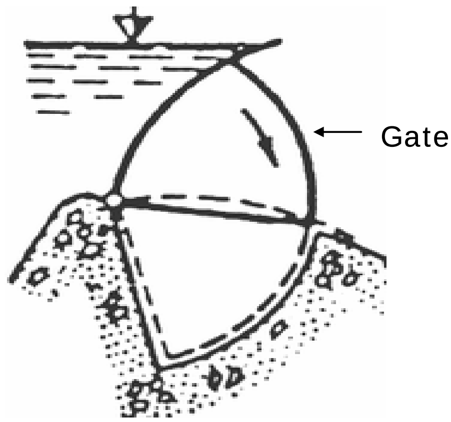
(A) Stop logs



(B) Radial gate

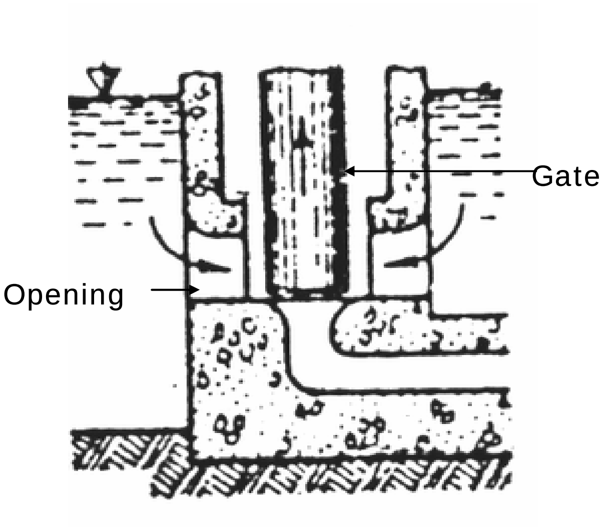


(C) Sector gate

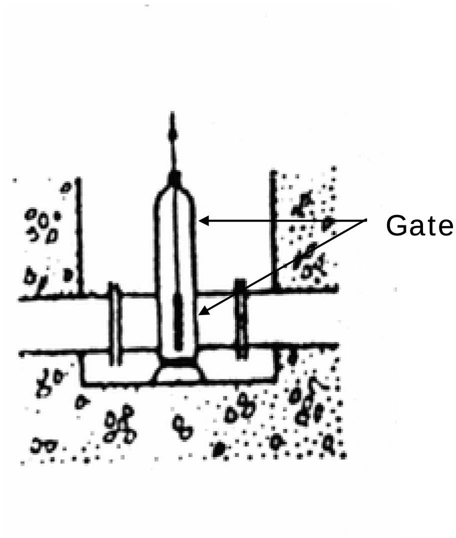


(D) Drum gate

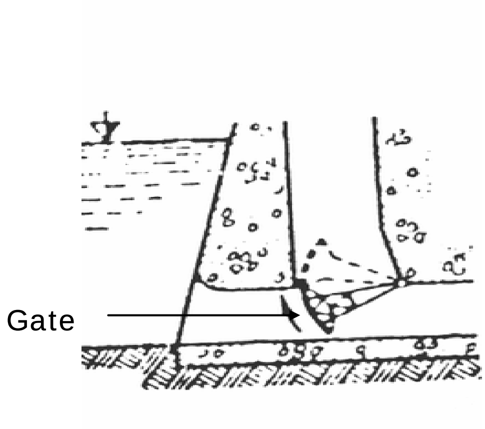
Figure 1.2 Crest type gates



(A) Cylindrical gate



(B) Plain gate valve



(C) Radial gate

Figure 1.3 Submerged type gates

1.2.2 Classification on the basis of operation The gates can be classified on the basis of operation as:

(a) Service gates - used for routine operations,

(b) Bulkhead gates - to close the opening for maintaining a service gate or structure below the gate,

(C) Emergency gates - to close the openings in case of accident of emergency,

(d) Construction gates - to shut off the openings during construction usually for closing diversion channels/conduits.

1.2.3 Classification on the basis of mode of operation The gates can be classified on the basis of mode of operation as:

(a) Regulating gates, operable when partly opened,

(b) Non-regulating gates, which can be opened only fully.

1.2.4 Classification on the basis of motion The gates can be classified on the basis of motion as:

(a) Translation gates,

(b) Rotary gates,

(c) Rolling gates,

(d) Buoyant gates.

1.2.5 Classification on the basis of water heads The gates can be classified on the basis of water heads as:

(a) Low head gates, suitable for water head of up to 15m,

(b) Medium head gates, suitable for water head of 15m to 30m,

(c) High head gates, suitable for water heads of 30m and above.

1.3 RADIAL GATE

The radial gate, as its name implies, is in the shape of a portion of a cylinder rotating about a horizontal axis. Normally, the water is against the convex side of the gate but also sometimes water load is applied on the concave side also.

A radial gate, also known as a tainter gate, has its water supporting face, made of steel plates, in the shape of sector of a circle, properly braced and hinged at the pivot. The gate can, thus, be made to rotate about fixed horizontal axis. The load of the gate and water etc. is carried on bearings mounted on piers. The gate can be lifted by means of ropes and chains acting simultaneously at both ends or with the help of power driven winches. A typical figure is shown in fig.1.4, showing skin plate, horizontal girder, stiffeners, trunnion on assembly etc.

Radial gates are widely used as check structures to head and flow discharge in irrigation canals and other similar conduits, Radial gates are inexpensive, simple to operate and can be installed in canals as an appurtenant structure, in large dam or where large gate must be installed. This is most popular one in certain range of size. These are more economical than the fixed wheel gates. These gates have following advantages:

(a) They do not require any grooves and therefore, ensure better flow conditions adjacent to the piers.

(b) The absence of wheel ensures overall economy in the cost of the wheels and bearings etc.

(c) There are less maintenance problems.

(d) As the radial gate is operated by rotating around its hinge. The inherent advantages of lever arm results in reduction in the hoist capacity and thus contributes to economy.

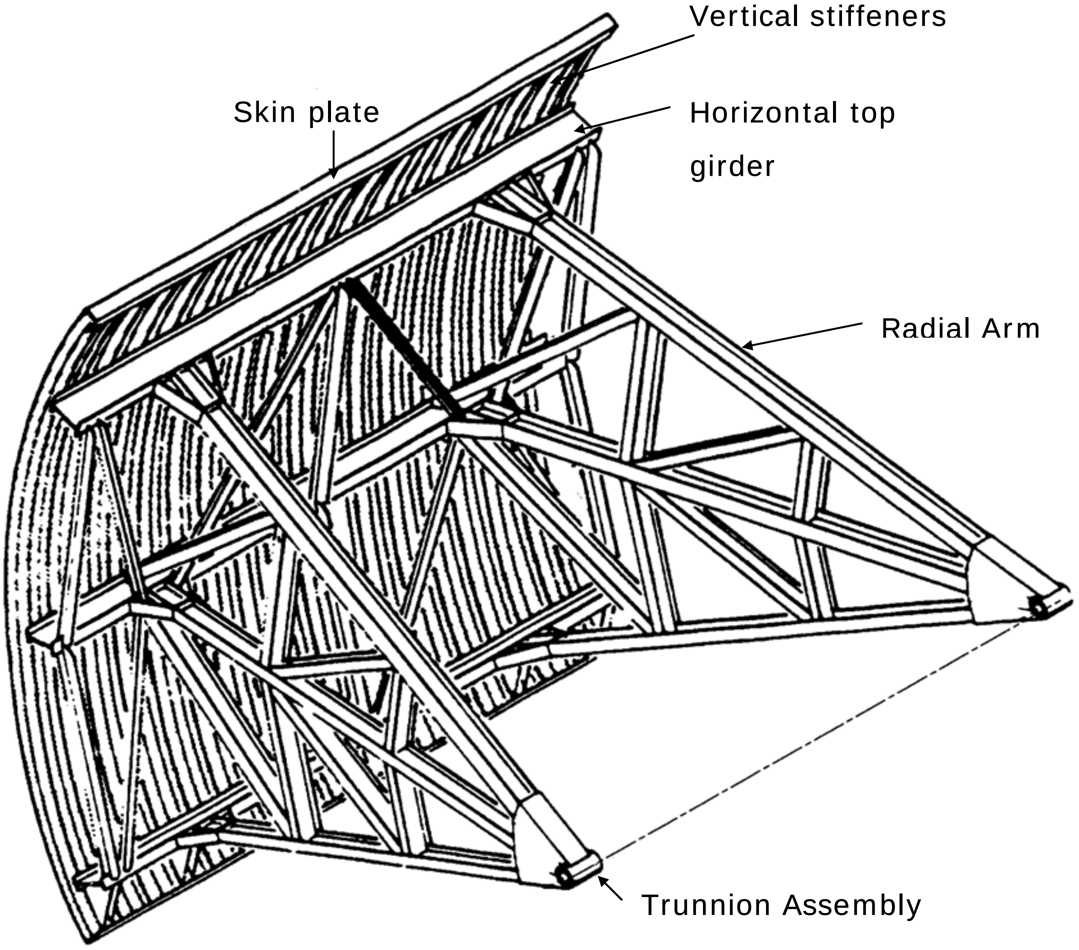


Figure 1.4 General model of radial gate

Spillway radial gates are effectively applied for use on spillways of various projects due to favorable operating and discharge characteristics. Radial Gates are used on flood control projects, navigation projects, hydropower projects, and multipurpose projects (i.e., flood control with hydropower). Although navigation and flood control radial gates are structurally similar and generally have the same maximum design loads, the normal loading and function may be very different. In general, gates on navigation projects are subjected to significant loading and discharge conditions most of the time, whereas gates on flood control projects are loaded significantly only during flood events. These differences may influence selection of the lifting hoist system, emphasis on detailing for resistance to possible vibration loading, and selection of a corrosion protection system. Table 1.1 shows some of the radial gates installed in various water resources project in India.

Table 1.1. Important radial gates installations in India.

SR NO	Project	State	Size of gate Width x Height mxm.	Number
1	Bhakra dam	Punjab	15.25 x 14.5	04
2	Hirakud dam	Orissa	15.55 x 6.1	34
3	Mandira dam	West Bengal	15.55 x 6.1	11
4	Kota barrage	Rajasthan	12.2 x 12.2	19
5	Koyna dam	Maharastra	12.5 x 7.63	06
6	Rihand dam	U.P	12.2 x 8.53	13
7	Barapani dam	Assam	12.5 x 12.2	02
8	Maithon dam	Bihar	12.5 x 12.2	12
9	Panchet Hill dam	Bihar	12.5 x 12.2	15
10	Ichari dam	U.P	9.5 x 16.5	07

1.3.1 Navigation Projects

Navigation projects are normally built in conjunction with a lock. Navigation gates are designed to maintain a consistent pool necessary for navigation purposes, while offering minimum resistance to flood flows. Gate sills are generally placed near the channel bottom, and during normal flows, damming to the required upper navigation pool elevation is provided by radial gates. Under normal conditions, most gates on a navigation dam are closed, while several other gates are partially open to provide discharge necessary to maintain a consistent upper lock pool as shown in fig. 1.5

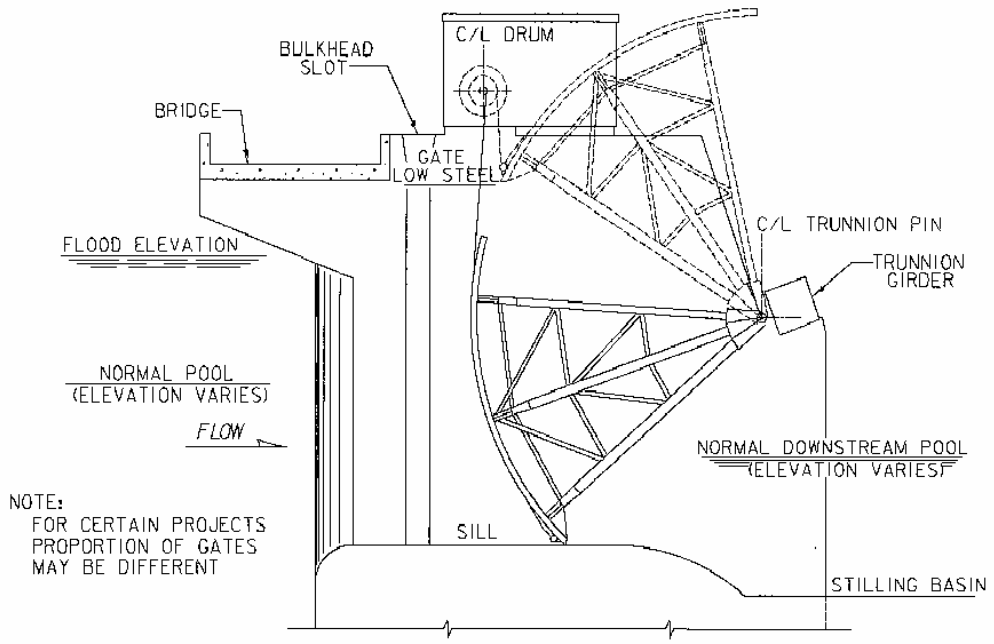


Figure 1.5 Typical navigation radial gate

During flood events, gates are open and flood flow is not regulated. The upper pool elevation often rises significantly during flood events and the open gate must clear the water surface profile to pass accumulated drift. As a result, the trunnion elevation is often relatively high and the gate radius is often longer than gates designed for other applications. Under normal conditions, navigation gates are generally partially submerged and are significantly loaded with the upstream-downstream hydrostatic head. In addition, these gates are more likely to be subject to flow-induced vibration and cavitations.

1.3.2 Flood control and Hydropower projects

Flood control projects provide temporary storage of flood flow and many projects include gated spillways to provide the capability to regulate outflow. On flood control projects with gated spillways, gate sills are generally located such that the gates are dry or only partially wet under normal conditions. In general, gates are exposed to the atmosphere and are subjected to slight loads, if any. Only during infrequent flood events are gates loaded significantly due to increases in pool, and during subsequent discharge hydraulic flow-related conditions exist as shown in fig.1.6. Trunnions are typically located at an elevation approximately one-third the height of the gate above the sill. Some unique multipurpose projects (Projects that provide flood control and reservoir storage)

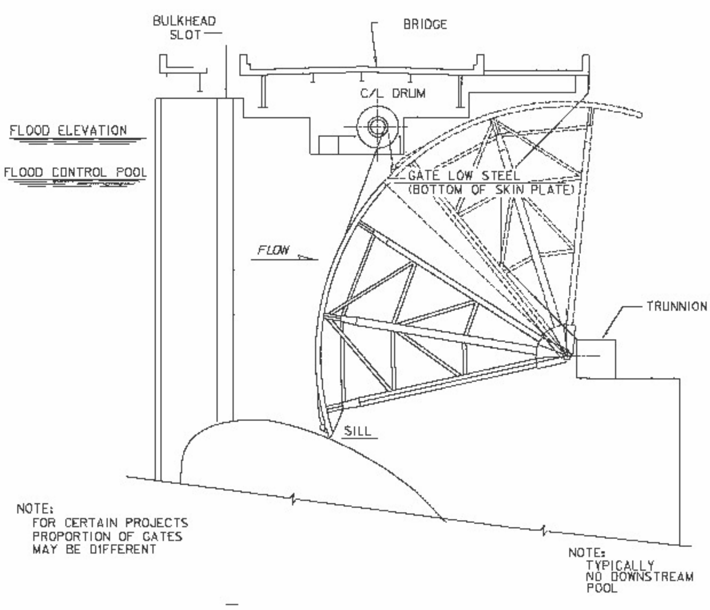


Figure 1.6 Typical flood control radial gate

and most hydropower projects include aspects of flood control and navigation gates. Gates on these projects are normally subject to significant hydrostatic loading on the upstream side and may be used to regulate flow on a regular basis.

1.4 PIER

A substructure unit that supports the spans of superstructure at an intermediate location between its abutments. Some common types of piers for supporting superstructure used are:

1) Wall type
2) Capped pile
3) Tee type (hammerhead)
4) Cap and Column

5) Multiple columns (no cap)

1.4.1 Wall type Pier

This is a full height, rectangular pier extending from the ground line or stream bed up to the bearing elevation. The pier extends the full width of the bridge, supporting all beam members. The most common type of material used for this type of pier is concrete (although stone was used for the older types). If the bridge is continuous over the piers (no expansion joint), there is very minimal maintenance required. If the bridge has multiple simple spans with unsealed joints over the piers, the pier seat must be rigorously maintained by power washing and sealing. Stream scour can be a threat at the base of the pier on bridges over water.

1.4.2 Capped pile pier

The capped pile pier consists of multiple piles driven into the ground in one straight line. The most common type of piling used for these piers is concrete and steel. The piling extends above the ground or streambed to a point just below the seat area. The top of the piling is capped by a continuous reinforced concrete cap, generally extending the full width of the bridge. This is probably the most common type of pier used because it is economical and easy to construct. As in the wall type pier, the capped pile pier requires very little maintenance unless it is located under an unsealed joint in the deck. There are, however, concerns about the steel piling at the ground line or waterline.

1.4.3 Tee type (Hammer head)

This type of pier is more modern than the wall type, and is commonly used when a taller pier is necessary. It consists of rectangular stem capped with a cantilever-type cap. This is cheaper to construct than a wall type pier because not as much concrete is used, and it is less intrusive to streams. There is generally not much maintenance required for this type of pier unless it located under an unsealed deck joint. In this case, power washing and sealing is required.

1.4.4 Cap and column Pier

This type of pier is commonly used in the construction of the highways, especially for highway overpasses. It consists of three or more round reinforced concrete columns capped with a continuous pier cap forming a rigid frame.

1.4.5 Multiple columns (No cap)

These types of columns are also used commonly on interstate highway bridges, especially on skewed bridges where the length of the pier cap on a cap and column pier became cost prohibitive. In this case, the pier cap is eliminated and a single concrete column is constructed under each beam. These types of piers are also fairly maintenance free unless they happened to be located under a leaking deck joint. Concrete struts between the individual columns up to the level of and behind the barrier can be installed to provide additional lateral restraint for the columns.

1.5 OBJECTIVE OF THE STUDY

The objective of the study is as follows:

(1) To carry out detailed analysis and design of Radial Gate and its supporting pier, along with its foundation.

(2) To prepare program in c^{++}, MS Excel etc. for design of various components.

(3) To check the validity of programs prepared with the help of standard soft wares.

(4) To carry out a few parametric study to understand the behavior of Radial Gate, pier and foundation, when data is manipulated with respect change in levels etc.

(5) To prepare relevant drawings for one of the gate, pier and foundation, for a problem solved.

1.6 SCOPE OF WORK

Based on the objective of study stated above, the scope of work is decided as follows:

(1) To carry out detailed literature survey regarding design of gates, piers, etc.

(2) To study the IS: 4623-1984 for design of various components of radial gate.

(3) To study IS: 13551-1992 for design of piers for support to radial gate.

(4) Based above design approach prepare a program in C^{++} language to design the radial gate for FRL(Full Reservoir level), HFL (High flood level) and Earthquake etc. loads.

(6) To prepare MS Excel worksheet to design trunnion assembly, anchor girder, horizontal anchorages, vertical anchorages, etc.

(7) To prepare MS excel worksheet for analysis and design for pier as per IS: 13551 -1992 codal provision.

(8) To prepare MS Excel worksheet for design of pier open footing.

(9) To prepare detailed drawings of calculated example of radial gate, pier and its footing.

(10) To carryout modeling and analysis the radial gate in STAAD PRO software.

(11) To carryout a few parametric studies by changing radius, trunnion level etc.

1.7 ORGANIZATION OF MAJOR PROJECT

In first chapter, the introduction is discussed wherein classification of the gate, classification of the pier, objective of study and scope of work is discussed to understand the various elements involved in the problem.

In second chapter, the literature review is discussed wherein various technical papers related to radial gate, etc are presented in an order to highlight the chronological developments.

In third chapter, the structural system of radial gate is discussed wherein radial gate sizing and its layout, primary parts like skin plate, tee verticals, horizontal girder, radial arms, and secondary parts like radial arm bracings, horizontal girder bracings, trunnion tie, and gate lifting system are discuses.

In fourth chapter, various loads acting on radial gate viz. hydrostatic hydrodynamic etc are discussed. Similarly various loads acting on pier viz. load from gate.

In fifth chapter, the analysis and design of radial gate is discussed. It includes the analysis and design of all parts of radial gate, trunnion assembly, anchor girder, horizontal anchorage and vertical anchorage.

In sixth chapter, the analysis and design pier is discussed. It includes the analysis and design of pier and its reinforcement requirement. This is followed by design of isolated footing supporting the pier.

In seventh chapter, various parametric studies are discussed.

In eight chapter, results, conclusions and future scope of work is discussed.

2. LITERATURE SURVEY

2.1 GENERAL

Literature Survey is carried broadly under two heads:

(1) Structural Design

In these papers, authors have generally discussed about gates, their locations, trunnion, anchorage arrangement, corrosion protection etc. Also the precautions that shall be taken during design and installation gate, modelling of radial gates, loading condition etc.

(2) Hydrology

In this head, mostly discharge characteristic, spillway profile, dimensional analysis for stage discharge relationship, critical depth for gate opening, Energy momentum equation etc. discussed.

As the objective of the thesis is to design gate and its pier. Therefore, more emphasis is given to the papers under the head 'structural design', and 'hydrology' related papers are covered to a very limited stage.

2.2 STRUCTURAL DESIGN

Chander.K.Sehgal [1] This paper, generally, focuses on common type of spillway gates including Radial gates, Vertical lift gates and Flap gates, inflatable gates and their application. Design consideration for each is explained. Design and application considerations for operation of Radial gates also included. Data covered for gates include gates geometry, gate proportions, location of gate hoist, location of gate trunnion, trunnion anchorage arrangements, use of dogging devices, wave deflectors, flow splitters, corrosion protection including use of cathodic protection associated with radial gates. In this paper, author has discussed mainly about two type radial gates:

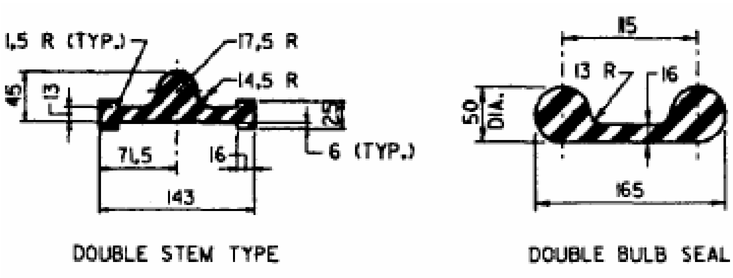
1) Crest type spillway radial gates, and
2) Orifice type spillway radial gates.

He suggested that crest type radial gates have been constructed in very large sizes with areas up to 560 m^2, widths up to 56.5m, and heights up to 22.5m. Orifice type radial gates have been constructed with areas up to 114 m^2, widths up to 12.8m, and heights up to 9.5m and heads up to 135m.

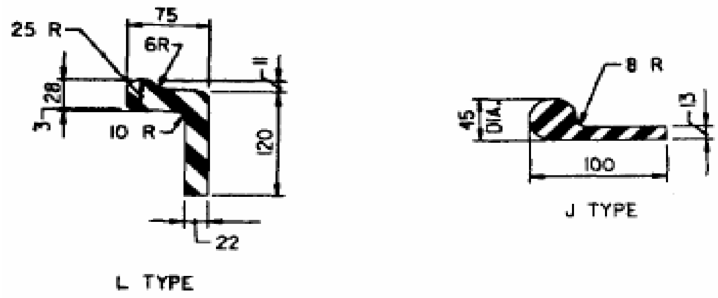
The author discussed about width to height ratio of radial gates. He suggested that the width to height ratio is not critical design consideration. Gates have been successfully designed with width to height ratios less than equals to, and greater than 1. For a given area of opening, however gates with smaller width to height ratios are cheaper, the cost of gate is proportional to approximately width$^{1.28}$ x height, Also the gates with smaller width to height proportions are less subjected to cocking and they provide flow regulation because of greater height of opening for a given flow. The overall cost of spillway, however, is generally smaller for wider gates because wider gates permit fewer and smaller height piers. The gates proportion should be based on the overall plan of the project and not on the gate itself.

In some instances, the trunnions of radial gate are provided eccentric with respect to the centre of skin plate radius. This arrangement uses the moment of the resultant hydrostatic load about the gate trunnion in reducing the hoisting load required to raise the gate. However, the eccentric trunnion complicates the fabrication and installation of gate and embedded parts and should be avoided, except for very large gates closures force of the gate should be verified. Gate trunnion of radial gates must be located sufficiently above the maximum flow nappe to prevent damage to trunnion by debris. The radius of gate is usually 1.25 times the vertical height of the gate above to the crest the sill of gate is usually located slightly downstream of crest.

L type seals as shown in fig. 2.1 (B), because of their better resilience, are preferred for radial gates side seals. Bar type bottom seals chamfered, as shown in fig. 2.1 (C), are used on radial gates. In radial gates, provided on orifice spillways, the bottom seal may be bar type or a flat type seal attached to the sill. Because of high heads associated with the orifice spillway, a flat seal is usually better than the bar seal for flow efficiency. Radial gates for orifice spillways require top seals. Two top seals are usually provided, one attached to embedded lintel beam to prevent leakage at the top while the gate is partially open and other attached to gate, Skin plate to ensure tight sealing while the gate is fully closed. The top seal attached to embedded lintel beam, should preferably be double-stem type as shown in fig. 2.1 (A). A J-type as shown in fig. 2.1 (B) is not suitable in this case because the bulb of J type seal is subjected to rolling and damages as the skin plate moves against it. For orifice radial gate square bulb seals may be used as side seals to well maintain better contact with the top seal attached to the lintel so that leakage is reduced at the top corners, when gate is in an open position.



(A)



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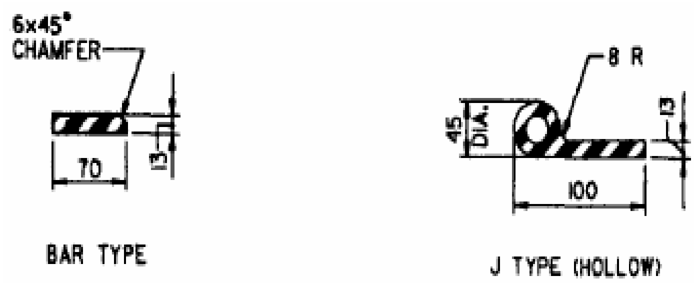


(C)

Figure 2.1. Rubber seals used in radial gate

B.T.A.Sagar [2] This paper discusses the various type of high head gates, including slide gates, fixed wheel gates, radial gates etc. Hydraulic design considerations for these gates are discussed below:

Vibration of gates

Vibration of gates can lead to structural failure of not only the gate components but the surrounding structure. Vibrations may be caused by several factors:

1) Shear flow under the gate leaf,
2) Shifting of flow control point under the gate leaf,
3) Excessive changes in the magnitude of hydraulic down pull for small vertical

movements of the gate,

4) Lack of adequate aeration and consequent pressure fluctuations in the zones under gate leaf or immediately down stream of leaf, and
5) Impingement of high velocity jets on down stream gate components.

Nonclosure of gate

Frequent reason for nonclosure of the gate are mechanical causes such as insufficient gate buoyant weight excessive wheel seal frictions, misalignment of gate guides and frames tilting and jamming of gates and physical obstruction in gate slots and on gate sill. Nonclosure of gates can sometimes be due to hydraulic uplift forces that can occur at some gate leaf openings.

Hydraulic down pull

Hydraulic down pull is the net hydrodynamic force acting on the gate leaf vertically or parallel to hoist pull at any gate leaf position. The down pull force varies with gate leaf magnitude depends on many factors in some situations. The net force may act upward causing uplift on the gate leaf.

Henning Fosker, Halvard Bjorndal, Terje Ellefrod,Kjell Knutsen [3]

This paper deals with experimental type work. Authors have measured bearing friction in different positions in radial gates. Arms are normally designed to withstand bending moments from nominal friction on bearings. Where experience shows that lack of lubrication leads to increased friction and even seizure of the bearings. The bending moments produced by bearing friction imposed on the gate arms are beyond the moment capacity resulting in bearing failure. Author used two strain gauges one at upper side of gate arm and other is lower side of gate arm. Each strain gauge measures the surface mechanical stresses parallel to the main stress direction in gate arm and calculates the bending stress Vs gate opening from that stress. Authors calculated the stress variation Vs gate opening. From that they calculated friction coefficients for different type bearing materials. Also, in other experiment authors calculated friction coefficient Vs gate opening. For original bearing, original bearing with lubricated and last was new bearings. From that they concluded that the initial and dynamic friction of the original unlubricated bearings is almost constant. When lubricated, the original bearing has almost the same static friction as unlubricated while the dynamic friction drops by 25%. Above method stated by author for bearing friction by means of strain gauges provides dam owner with better diagnostic technique. Experience has shown that the method will detected bearing failure at any early stage before the friction moment exceeds the gates arms bending moment capacity. The method is characterized by high reliability and accuracy.

Mostafiz R.Chowdhury Robert L.Hall [4] This paper demonstrates how the use of appropriately scaled model experiments can effectively be used to evaluate the dynamic performance of a gate system. The physical scale models studied in this investigation are exact replicas of the innovative actuated wickets to be placed at the lower Ohio River. Operating shapes of the flow induced gate motion at critical configurations are compared with the wet characteristic modes of the wicket to determine the potential resonant vibration problem. These wet modes are extracted such that the effects of structural and hydraulic boundary conditions of the flow field are accounted for in the modal experiment. Experiment results indicate that random energy contents of an ambient excitation can readily be used to estimate the physical characteristic modes of a wet structure. The study also shows that operational deflected shapes for potentially alarming excitation can be compared with such wet natural modes to identify resonance problems. This evaluation procedure demonstrates the convenience in using ambient vibration for the detection of self excitation problems during the service life of the system.

Farhang Daneshmand, Shailendra K Sharan,Mohammad H Kadivar [5]
This paper deals with modelling of fluid structure because the interaction between fluid and gate can significantly affect the response of the structure and hence has to be properly taken into account in the analysis. It means that the fluid structure interaction can significantly affect the dynamic response of the gates and needs to be taken into account in the analysis. Two and three dimensional solid elements were used to model the structure. Whereas the fluid was modeled by two and three dimensional fluid elements. This paper deals with vibration investigation of spillway radial gate. The problem was solved for fully open, fully closed and opening for 30% opening. It was also concluded that gate is free from resonance. The pressure fluctuations on the gate were not large under the normal discharge of the gate local opening. It was also concluded that intensive pressure fluctuations exist in the vicinity of the natural frequency of the gate.

Crest and Head Gates Engineering Manual [6] This is engineering manual from Department of the Army Corps of Engineers. It is basically related to drawings of gate and material used in gate. The principal objectives of this chapter are to outline for the procedure involved in developing a gate that will meet the requirements for dependable operation, minimum maintenance, and long life; to standardize on the use of materials and working stresses; and to show typical details recommended.

R.S Varshney [7] This book covers design of gates and valves. In design of radial gates the book discusses important features like skin plate, tee verticals, horizontal girder radial arms trunnion anchor girder and hoist equipments. The book also discusses about the maintenance of each part. Attempt has been made to first present clearly and concisely the fundamental principles of design as outlined in code of bureau of Indian standards, second to demonstrate the practical application of these criteria by solved examples. The Design examples have been worked out at every stage, from the basics. The methods used in current design practice have been explained, and relevant codes and specification governing practical design have been referred to at every step.

Dean Peabody [8] This book was referred to understand the procedure of the design of column under biaxial eccentricity.

IS 4623-1984 [9] This standard provides guidance for the structural design criteria of radial gates. This code considers materials, design considerations of all components of radial gate.

IS 13551-1992 [10] This standard provides guidance for the structural design criteria of Spillway pier and crest. This code discusses various aspects of loads on pier, and design clauses for the same.

2.3 HYDROLOGY

M.A Shahrokhnia, M.Javan [11] In this paper, dimensional analysis was used to obtain stage discharge relationship under submerged and free flow conditions in radial gates to develop a management tool. Experimentally data from a laboratory flume and the indicial method of dimensional analysis were used for this purpose. The resulting equation relates the discharge (or critical depth) to upstream and downstream water depth and gate opening. This equation was then validated by experimental data obtained from field radial gates and compared with the conventional gate equation. Results showed that there was a good agreement between dimensionless equation and field and laboratory data under submerged or free flow conditions. Dimensionless equations are more general and accurate than the conventional ones when there is not an accurate estimation of discharge coefficient.

Tony L.Wahl [12] The Energy Momentum method for calibrating submerged radial gates was refined using a large laboratory data set collected at the Bureau of Reclamation Hydraulics Laboratory. The original E-M method was accurate in free flow, and when the gate significantly controls submerged flow but for large gate openings with low head loss. Though the gate discharge prediction errors were sometimes large, several empirical factors were investigated with the laboratory data, including the combined upstream energy loss and velocity distribution factor and the submerged flow energy correction. The utility of the existing upstream energy loss and velocity distribution factor relation was extended to larger Reynolds numbers. The relation between the relative energy correction and the relative submergence of the vena contracta was found to be sensitive to the relative jet thickness. A refined energy correction model was developed which significantly improved the accuracy of submerged flow discharge prediction. Although the focus of this work was radial gates the energy correction concept and these refinements potentially have application to all submerged sluice gates.

A.J Clemmens,T.S Strelkoff,J.A Replogle [13] Calibration equations for free flowing radial gates typically provide sufficient accuracy for irrigation district operations. However, many water purveyors have difficulty in determining accurate discharges when the downstream water level begins to submerge the gate. Based on experimental laboratory studies authors have developed a new calibration method for free flowing and submerged radial gates that allows for multiple gates and widely varying upstream and downstream channel conditions. The method uses the Energy Momentum method. An iterative solution is required to solve these two equations for free flow are described along with an additional energy adjustment. For the transition to submerged flow, an application is used to describe the new procedure and how it overcomes the limitations of current energy based methods.

3. STRUCTURAL SYSTEM OF RADIAL GATE

3.1 GENERAL

The radial gate has an upstream skin plate bent to an arc, with convex surface of the arc on the upstream side. The centre of the arc is generally at the centre of the trunnion pins, about which the gate rotates. The skin plate is supported by stiffeners either horizontal or vertical or both. If horizontal stiffeners are used, these are supported by vertical diaphragms which are connected together by horizontal girders transferring the load to the two end vertical diaphragms as shown in fig. 3.1. The end beams are supported by radial arms, emanating from the trunnion hubs located at the axis of the skin plate cylinder. If vertical stiffeners are used, these are supported by horizontal girders which are supported by radial arms as shown in fig.3.1. The arms transmit the water load to the trunnion anchorage girder. Suitable seals are provided along the curved ends of the gate and along the bottom.

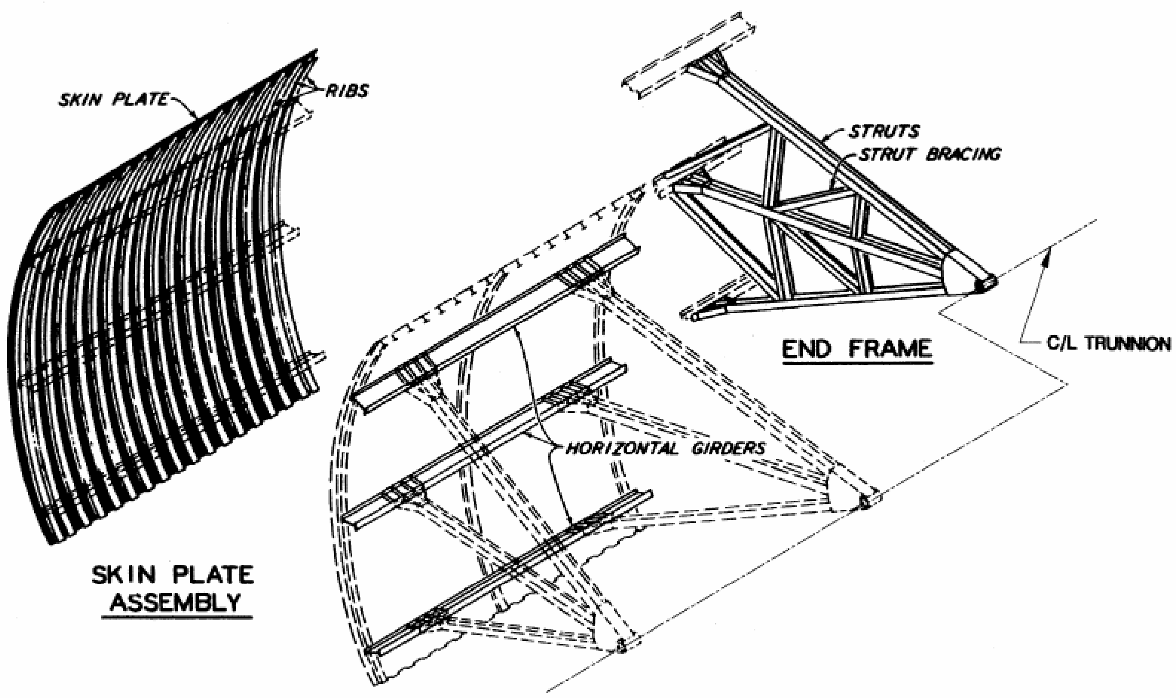


Figure 3.1 Primary radial gate components

3.2 GEOMETRY AND COMPONENTS

3.2.1 Primary components

The principal elements of a conventional radial gate are the skin plate assembly, horizontal girders, end frames, and trunnions as shown in fig.3.1. The skin plate assembly, which forms a cylindrical surface, consists of a skin plate stiffened and supported by curved vertical ribs. Structurally, the skin plate acts compositely with the ribs (usually structural Tee sections) to form the skin plate assembly. The skin plate assembly is supported by the horizontal girders that span the gate width. The downstream edge of each rib is attached to the upstream flange of the horizontal girders. The horizontal girders are supported by the end frames as shown in fig.3.1. End frames consist of radial struts or strut arms and bracing members that converge at the trunnion which is anchored to the pier through the Anchor girder. The end frames may be parallel to the face of the pier (support the horizontal girders at the ends) or inclined to the face of the pier (support the horizontal girders at some distance from the end with cantilever portions at each end). The trunnion is the hinge upon which the gate rotates. The trunnion is supported by the Anchor girder.

3.2.2 Other structural members

Structural bracing members are incorporated to resist specific loads and/or to brace compression members. Certain bracing members are significant structural members, while others can be considered secondary members in which mainly horizontal girder lateral bracing, down stream vertical truss, end frame bracing, trunnion tie and gate lifting system are considered. The Horizontal girder lateral bracing may simply provide lateral bracing for the girders or may serve to carry vertical forces from the skin plate assembly to the end frame. The end frame bracing members are ordinarily designed to brace the struts about weak axis to achieve adequate slenderness ratio as shown in fig. 3.2. The trunnion tie is a tension member provided on some gates with inclined strut arms that is designed to resist lateral end frame reaction loads. In standard lifting arrangements presently recommended for new construction are the wire rope hoist and hydraulic hoist system.

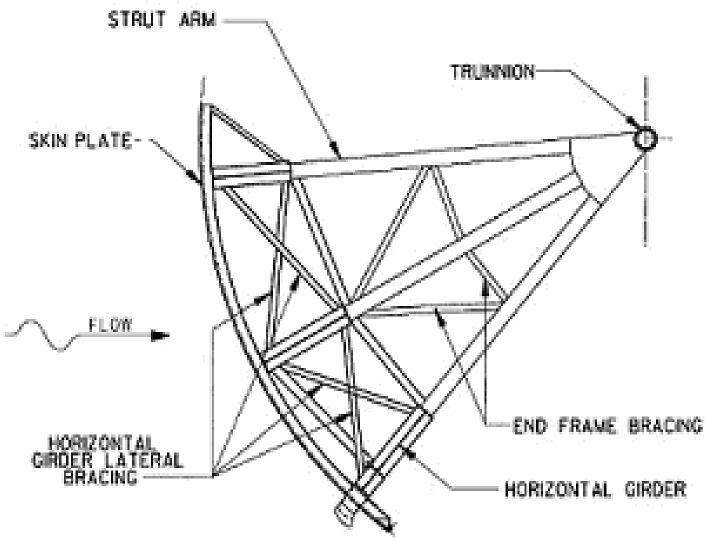


Figure 3.2 Bracing system in radial gate

3.3 RADIAL GATE SIZING AND LAYOUT

The sizing of the gates is an important early step in the design process. Gate size affects other project components, project cost, operation, and maintenance of the project. The following paragraph includes various considerations that should be taken into account while selecting a practical and economical radial gate size. The best alternative is not necessarily a gate with the lightest gate weight-to-size ratio.

3.3.1 Gate size

The hydraulic engineer will normally establish the limiting parameters for gate height and width. Within those limits, various height-to-width ratios should be studied to find the most suitable gate size for the project. The structural designer should coordinate closely with the hydraulic engineer in determining the basic limiting requirements for size and shape. The size, shape, and framing system of the gates should be selected to minimize the overall cost of the spillway, rather than the gate itself. Determination of gate size will also consider practical operation and maintenance considerations specific to the project.

3.3.2 Gate width

The gate width will be determined based on such factors as maximum desirable width of monoliths, length of spillway, bridge spans, drift loading, overall monolith stability, and loads on trunnions and anchorages. On navigation projects, the gates may be set equal to the width of the lock, so that one set of bulkheads can serve both structures. It is usually desirable to use high gates rather than low gates for a given discharge, since the overall spillway width is reduced and results in a more economical spillway.

3.3.3 Gate radius

The skin plate radius will normally be set equal to or greater than the height of the gate. The radius of the gate will also be affected by operational requirements concerning clearance between the bottom of the gate and the water surface profile. This is often the case for navigation dams on rivers where the gate must clear the flood stage water surface profile to pass accumulated drift. On such projects requiring larger vertical openings, it is common to use a larger radius, up to four times the gate height, to allow for a greater range of opening. This will require longer piers for satisfactory location of the Anchor girder.

As per IS 4623-1984 clause 5.3, the radius of the gate, i.e. the distance from the centers of the trunnion pins to the inside face of the skin plate shall as far as possible be from H to 1.25H consistent with the requirements of the trunnion location outlined below where H is the vertical distance between the top of the gate and the horizontal through the sill.

3.3.4 Trunnion location

It is generally desirable to locate the trunnion above the maximum flood water surface profile to avoid contact with floating ice and debris and to avoid submergence of the operating parts. However, it is sometimes practical to allow submergence for flood events, especially on navigation dams. Designs allowing submergence of 5 to 10 percent of the time are common. Gates incorporating a trunnion tie should not experience trunnion submergence. If other considerations do not control, it will usually be advantageous to locate the trunnion so that the maximum reaction is approximately horizontal to the Anchor girder (typically about one-third the height of the gate above the sill for hydrostatic loading). This will allow for simplified design and construction by allowing the trunnion post tensioned anchorage to be placed in horizontal layers.

As per IS 4623-1984 clause 5.2, the trunnion of the gate shall be so located that under conditions of maximum discharge over the spillway barrage, these should preferably remain at least 1.5m clear of the water profile. With gates having the trunnions on the upstream side the trunnions have to remain submerged in water but suitable precautions should be taken to prevent corrosion of the trunnion parts under such conditions.

The trunnions shall be so located that the resultant hydraulic thrust through the gate in the closed position for reservoir full condition lies as close to the horizontal as possible. This will reduce the upward or downward force that will otherwise be imposed on the anchorage girders.

In the case of conduits and tunnels, the trunnion shall be located clear of the water profile under free flow conditions. However, in case of pressure conduits these shall be designed for submerged condition.

The location of the trunnions shall be such as to allow the gate to be fully raised or lowered without interfering with the spillway or Hoist Bridge or any other part of the civil structure housing the gate.

3.3.5 Location of the sill

As per IS 4623-1984 clause 5.4, the sill of the gate shall preferably be located slightly downstream of the crest, to avoid cavitations of the downstream glacis.

The sill shall, as far as possible, be located so that a vertical plane tangent to the upstream face of the skin plate will intersect the spillway at or downstream from the crest. This requirement would place the sill downstream of the crest. Operating clearances from the bridge and the location of the hoist may require the sill to be shifted further downstream.

The distance from the centre line of crest to the centre line of the sill shall be as small as possible in order to economize on the height of gate and pier size.

3.3.6 Operating equipment location

The type and position of the gate lifting equipment can have a significant effect on gate forces as the gate is moved through its range of motion. As stated previously, the two gate lifting systems recommended for new construction are the wire rope hoist system and the hydraulic hoist system. Many new gate designs utilize hydraulic cylinder hoist systems because they are usually cost effective. However, these systems have some disadvantages and are not suited for all applications.

As per IS 4623-1984 clause 5.5, in case of crest gates, the hoists may be installed on the road way or on the piers or on an under-deck below the roadway. The hoist shall be so located that as far as possible the hoisting force is applied to the gate at the largest possible radius and the hoisting angle does not change much during the travel of the gate. The hoisting may also be located on the downstream side of the gate depending on the site requirements.

As per IS 4623 1984 clause 5.1.2, the gate, in general, shall satisfy the following requirements:

(a) It shall be reasonably watertight;

(b) It shall be capable of being raised or lowered by the hoist at the specified speed;

(c) Power operated gates shall normally be capable of operation by alternate means in case of power supply failure; and

(d) If meant for regulation, it shall be capable of being held in position within the range of travel to pass the requited discharge without cavitations and undue vibration.

3.4 SKIN PLATE ASSEMBLY

For the 2-D approximate model, the skin plate and ribs are assumed to have zero curvature. The skin plate serves two functions. First, each unit width of skin plate is assumed to act as a continuous beam spanning the ribs in the horizontal direction as shown in fig.3.3. Second, the skin plate acts as the upstream flange of the ribs. The ribs, with the skin plate flange, are continuous vertical beams that are supported by the horizontal girders as shown in fig.3.3. A portion of the skin plate is considered to act as the upstream flange of the rib.

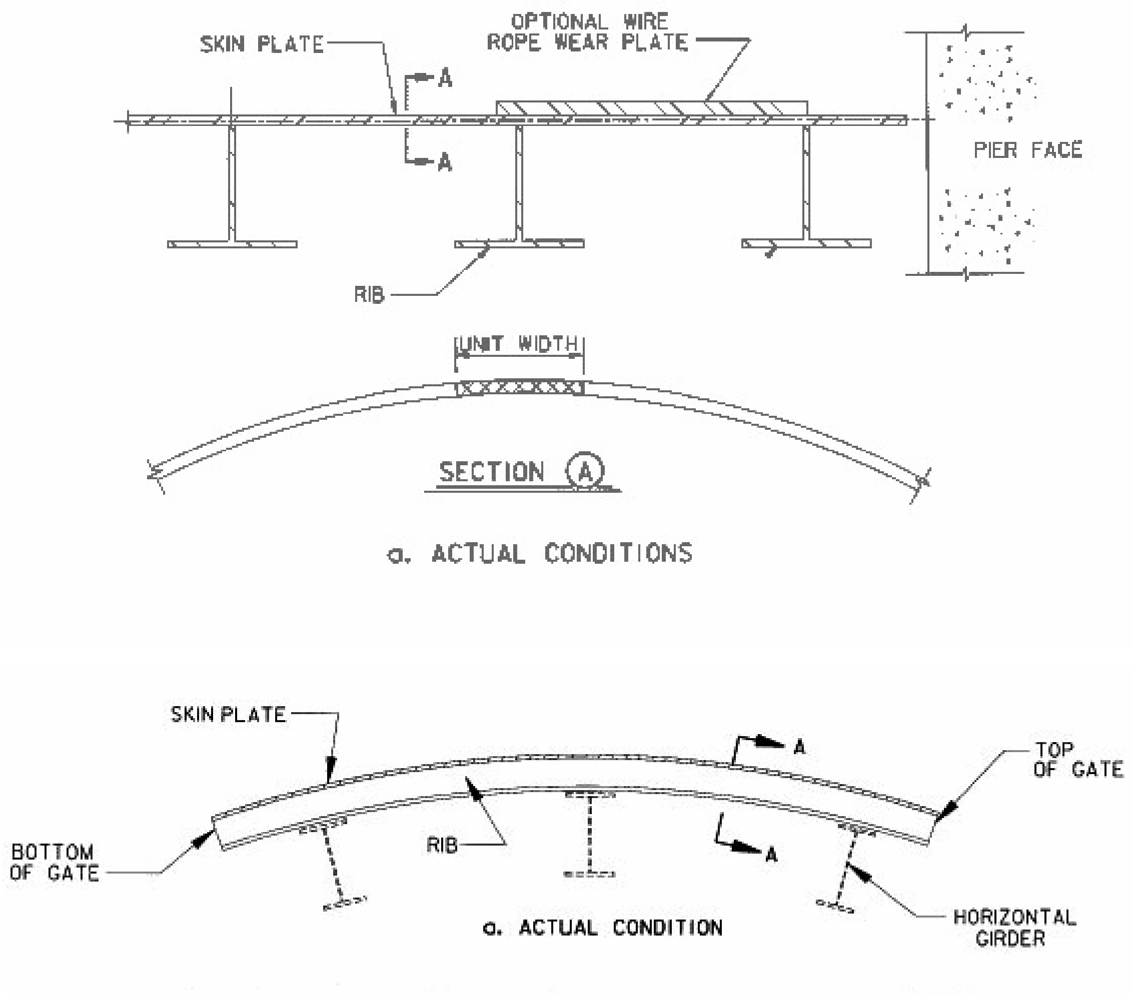


Figure 3.3 Skin plate assembly

The skin plate assembly consists of the skin plate and vertical ribs. Horizontal intercostals are not recommended since material savings realized in the design of the skin plate are offset by higher fabrication and maintenance costs. The designs of the skin plate and ribs are interrelated. The required skin plate thickness is dependent on the rib spacing (skin plate span), and the required rib size is dependent on the skin plate thickness since an effective portion of skin plate acts as a rib flange.

3.4.1 Skin plate design considerations

The skin plate design stress is based on the negative moment at the supports for equally spaced interior ribs (fixed-end moment). The spacing between the exterior ribs at the ends of the gate is adjusted such that the moment does not exceed the fixed-end moment of the interior spans. For gates with a wire rope hoist, thicker plate and/or closer rib spacing is normally required under the wire rope due to the rope pressure exerted on the plate. Because of the varying loading on the skin plate, it is economical to vary the thickness of the plate over the height of the gate. For gates less than approximately 3m high, it is generally economical for the entire skin plate to be of one thickness. It is recommended to maintain a minimum thickness of 8 mm, while a thickness greater than 20 mm will rarely be required for any gate.

3.4.2 Rib design considerations

Although wide-flange or built-up sections are acceptable, structural tee sections with the web welded to the skin plate are recommended for ribs. In determining member geometric properties, an effective width of skin plate is assumed to act as the upstream flange of the vertical rib. The effective width b_e of skin plate shall be based on width-to-thickness ratios for compact or non compact limits that are consistent with rib design assumptions. For rib sections that are considered as compact,

3.4.2.1 *I.S considerations*

As per IS 4623 1984 clause 5.6, the Skin plate shall be designed for the following two conditions:

(a) In bending across the stiffeners horizontal girders or as panels.

(b) In bending, co acting with the stiffeners and/or horizontal girders.

Clause 5.6.3, because of the constant span under varying loading on the skin plate, it is economical to use two or more sizes of plates with a minimum of 8 mm thick plate. For the gates in tunnels and conduits the same thickness of plate may preferably be used throughout.

Clause 5.6.5, for conditions the width of the skin plate co acting with beam or stiffeners continuous or simply supported shall be assumed, and stresses due to beam action calculated. Alternatively, the co acting width of the skin plate in non panel fabrication may be restricted to the following values.

(a) $40t + B$

Where,

t=thickness of plate, and

B=Width of the stiffeners flange in contact with the skin plate.

(b) 0.11 span.

(c) Center to center of stiffener or girders.

Clause 5.6.8, take care of corrosion, the actual thickness of skin plate to be provided shall be at least 1.5 mm more than the theoretical thickness computed based on the stresses specified under 'Dry Condition' in Appendix A. Alternately, the design stresses specified under 'Wet Condition' in Appendix A shall apply, for which case corrosion allowance shall not be necessary. The minimum thickness of the skin plate shall not be less than 8 mm, inclusive of corrosion allowance, when considered.

Clause 5.6.9, the stiffeners may, if necessary, be of a built-up section or be of standard rolled section that is, tees, angles, channels, etc.

3.4.3 Skin plate fabrication

All skin plate splices shall be full penetration welds and smooth transitions shall be provided at splices between plates of different thickness. Corrosion is controlled by protective coating systems and maintenance, and increasing skin plate thickness to allow for corrosion is not recommended. However, due to inevitable wear and deterioration, it is appropriate to increase the skin plate thickness along the bottom of the gate or under wire ropes for gates with wire rope hoists.

3.4.4 Ribs fabrication

Ribs should be spaced and proportioned to provide adequate clearances required for welding and maintenance painting, even with a slight increase in steel quantity. The depth of the ribs must be sufficient to provide access for welding or bolting the rib flanges to the supporting girders.

3.5 HOROZONTAL GIRDER

Girders provide support for the skin plate assembly and transfer all loads from the skin plate assembly to the end frames. The girders act as rib supports and are generally located to achieve an economical design for the ribs. However, the location of girders also affects the load on each girder since the rib reactions are the girder loads. The overall economy considering the effect on girder design should be considered. The end frame (strut) design affects girder forces since the struts are the girder supports.

3.5.1 Design considerations

Horizontal girders are singly or doubly symmetric prismatic members that are designed primarily for flexure about their major axis. The distribution of flexure along the length of the girder is significantly influenced by the orientation of the end frames. Maximum girder moments will result with struts that are parallel to the pier face. The maximum girder moment is reduced (moments are redistributed) if the struts are inclined and will be minimized if the struts intersect the girder at approximately one-fifth the gate width from the pier face. With inclined struts, the limit state of lateral torsional buckling of the girder should be checked since a significant length of the downstream flange of the girder will be in compression. The unsupported length of the downstream flange is the distance between supports consisting of downstream vertical truss members and the end frame strut. (The upstream flange is laterally supported nearly continuously by the vertical ribs). The downstream vertical truss is designed primarily to resist forces that occur when the gate is supported at only one end and also provides lateral stability and resistance to torsional buckling for the girders. The tension flange of critical girders may be fracture critical and should be designed and fabricated accordingly.

3.5.1 I.S considerations

Clause 5.7.1, the number of girders used shall be a minimum to simplify fabrication and erection and to facilitate maintenance.

The girders may be so spaced that the bending moment in the vertical stiffeners at the horizontal girders as a continuous beam, are about equal. (5.7.3)

When more than three girders are used, it may become necessary to allow the bending moment in the vertical stiffener at the top most girder, of a value higher than at the other girders, so as to adequately stress the skin plate.(5.7.3)

The girders shall be designed taking into consideration the fixity at arms support. Where inclined arms are used, the girders should also be designed for the compressive stress induced. (5.7.4)

The girders shall be checked for shear at the points where they are supported by the arms. The shear stress shall not exceed the value specified in Appendix A. (5.7.5)

The horizontal girders should also be suitably braced to ensure rigidity. (5.7.6.1)

The spacing and design of the bearing and intermediate stiffeners shall be governed by relevant portions of IS: 800-1984. (5.7.6.2)

3.6 END FRAMES (RADIAL ARMS)

The end frames transfer loads from the girders and skin plate assembly to the trunnion. End frames include the struts and associated bracing and the trunnion hub flange plates. The arrangement and orientation of the end frames affects the magnitude and distribution of end frame and horizontal girder forces, trunnion fabrication, trunnion pin binding, and thrust forces into the pier. To achieve an economical design, these effects should be recognized considering all applicable load cases.

3.6.1 Design considerations

Struts include flexure about both axes and significant axial forces. The strut strong axis moments shall be determined from the girder model and the axial forces and weak axis moments shall be determined by the end frame model. End frame bracing should be spaced to achieve adequate weak axis slenderness ratios for the struts and must be designed to resist calculated forces. Bracing members may include significant flexural forces depending on member sizes, connection rigidity, and trunnion friction. Critical bracing members subject to flexural tension can be fracture critical and should be designed and fabricated accordingly. Trunnion hub flanges shall be proportioned to resist the strut flexural, shear, and axial loads.

3.6.2 I.S considerations

As many pairs of arms as the number of horizontal girders, shall be used, unless vertical end girders are provided. (5.8.1)

The arms may be straight or parallel. Inclined arms may conveniently be used to economize on the horizontal girders where other conditions permit. (5.8.2)

The arms shall be designed as columns for the axial load and bending moment transmitted by the horizontal girders and shall be in accordance with ARE: 800-1984, taking into consideration the type of fixity to the girder. (5.8.3)

The total compressive stress shall be in accordance with IS: 800-1984 for various values of l/r where l is the effective length, and r is the least radius of gyration. For bending stresses, the stresses specified in Appendix A shall apply. (5.8.4)

The arms if inclined may be fixed to the horizontal girders at about one-fifth of the width of the gate span consistent with the design requirements from each end of the girder. (5.8.5)

The joints between the arms and the horizontal girders shall be designed against the side thrust due to the inclination of the arms, if inclined arms are used. (5.8.6)

The arms shall be suitably braced by bracings in between the arms. The bracings connecting the arms shall be so spaced, that the l/r ratio of the arms in both the longitudinal and transverse directions is nearly equal. (5.8.7)

In case of gates likely to be overtopped, the end arms and other components should suitably be protected by means of side shields to prevent direct impact of water on arms. (5.8.8)

The thrust block or trunnion tie is required only if inclined arms are used with the gate. Alternatively, the lateral thrust can be directly transferred to the concrete pier through bearing from a plate embedded in concrete. (5.15.1)

The thrust block is used when the horizontal force from the trunnion is directly transferred to a yoke girder immediately behind the trunnion. (5.15.2)

The thrust block is fixed to the anchorage girder and is designed to withstand the bending and shear force caused by the side thrust on the trunnion due to the inclined arms. Ultimately the side thrust is transferred through the anchorage girder to the concrete as a bearing stress. A thrust washer should be used between the trunnion hub and trunnion bracket to transfer the thrust. (5.15.2.1)

3.6.3 Fabrication and maintenance considerations

Strut bracing members can be structural tee or wide flange sections; however, using wide flange sections with the same depth as the struts generally facilitates fabrication of connections. Trunnion hub flanges as shown in fig. 3.9 should be proportioned to provide a surface perpendicular to each strut with clearance provided between the ends of intersecting struts. Critical connections include the strut-to-girder connection and the strut-to trunnion hub flanges. The latter connection generally involves full-penetration butt splices involving thick plates, and welding requirements should be developed to minimize associated problems. The layout of end frames, parallel to the pier or inclined, is an important consideration regarding the overall design of the structure. Each configuration has unique advantages and disadvantages.

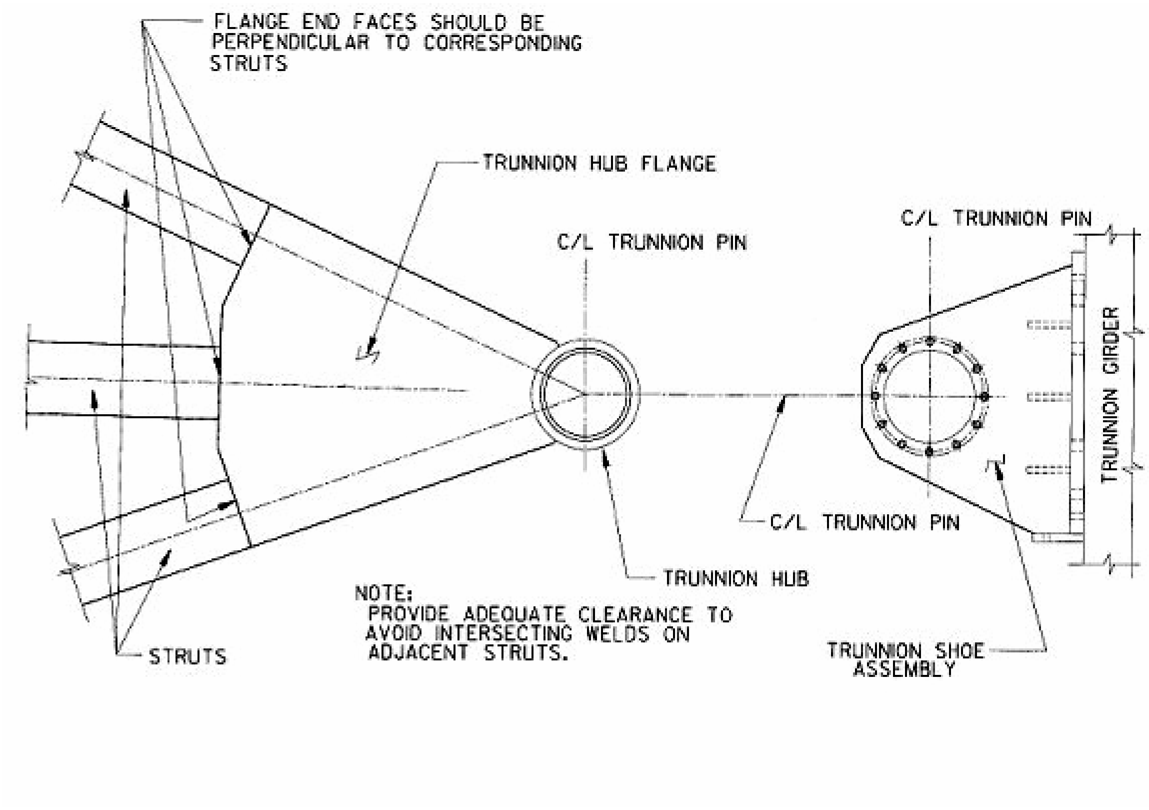


Figure 3.4 Trunnion Hub

3.6.4 Parallel End Frames

End frames that are parallel to the pier and perpendicular to the horizontal girders are positioned such that interference to flow and debris accumulation on the struts is minimized. Fabrication and layout is relatively simple and struts that are parallel to the pier face transfer minimal lateral thrust into the piers. However, economy of the gate is sacrificed due to large flexural loads in the struts and girders, and clearance for maintenance painting between the pier and struts is limited.

3.6.5 Inclined End Frames

By inclining the end frames from the pier face, flexural forces are redistributed and the girder and strut flexural forces are reduced. The point of intersection of the strut and girder can be selected such that the girder moment at mid span is equal in magnitude to the cantilever moment. With this arrangement, maximum economy of the girder design is achieved, and flexure in the end frames is minimal. A second option is to locate the intersection where there would be zero rotation in the horizontal girder. This would theoretically eliminate bending in the end frames. The side thrust component of the gate reaction introduced by inclining the end frames is transmitted directly to the pier or is resisted by a trunnion tie. Where the lateral thrust appreciably increases the pier requirements, it may be preferred to reduce the degree of inclination to achieve a more economical design. While inclined end frames are usually desirable for flood control projects, they are often not feasible for navigation dam projects where floating debris is a concern.

3.7 SEALS

The seals used in radial gates follow standard details. However, there will be some differences based on operational requirements and the degree of water tightness required for the specific project.

3.7.1 Side seals

The standard side-seal arrangement is shown in Fig. 3.5. This arrangement employs standard, readily available, J-bulb seals. The seals may have a hollow bulb where increased flexibility of the bulb is desired such as low head applications. The seals are available with the rubbing surface coated with fluorocarbon (Teflon) to reduce friction. This is beneficial especially for high head gates.

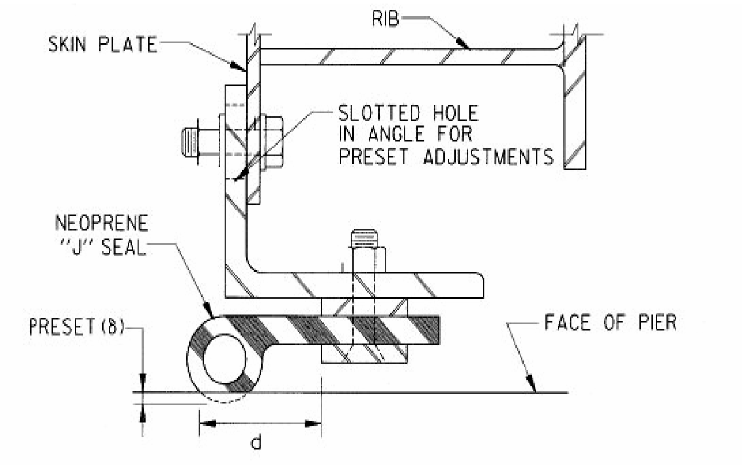


Figure 3.5 Side seal configuration

3.7.2 Bottom seals

The recommended bottom-seal configurations are shown in Fig. 3.6.

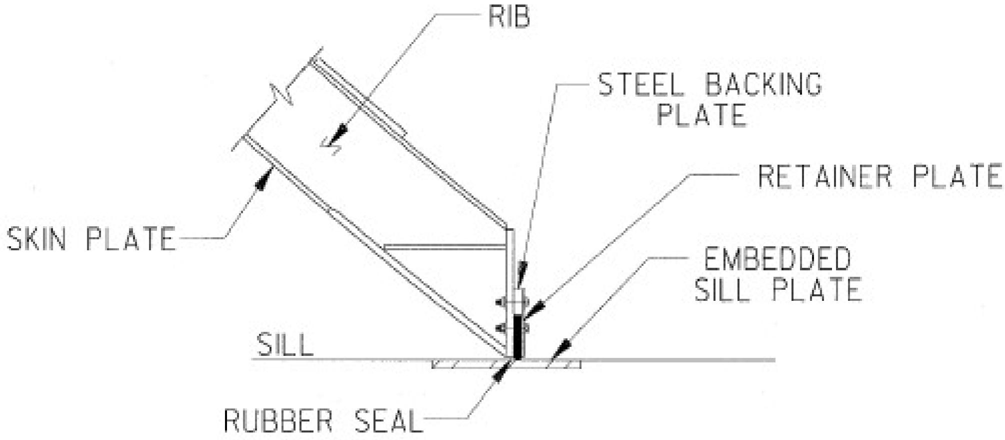


Figure 3.6 Bottom seal configuration

3.7.3 I.S considerations

The seals shall be fixed so as to bear tightly on the seal seats to prevent leakage in the fully closed position of the gate For reducing seal friction clad seals may be used. The cladding may be of brass, bronze, stainless steel or fluorocarbon. (5.16.1)

3.8 TRUNNION ASSEMBLY

The trunnion assembly is made up of a fixed trunnion yoke that is bolted to the Anchor girder, a trunnion hub, and a trunnion pin with a bushing or bearing. Bushings or bearings are provided to minimize friction and wear during rotation of the gate about the trunnion pin. The trunnion assembly is designed to transmit gate load directly to the Anchor girder. Fig. 3.7 illustrates typical details for a cylindrical bushing assembly.

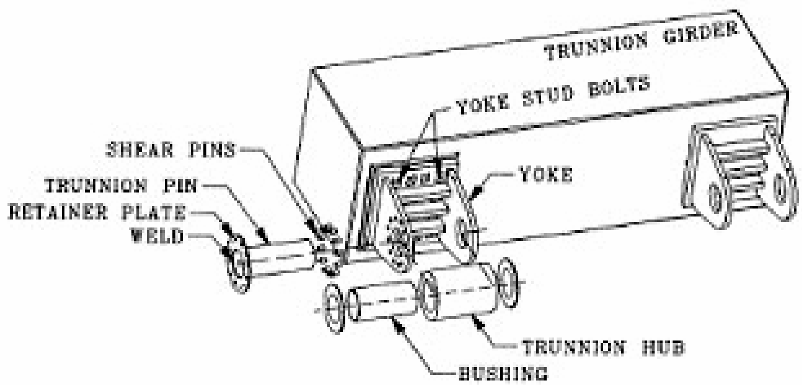


Figure 3.7 Trunnion assembly with cylindrical bushing

3.8.1 Structural Components

The layout of structural components is shown in fig. 3.8.

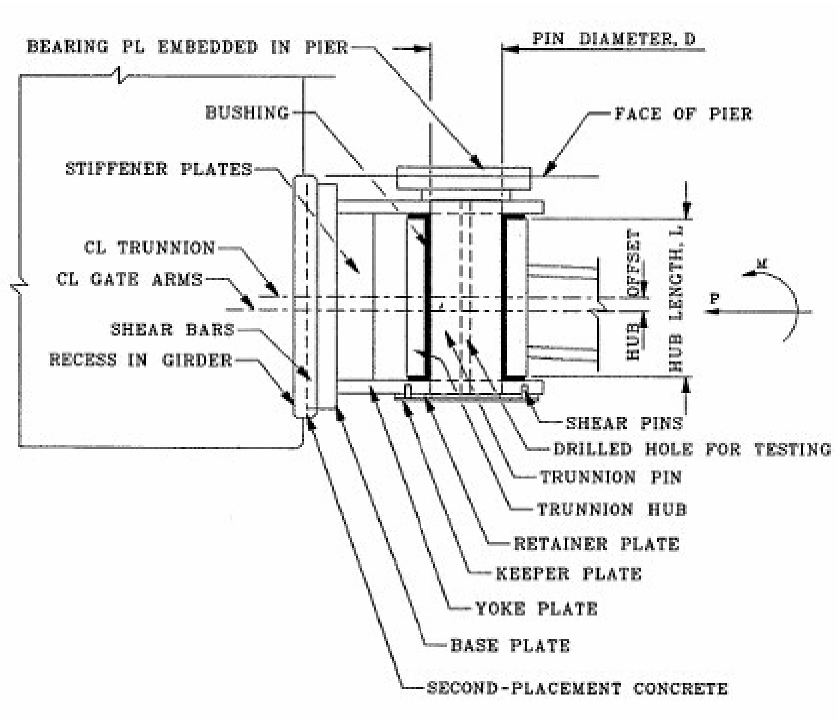


Figure 3.8 Trunnion assembly structural component layout

3.8.1.1 *Trunnion yoke*

The yoke is typically fabricated of welded structural steel and consists of two parallel plates (yoke plates) that are welded to a stiffened base plate as shown in fig. 3.9. The yoke plates are machined to receive the trunnion pin and associated components. The assembly is bolted to the Anchor girder after final installation adjustments have been made by horizontal and

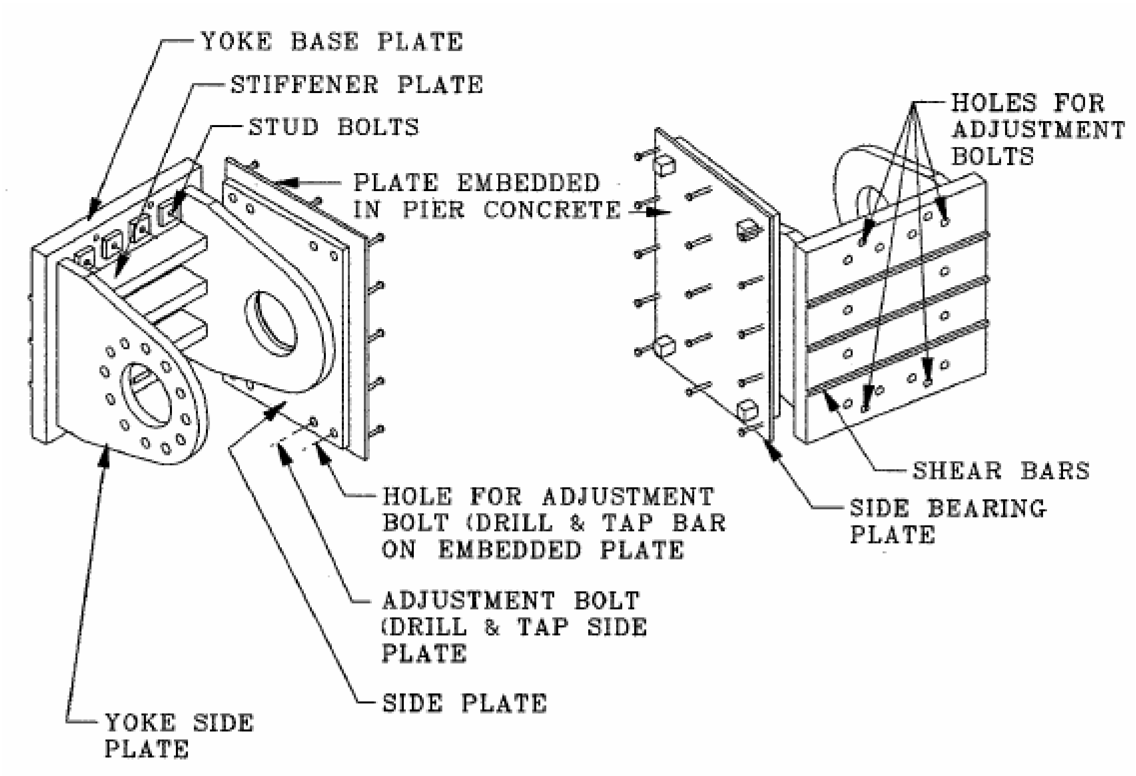


Figure 3.9 Typical trunnion yoke assembly

vertical jackscrews. Shear bars that are welded to the base plate may be required to resist shear at the interface with the Anchor girder for loads that produce resultant forces that are not normal to the bearing surface.

3.8.1.2. *Trunnion hub*

The hub can be fabricated of cast, forged, or structural steel. Castings and forged steel are typically more costly than welded steel construction. The inside bore is machined to tolerance for proper fit with the trunnion bushing or bearing.

The hub is welded to the gate arm extensions and is joined to the yoke with the trunnion pin. The hub is typically wider than the gate arm extensions to allow for a more uniform distribution of stress and to provide clearance for a welded connection. A bushing or bearing is provided between the hub and trunnion pin to reduce friction. The trunnion hubs and yokes shall be stress relieved by heat treatment and machined after fabrication welding is completed.

3.8.1.3. *Trunnion pin*

The trunnion pin transfers the gate loads from the hub to the yoke side plates. A retainer plate that is welded to the pin is fitted with shear pin to prevent the trunnion pin from rotating. The retainer plate and pin are connected to the yoke with a keeper plate.

3.8.1.4. *Trunnion bushing*

Bushings are provided between the trunnion pin and hub and between the hub and yoke plates. The bushings provide a uniform bearing surface and reduce torsional loads due to friction. The required thickness will depend on the size of the trunnion pin.

3.8.2 Design considerations

3.8.2.1 *Bushing*

The bushing is designed as a boundary lubricated bearing. The bushing is proportioned such that the actual bearing stresses do not exceed allowable stresses. The bearing pressure between the trunnion pin and bushing is based on an effective area and the gate reaction forces. The effective bearing area is commonly based on the dimensions of the projected width of the trunnion pin onto the bushing (or bearing). The magnitude of bearing pressure is dependent on the length and diameter of the trunnion pin. The contact pressure is assumed to act uniformly across the diameter of the trunnion pin.

3.8.2.2 *IS Considerations*

The slide type bushing shall be force fit in the trunnion hub and a running fit on the trunnion pin. (5.11.2)

If roller type bearing is provided, the outside diameter of the bearing shall be force fit in the trunnion hub and the inside diameter of the bearing shall be a force fit on the trunnion pin. (5.11.3)

Clause 5.11.4, thickness of bushing can be determined by the following formula:

(a) Minimum thickness of bushing in mm = 0.08 d + 3, where d is the pin diameter in mm.

3.8.2.3. *Trunnion pin*

The trunnion pin is designed as a simply supported beam with supports located at the centerlines of the yoke plates. Loading consists of the bearing stresses from the bearing or bushing. The length and diameter of the trunnion pin shall be proportioned such that the bearing pressure onto the bushing and flexural and shear stresses in the trunnion pin are less than the allowable stress limits as The bearing, flexural and shear stresses are calculated using commonly accepted engineering practices. The retainer plate and shear pins are designed to carry frictional loads produced when the radial gate is raised or lowered. The magnitude of torsion produced by friction is a function of the trunnion pin diameter, the coefficient of friction and the magnitude of the gate thrust. The weld connecting the retainer plate to the trunnion pin shall be sized to prevent rotation as shown in fig. 3.10. For spherical bearings, bearing movement occurs between the inner and outer rings only and not on the pin. Therefore, the pin is designed only to support the bearing inner ring.

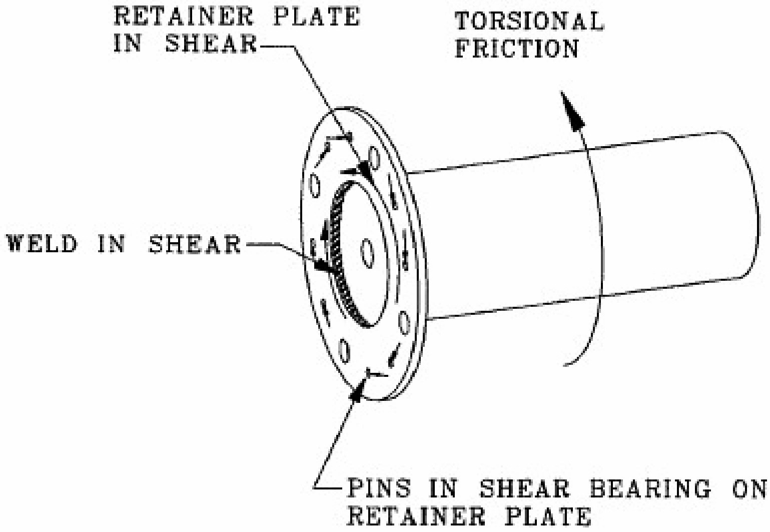


Figure 3.10 Trunnion pin and retainer plate

3.8.2.4 *IS Considerations*

The trunnion pin shall normally be supported at both ends on the trunnion bracket which is fixed to the anchorage or support girder. Where convenient the trunnion pin may be cantilever from the anchorage box, embedded in the piers or abutments. IS: 800-1984. (5.10.1)

The trunnion pin shall be designed against bending for the total load transferred through the trunnion hub. The load shall be taken to be uniformly distributed over the length of the pin bearing against the hub. (5.10.2)

The trunnion pin shall also be checked against shear and bearing due to the same load. (5.10.2.1)

The bending, bearing and shear stress in the trunnion pin shall not exceed 0.2 UTS, 1.0 UTS and 0.66 of bending stress respectively. (5.10.3)

3.8.2.5 *Yoke*

The yoke side plate shall be sized to resist trunnion pin bearing load and lateral gate loads. The base plate and stiffeners shall be designed to resist contact pressure between the yoke bearing plate and Anchor girder based on gate reaction forces and stressing loads imposed by steel stud bolts, as shown by Figure 3.10 To determine the required strength, it is recommended that the base plate be analyzed as a simple beam supported by the parallel yoke plates with a distributed load equal to the bearing pressure.

3.8.2.6 *IS Considerations*

The Anchor girder may or may not be embedded in concrete. It shall support the trunnion bracket and be held in place by the load carrying anchors. (5.14.1)

The maximum compressive stress in bearing at any point in the concrete in contact with anchor plate/girder shall not exceed 0.2 f_{ck} where f_{ck} is compressive strength, at 28 days, of the concrete used. (5.14.9)

Where the horizontal force from the trunnion pin is directly transferred to a yoke or Anchor girder immediately behind the trunnion pin the yoke or anchorage girder shall be designed against bending and shear caused by this force. (5.14.10)

The girder shall be treated as a simply supported beam loaded at the centre and supported at the junction of the girder and the load carrying anchors. (5.14.11)

3.8.2.7 *Trunnion hub*

The trunnion hub can be modeled as a cantilevered beam subjected to a distributed load from the trunnion pin. The cantilevered portion of hub extends beyond the flange of the trunnion arm extension.

The trunnion hubs shall rotate about the trunnion pins. The arms of the gate shall be rigidly connected to the hubs to ensure full transfer of loads. (5.9.1)

The hubs shall be sufficiently long so as to allow the arms of the gate to be fixed to the respective limbs of the hubs, without having to cut and shape the flanges of the arms. Where the arms are inclined, the limbs of the hub shall be on the apex of a cone with the base of the cone along the joints of the arms and the horizontal girders. (5.9.2)

The thickness of the webs and flanges of each of the limbs of the hub shall be greater to the extent so as to provide adequate space for the weld. (5.9.3)

To ensure rigidity of the trunnion hub, sufficient ribs and stiffeners shall be provided in between its webs and flanges. (5.9.4)

Thickness of hub may be calculated from the following relationship: (5.9.5)

$$t = 0.3\ d$$

Where,

t = hub thickness, and

d = diameter of the pin.

3.9 ANCHOR GIRDER

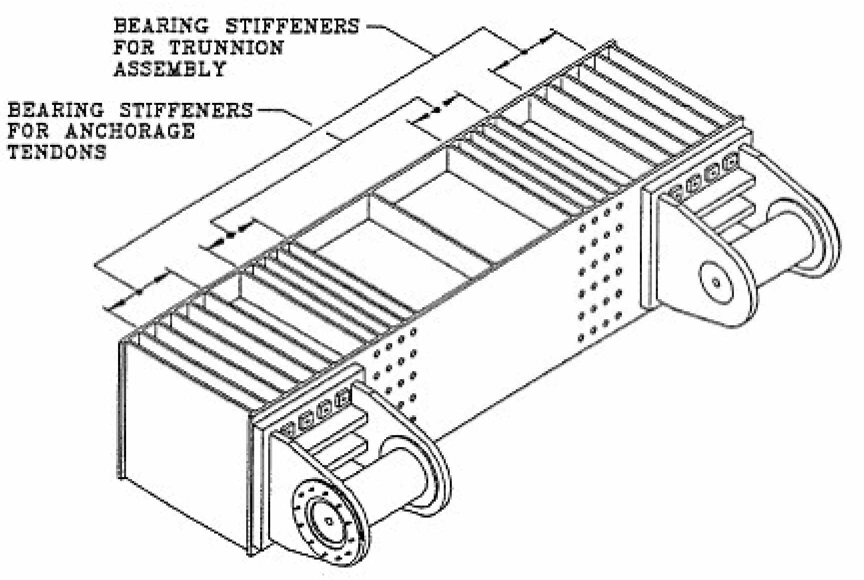


Figure 3.11 Anchor girder

3.9.1 Design considerations

The size of the Anchor girder is dependent on the magnitude of the flexural, shear, and torsion forces due to trunnion loads, and those forces due to anchorage jacking forces. The maximum shear and maximum bending forces act at the fixed end of the cantilever portion of the Anchor girder. These forces are combined with torsional and axial forces that result in a complex interaction of stresses. The design is accomplished by separating different stress contributions and designing for each individually. In general, shear and bending stresses due to transverse loading can be significant, while for most radial gate configurations, axial stresses are minimal. Maximum torsion will usually occur in the girder when the gate is partially raised and the pool is at maximum level.

3.9.2 IS considerations

The bracket shall be rigidly fixed to the anchorage or support girder by bolts. It shall transfer the total load from the trunnion to the anchorage. (5.12.1)

The arms of the bracket shall be designed to transfer the load from each trunnion in bearing. (5.12.2)

The arms of the bracket shall also be designed to resist any bending, which may come on them due to the component of the load parallel to the base of the trunnion bracket. (5.12.3)

Ribs and stiffeners shall be provided on the trunnion bracket, particularly on the sides of the bracket arms to ensure sufficient structural rigidity. (5.12.4)

Guide rollers shall be provided on the sides of the gate to limit the lateral motion or side sway of the gate to not more than 6 mm in either direction. (5.18.1)

Permissible stresses in design shall be same as given in Appendix A**.** (5.18.3)

3.10 GATE ANCHORAGE SYSTEM

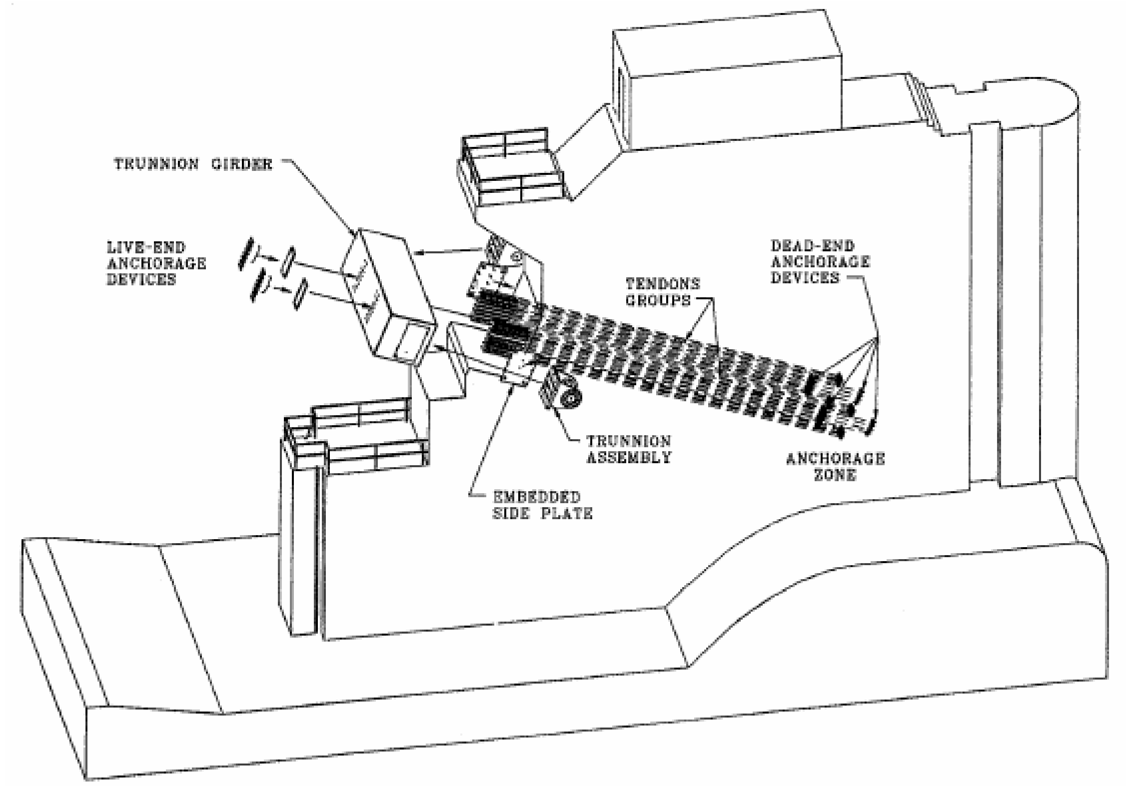


Figure 3.12 General arrangement of gate anchorage

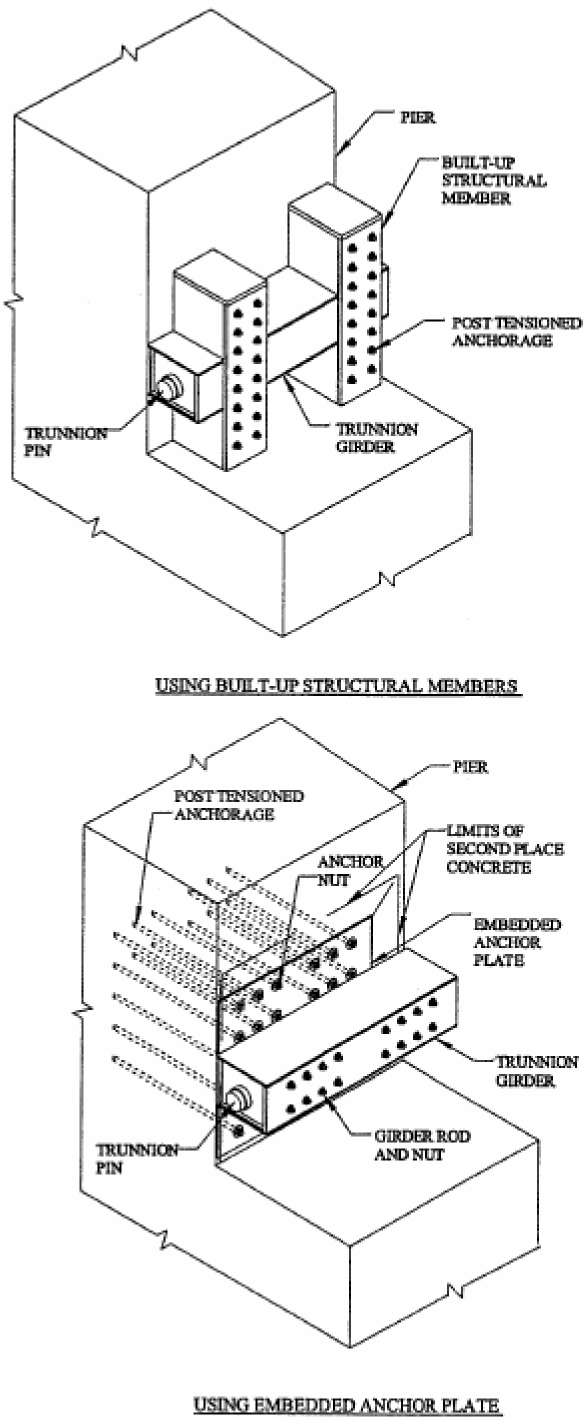


Figure 3.13 Alternate anchorage systems

Anchorage is designed to withstand the total water load on the gate.

These shall be designed to withstand the total water load on the gate and transfer it to the piers and the abutments or to the civil structure within the tunnel or the conduit. (5.13.1)

The load may be transferred to the civil structure either in bond as a bond stresses between the anchors and the concrete or in bearing as a bearing stress between the concrete and the embedded girder at the upstream and of anchors, which in this case are insulated from concrete. (5.13.2)

This is beyond to the scope of present study. Therefore same is not concerned in detail.

4. LOADS ACTING ON RADIAL GATE AND ITS PIERS

4.1 LOADS ACTING ON RADIAL GATE

It is prime important to understand the type of load acting and their combination, so that design could be made reliable. If loads are underestimated it may result into unsafe structure, on the other side, if loads are overestimated, it may result into uneconomic structure. In case radial gates, the loads that act are dead load, hydrostatic load, wind load, earthquake load. Hydrostatic loads are considered into two categories,

(I) Water load at FRL, and

(II) Water load at HFL.

The details of these loads are discussed under appropriate head.

4.1.1 Gravity load

The dead load of gate is obtained by multiplying the unit weight of steel 7850 kg/m^3 to the volume of section for fabricated section and directly from steel table for rolled section. For gate closed condition, the total dead load is supported, on one end by spillway crest and at the other end by trunnion. In case of gate open condition, the gate weight is supported at trunnion and at hoist, through hoisting cable as shown in fig.4.9.

4.1.2 Hydrostatic load

Hydrostatic load consists of hydrostatic pressure on the gate considering both FRL (full Reservoir level) and HFL (high flood level) conditions. As shown in fig.4.1, the distribution of hydrostatic pressure due to FRL is considered as triangular distribution with zero intensity at top of gate level and maximum intensity at the gate bottom level. Similarly, in case hydrostatic pressure due to HFL, the distribution is considered as trapezoidal as water level rises above top of gate. The design of gate is carried out for FRL condition, then the same is checked for HFL condition with increase in permissible stresses as per IS 4623 1984 page 25 clause 6.5.

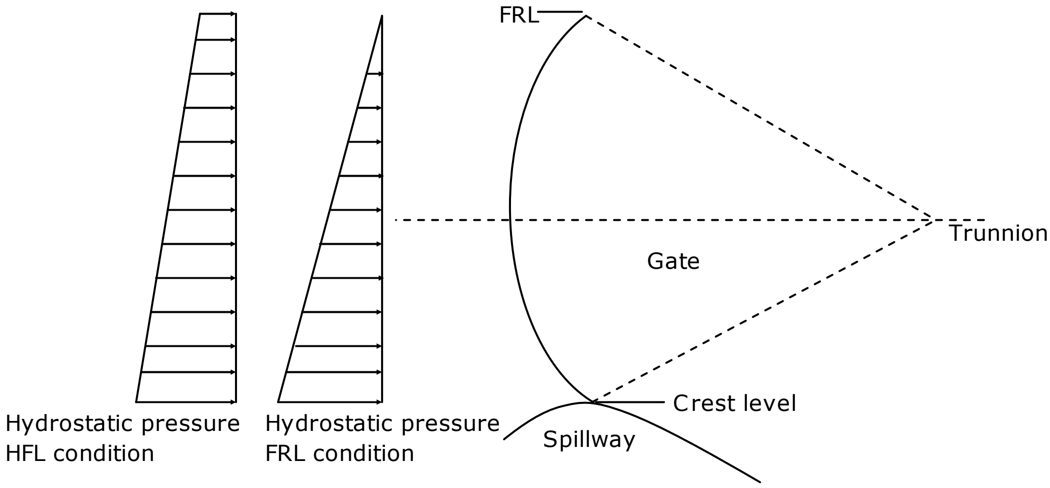


Figure 4.1 Hydrostatic pressures on radial gate in FRL and HFL conditions

4.1.3 Occasional forces

4.1.3.1 *Seismic load*

Seismic loads are the type of occasional loads that has to be taken into account quite precisely. Seismic produces dual effect on the structure. One, it causes additional horizontal and vertical forces in the gate, piers etc. Secondly, it produces hydrodynamic forces, in addition to hydrostatic forces as per IS 1893 1984 page 39 clause 7.2.1, as shown in fig 4.2.

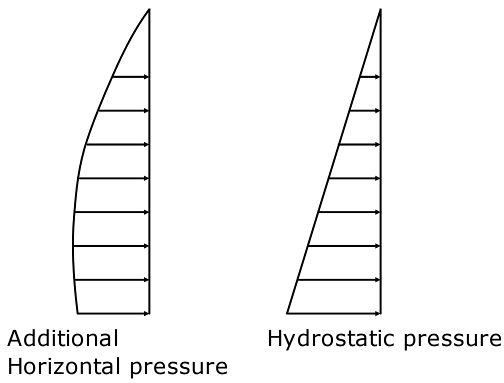


Figure 4.2 Hydrostatic and hydrodynamic forces

Due to horizontal acceleration of the foundation and dam there is an instantaneous hydrodynamic pressure (or suction) exerted against the dam in addition to hydrostatic forces the direction of hydrodynamic force is opposite to the direction of Seismic acceleration. Based on the assumption that water is incompressible, the hydrodynamic pressure at depth y below the reservoir surface shall be determined as follows:

$$p = c_s \times \alpha_h \times w \times h \tag{4.1}$$

Where,

p = hydrodynamic pressure in kg/m^2 at depth y, as shown in fig. 4.2,

c_s = coefficient which varies with shape and depth, and

α_h = design horizontal seismic coefficient.

$$\alpha_h = \beta \times I \times \alpha_0 \qquad (4.2)$$

Where,

β = a coefficient depending upon the soil foundation system,

I = a factor dependant upon the importance of the structure,

α_0 = basic horizontal seismic coefficient,

w =unit weight of water in kg/m^3 and

h =depth of reservoir in m.

The approximate values of total horizontal shear and moment about the centre of gravity of a section due hydrodynamic pressure are given by the relations,

$$V_h = 0.726 \times p \times y \quad \text{and} \qquad (4.3)$$

$$M_h = 0.299 \times p \times y^2 \qquad (4.4)$$

Where,

V_h= hydrodynamic shear in kg/m at any depth, and

M_h= moment in kg.m/m due to hydrodynamic force at any depth y.

Thus, skin plate is checked for carrying hydrostatic and hydrodynamic pressure together. Then equation (4.3) and (4.4) are used for calculation of V_h and M_h. These values are applied to the C.G of gate, and then stresses developed in horizontal girder, radial arms, trunnion girder etc. are checked to ensure whether stresses remain within the limits of the permissible stresses. As mentioned earlier the permissible stresses due to seismic forces are taken 33% greater than permissible stresses.

4.1.3.2 *Wind load*

Wind loads are site specific and small compare to hydrostatic loads. The wind load is calculated based on provision of IS 875 (part 2) 1987 clause 5.4

$$P_z = 0.6\ V^2_z \qquad (4.5)$$

Where,

$$V_z = V_b \text{ x } k_1 \text{ x } k_1 \text{ x } k_3 \qquad (4.6)$$

Where,

V_z = design wind speed at any depth,

k_1 = probability factor,

k_2 = terrain height and structure size factor,

k_3 = topography factor.

Consider gate size of 9.15m x 6.3m in which Seismic pressure is 73 kN/m^2.

For same size of gate the wind pressure through following steps.

From IS 875 (part 3) 1987 table 1, basic wind speed = 50 m/s.

For 100 year mean probable design life of structure, k_1 = 1.08.

From IS 875 (part 3) 1987 table 2, for the terrain category 1, for class of structure B, for height 50m.

$k_2 = 1.18$

From IS 875 (part 3) 1987 clause C-2, $k_3 = 1.36$

Thus, Vz = 1.08 x 1.18 x 1.36 x 50.

$P_z = 0.6\, V_z^2$

$P_z = 0.6 \times 86.65^2$

$P_z = 4504.93\ N/m^2$.

And due to seismic forces 73000 N/m^2.

So, 73000 N/m^2 > 4504.93 N/m^2

Sample calculation for wind is presented over here to find the pressure acting on gate. It was found that the pressure generated is quite less as compared to that of Seismic force. Thus, the load combination of dead load and wind load is not carried out in detail.

4.1.3.3 *Wave effect*

When the reservoir is full of water, there is formation of waves due to wind as shown in fig.4.3. The height of wave depends upon the wind velocity and the fetch distance. In the problem solved, fetch distance considered as 3.2 km.

Wave height is given by formula

$$h_w = 0.032 \times \sqrt{VF} + 0.763 - 0.271 \times F^{0.25} \tag{4.7}$$

Where,

h_w = height of wave from trough to crest in m,

V = wind velocity in km/h and

F = 'Fetch' or straight length of water subject to wind action in km.

Above equation 4.7 gives the height of wave that is added to FRL to get the value of HFL.

As per IRC 6-2000 table 4, wind velocity for height of 60m, is 168 km/h.

Wind velocity=168 km/hr.

Therefore, $h_w = 0.032 \times \sqrt{168 \times 3.2} + 0.763 - 0.271 \times 3.2^{0.25}$ (From eq… 4.7)

$= 1.142$m.

Additional wave height above FRL

$$= \frac{2}{3} \times hw = \frac{2}{3} \times 1.142 = 0.7613\text{m}.$$

As gate is checked for an additional water height of 1m above FRL, therefore, wave effect is not considered. As stated earlier, in wave effect also stresses are increased by 33% and gates are checked for these stresses.

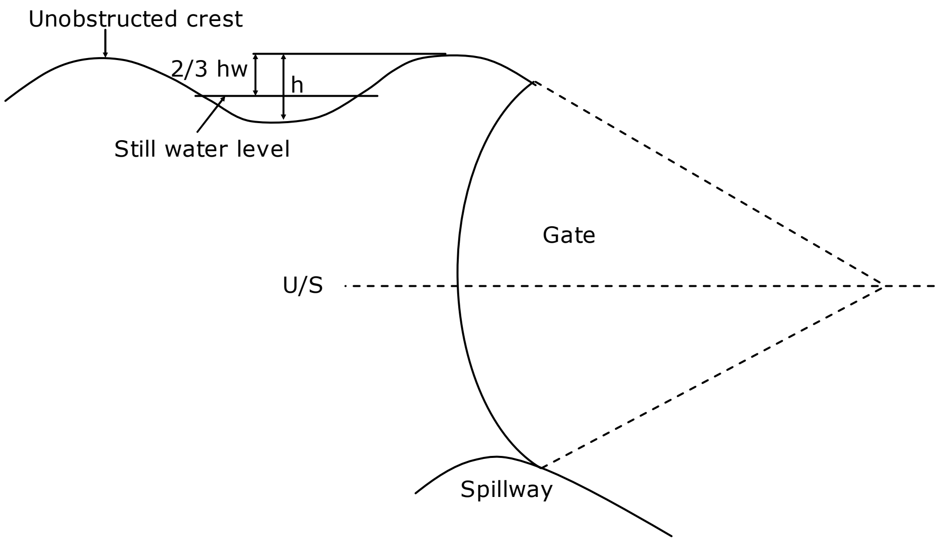


Figure 4.3 Wave effect on radial gate

4.1.4 Side Seal friction load

The total hoist capacity is decided by total load to be lifted and the frictional load developed between the side seals and the side seal plate while opening or closing the gate. The product is equal to the product of the coefficient of friction and normal force between the seal plates and the side seal.

4.2 LOADS ACTING ON PIER

Piers are the structures that withstand all the horizontal and vertical load; be it dead load of gate, hydrostatic pressure, spillway bridge etc. The RCC wall type pier is normally provided to hold the gate in position. The gate is connected to pier at trunnion via anchor rods and bars. Pier has to be designed for conditions like adjacent gates closed, adjacent gate gates open, and one gate open and adjacent gate closed. The details of various loads acting on pier as per provision of IS 13551-1992 is discussed below.

4.2.1 Longitudinal hydrostatic pressure due to steady water (P2)

This is considered as triangular load acting on projected face of pier, with zero intensity at FRL and maximum intensity at the spillway crest level.

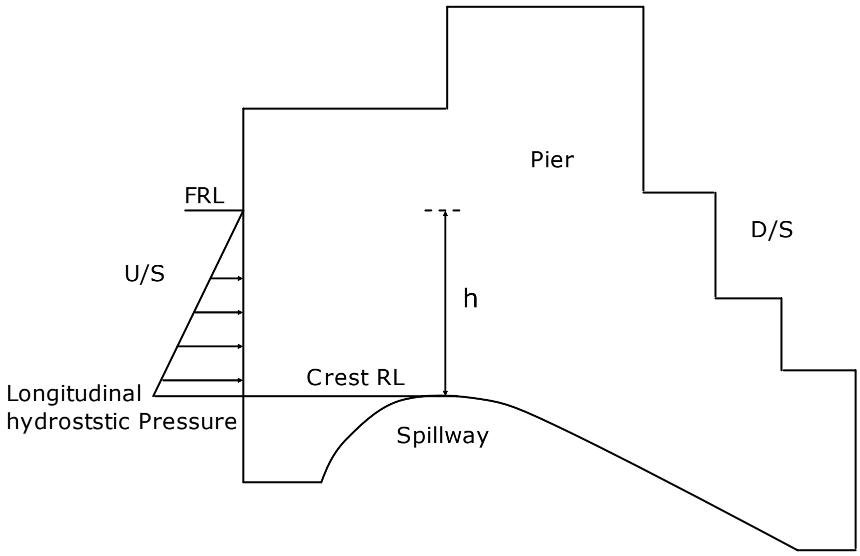


Figure 4.4 Longitudinal hydrostatic pressure on pier

Longitudinal hydrostatic pressure acting in longitudinal direction is shown in fig.4.4, and is calculated as follows:

$$P_2 = \frac{W \times h^2 \times \text{Pier thickness}}{2 \times 2} \tag{4.8}$$

Where,

P_2= longitudinal hydrostatic pressure due to steady water,

w = unit Weight of water, and

h = height of water (FRL – CREST RL)

4.2.2 Longitudinal water pressure due to flowing water (P3)

The hydrostatic water pressure due to flowing of water is calculated as per provision of IRC 6-2000 clause 2.13 where pressure is given by

$$P = 52 \times KV^2 \tag{4.9}$$

Where,

K = 0.66 (co efficient depend upon shape of pier),

V = velocity of current.

$$V = \sqrt{2 \times g \times h} \tag{4.10}$$

g = gravitational acceleration = 9.81 m/s^2,

h = height of water (FRL – CREST RL),

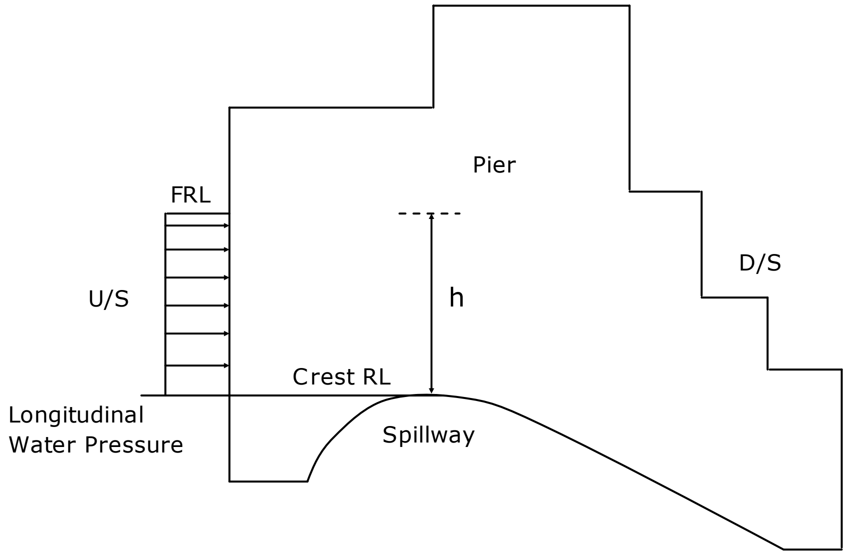
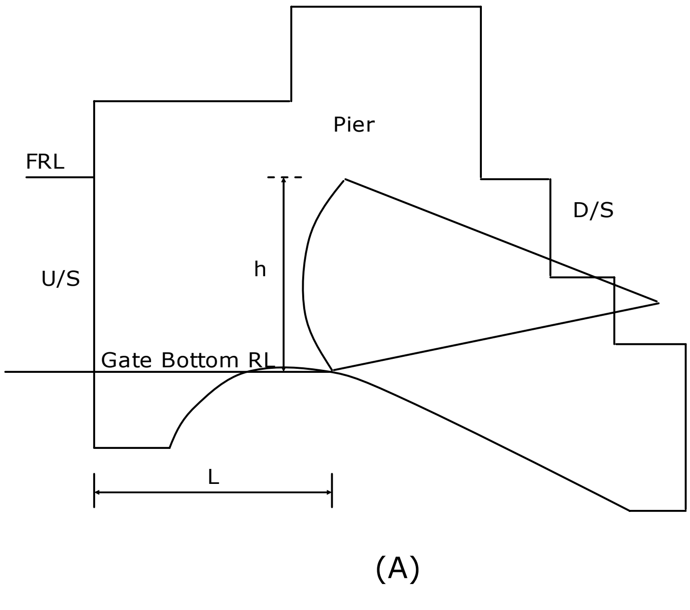


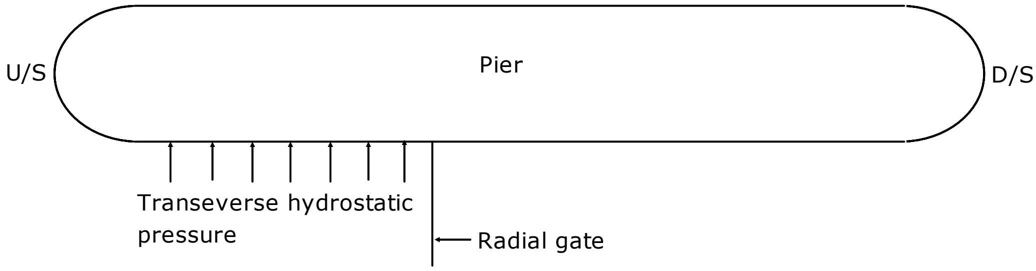
Figure 4.5 Longitudinal flowing water pressure

4.2.3 Transverse Water pressure due to gate closed (P4)

Water also causes transverse loading on the pier. This acts in transverse direction when gate is closed position as shown in fig.4.6



(A)



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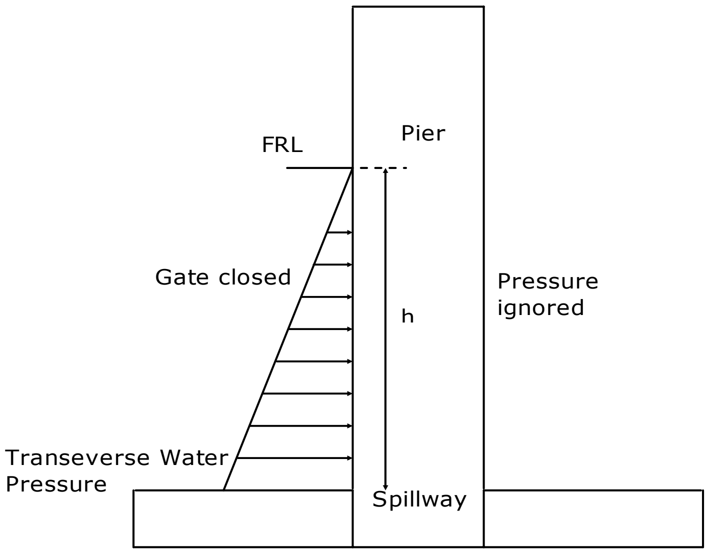


(C)

Figure 4.6 Transverse water pressures due to closed gate

$$P_4 = \frac{w \times h^2}{2} \tag{4.11}$$

Where,

P_4 = transverse water pressure due to gate closed

W=unit Weight of water

h = height of water (FRL –Gate Bottom RL)

4.2.4 Transverse water pressure due to gate open (P5)

This pressure acts when water is flowing while gate is in open condition. The pressure is assumed to act on the entire length of pier, as shown in fig.4.7.

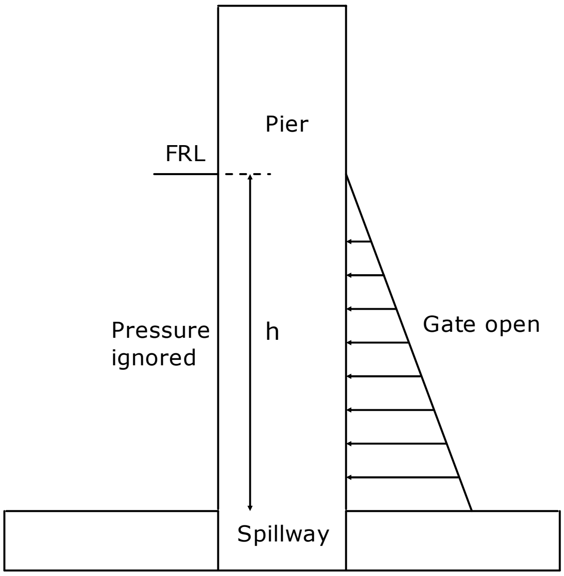


Figure 4.7 Transverse water pressure

4.2.5 Hydrodynamic force due to Earthquake (P6)

Hydrodynamic forces act on pier and is calculated as per the provision of IS 1893 1984.

$$p = c_s \times \alpha_h \times w \times h \tag{4.12}$$

Where,

p =hydrodynamic pressure in kg/m^2 at depth y.

c_s =coefficient which varies with shape and depth.

α_h = design horizontal seismic coefficient.

$$\alpha_h = \beta \times I \times \alpha_0 \tag{4.13}$$

Where,

β = a coefficient depending upon the soil foundation system.

I = a factor dependant upon the importance of the structure

α_0 = Basic horizontal seismic coefficient.

w =unit weight of water in kg/m^3.

h =Depth of reservoir in m.

Above equation 4.13 is used to find the additional horizontal shear (V_h) due to hydrodynamic pressure at any depth y and moment (M_h) due to hydrodynamic force at any depth y about the centre of gravity of a section due hydrodynamic pressure as per provision of IS 1893 1984.

$$V_h = 0.726 \times p \times y \tag{4.14}$$

$$M_h = 0.299 \times p \times y^2 \tag{4.15}$$

As per the provision of IS 1893 1984 clause 7.3.2, the distribution of pressure is assumed to be linear with an increase intensity of 1.5 p, as shown in fig.4.8

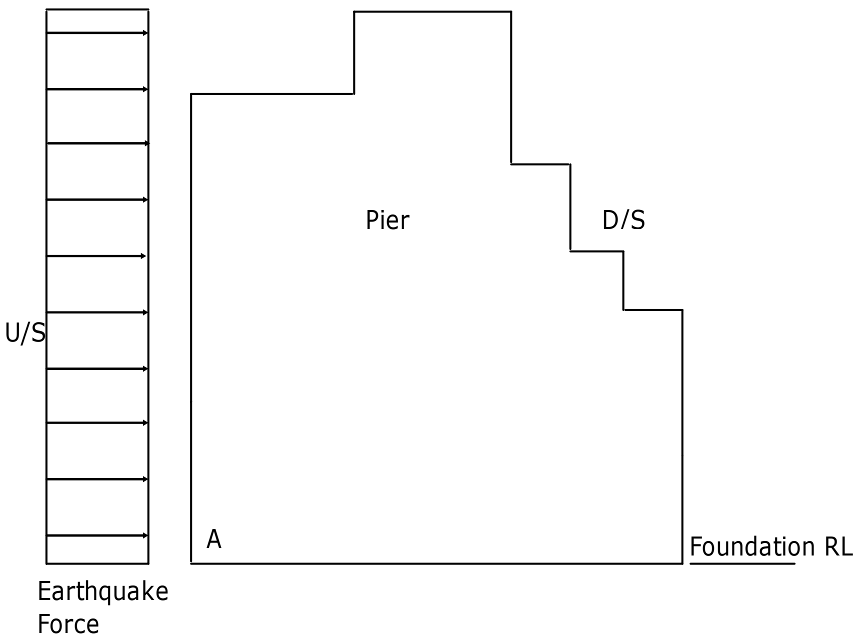


Figure 4.8 Earthquake force on pier

4.2.6 Reactions due to opening of gate (P7)

When gate is opened, the weight of gate is transferred to the pier partly at the trunnion level and partly at the hoist level via tension in hoisting ropes as shown in fig. 4.9

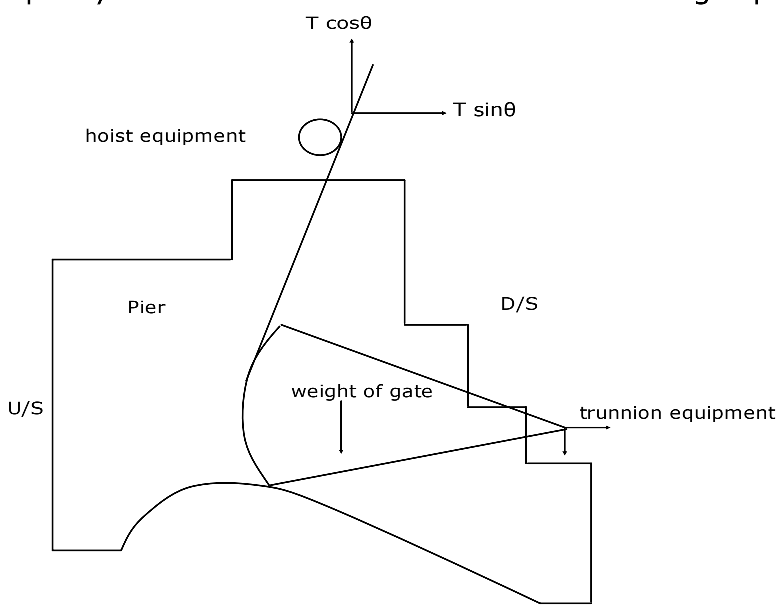


Figure 4.9 Loads due to opening of gate

4.2.7 Dead load and live load of bridge (P8,P9,P10,P11)

To allow the movement across the spillway a RCC bridge is provided and the same is supported on the piers. The dead load of the bridge and live load on the bridge is calculated as per IRC. Breaking force and frictional force because of live load is also calculated as per relevant clauses of IRC. As this is beyond the scope of study, a live load of class 70R is considered on the bridge for designing the piers.

4.2.8 Uplift pressure (P16)

As the spillway stores water on one side, uplift pressure is assumed to act at the bottom of pier. The distribution of uplift pressure acting is equal to Wh at upstream side and varies linearly up to the face of gate, and then it decreases linearly to zero at the downstream face of the pier as shown in fig 4.10.

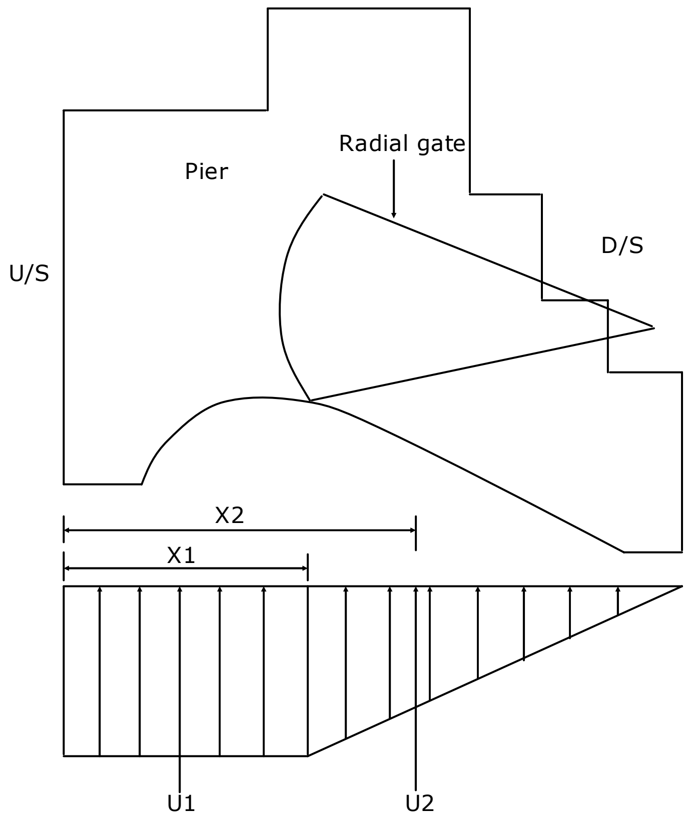
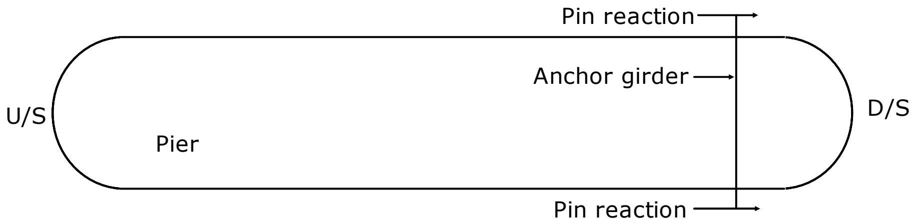


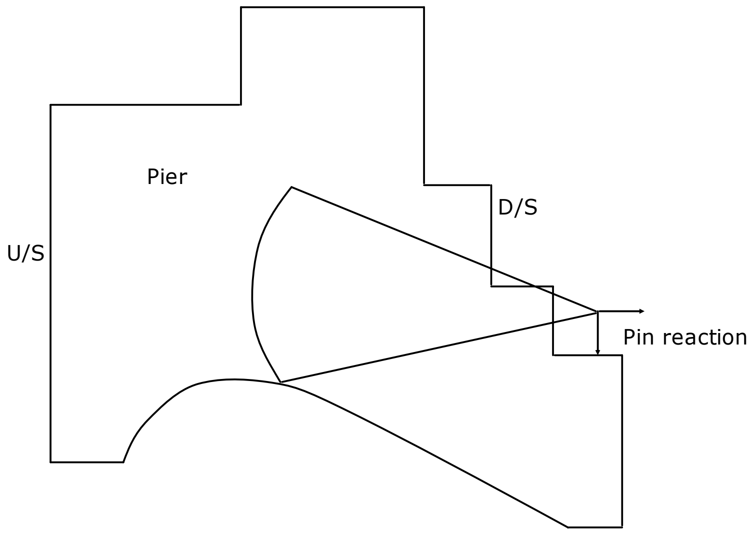
Figure 4.10 Uplift pressure

4.2.9 Pin reaction (P12)

As per provision of IS 13551-1992, it is a reaction at trunnion pin due to hydrostatic pressure as shown in fig. 4.11.



(A)



(B)

Figure 4.11 Pin reaction

5. ANALYSIS AND DESIGN OF RADIAL GATE

Radial gate resist mainly hydrostatic load. For analysis of radial gate hydrostatic loads in FRL and HFL conditions is considered.

5.1 SKIN PLATE

First of all for the design of the skin plate the curvature of the skin plate is ignored. Curvature length is considered as full straight length of radial gate as shown in fig.5.1. The width of radial gate is given from hydrological data.

Skin plate is designed as bending element resting on series of verticals. Hydrostatic pressure is calculated and accordingly stresses due to bending are calculated. If these stresses exceed the permissible stresses given in appendix A, either the spacing of verticals is increased and/or the thickness of skin plate is increased. In order to ensure proper care for corrosion, the thickness found is increased by 1.5mm as per provision of IS 4623.

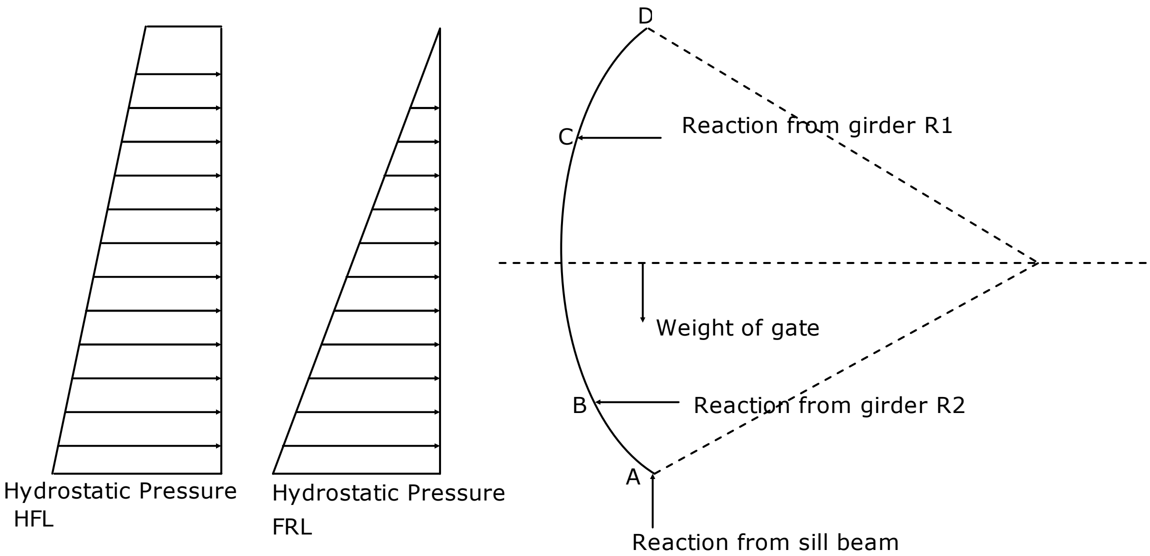


Figure 5.1 Loads acting on radial gate

5.2 TEE VERTICALS

Tee verticals are vertical members that run parallel to the height of the radial gate, and support the skin plate, that in turn rest on horizontal girder as shown in fig.5.2. As Tee sections are welded through out the length, the skin plate behaves as the flange of the tee verticals. The co acting width is found as follow:

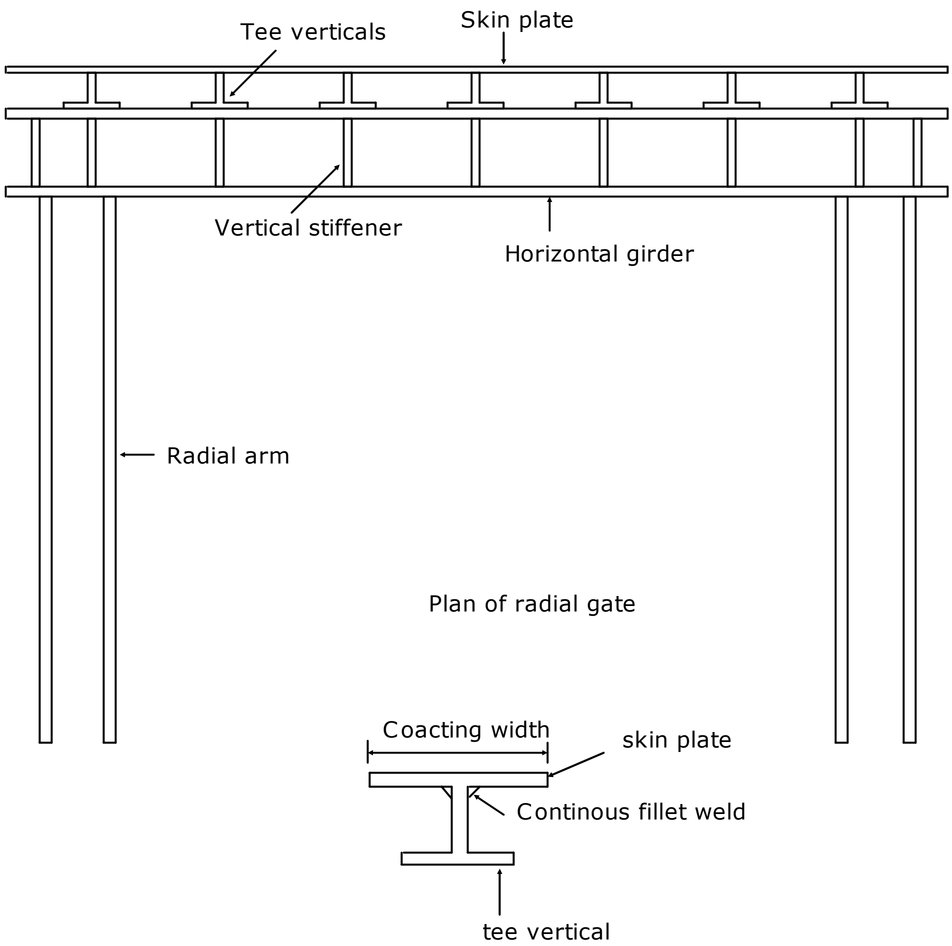


Figure 5.2 Plan of radial gate and skin plate assembly

IS 4623-1984 page 13 clause 5.6.5

It is least of the following values

1) 40 t + B

Where,

T = thickness of skin plate, and

B= width of the stiffener flange in contact with the skin plate.

2) 0.11 span

3) Centre to centre of stiffener or girders.

The stresses developed at A, B, C, D (refer fig.5.1) are the calculated, and are checked with permissible stresses. Permissible stresses are mentioned in appendix A.

5.3 HORIZONTAL GIRDER

Tee verticals transfer their loads to horizontal girder that, in turn, transfers the load to radial arms. In this project, two horizontal girders are considered. These horizontal girders are designed as simply supported beams. Girder section is designed for maximum bending moment at the centre. These girders are then stiffened with intermediate stiffener as per IS 800 1984 clause 6.7.4.2, within the span and bearing stiffeners at the ends of horizontal girder as per IS 800 1984 clause 6.7.5.3. All stresses are mentioned in Appendix A

5.4 RADIAL ARMS

Radial arms support the horizontal girder and are supported on trunnion. The radial arms are designed as compression members, suitably braced by bracing to take care of the slenderness ratio as shown in fig.5.3. The radial arms are generally made up of I-section, or two channels placed back to back. The bracings are designed for 2.5% of force in radial arms. It means 0.025x (nR1/2 + nR2/2), as per IS 800 1984 clause 5.7.2.1, all permissible stresses are mentioned in Appendix A.

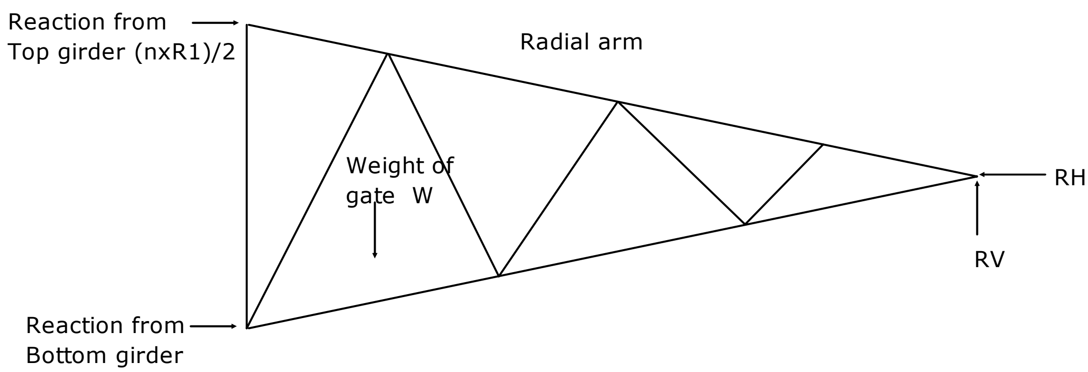


Figure 5.3 Free body diagram of radial arm

5.5 DESIGN OF TRUNNION PIN

Trunnion pin as shown in fig.5.4 is subjected to horizontal as well as vertical forces due to hydrostatic pressure and weight of various components. These values are found for various gate openings, for an interval. The resultant of all these vertical and horizontal forces is found out. This resultant is divided into two halves then trunnion is designed for this value, as shown in fig. 5.4, thereafter it is checked for shear and bending stresses. All permissible stresses are mentioned in appendix A. in fig. 5.4 RF is maximum resultant force and Dtp = outer diameter of trunnion pin, dtp = inner diameter of trunnion pin. Also section of trunnion pin is shown in Fig.5.4.

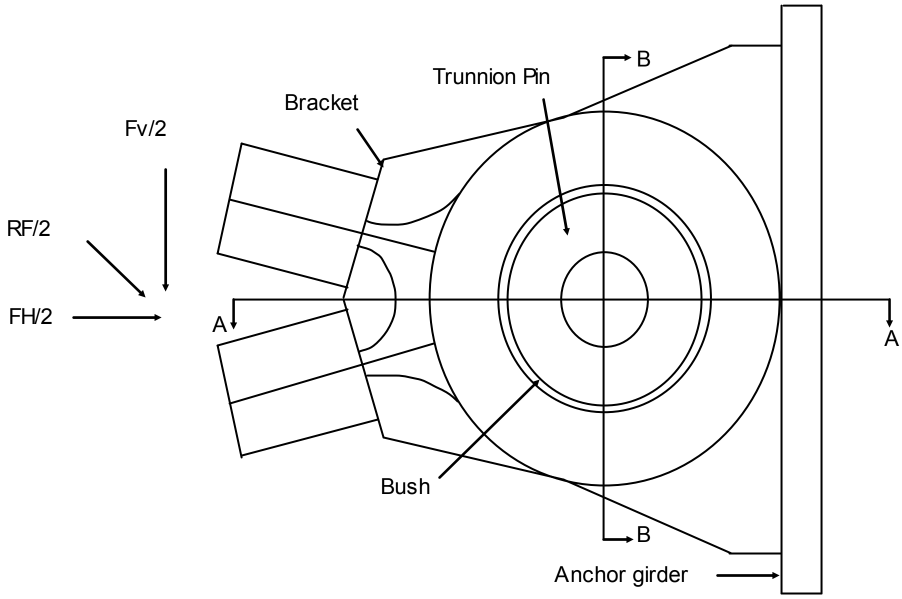


Figure 5.4 Half section elevation of bracket, trunnion pin and bush

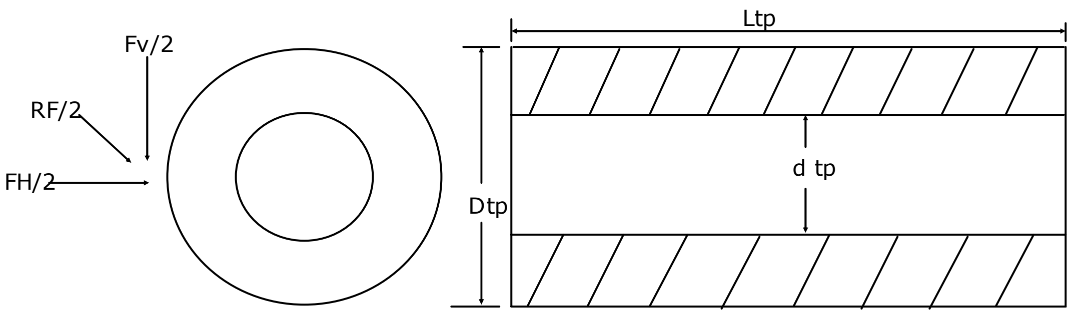


Figure 5.5 Section B-B of trunnion pin

5.6 BUSH DESIGN

Bush is generally bronze type material and tied with trunnion pin as shown in fig.5.5. The outer diameter of the trunnion pin is equal to the internal diameter of bush. The length of the bush is same as that of trunnion pin. The bush is designed for half of the total resultant and designed for safe bearing stress. Stress as per provision of IS 305 1981. As shown in fig.5.6, dbi=internal diameter of bush.

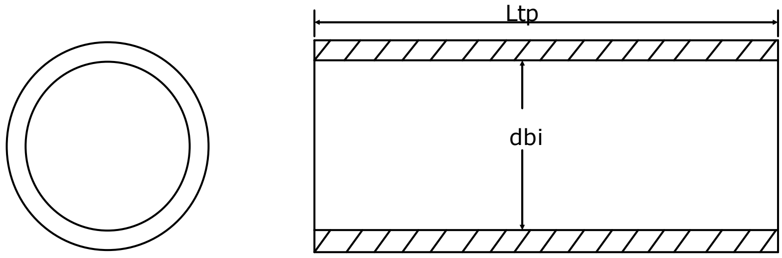


Figure 5.6 Section B-B of bush

5.7 DESIGN OF BOLTS AND BRACKET TO CONNECT ANCHOR GIRDER

The trunnion assembly is connected to the anchor girder through bolts. These are designed for vertical load of $\frac{1}{2}F_V\cos\theta$ and horizontal load of $\frac{1}{2}F_H\sin\theta$ where, θ = angle of tilt. These bolts are designed for combined stresses due to tension and shear. The bracket is checked in bearing stress only. As shown in fig.5.7 Ftbv = $\frac{1}{2}F_v\cos\theta$ and Ftbh = $\frac{1}{2}F_H\sin\theta$.The permissible stresses are mentioned in appendix A.

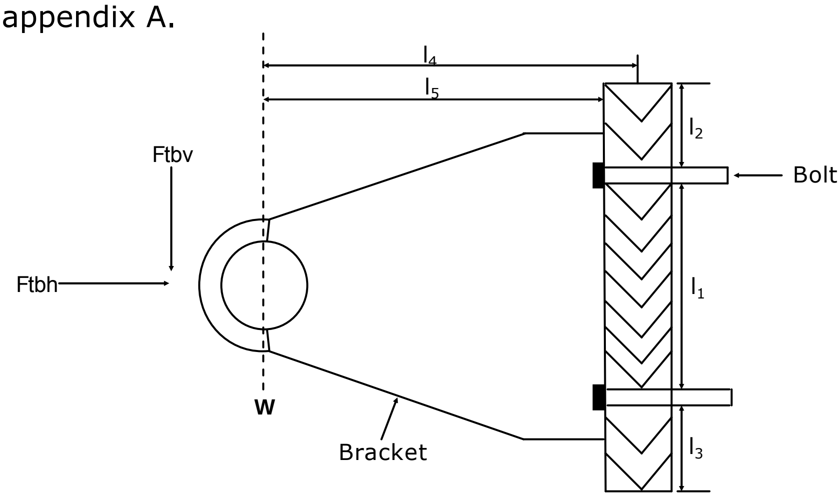


Figure 5.7 Elevation of bracket assembly

5.8 DESIGN OF ANCHOR GIRDER

Anchor girder is a built up section. It supports the trunnion assembly at its ends. Between these ends, it serves as anchorage for the horizontal anchors that runs in the body of the pier. Entire loads of the radial gate and water is transferred into pier through the horizontal anchors. Anchor girder may be of I section as shown in fig.5.8 or box type as shown in fig.5.10

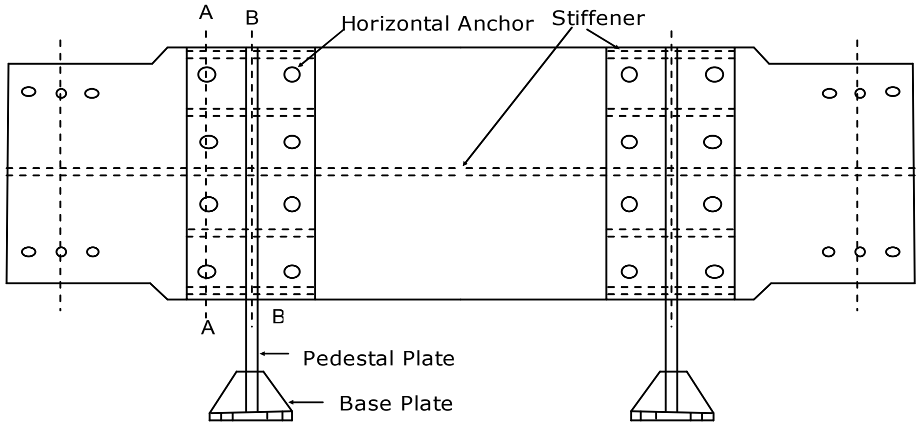


Figure 5.8 Elevation from anchor girder from pier side (type 1)

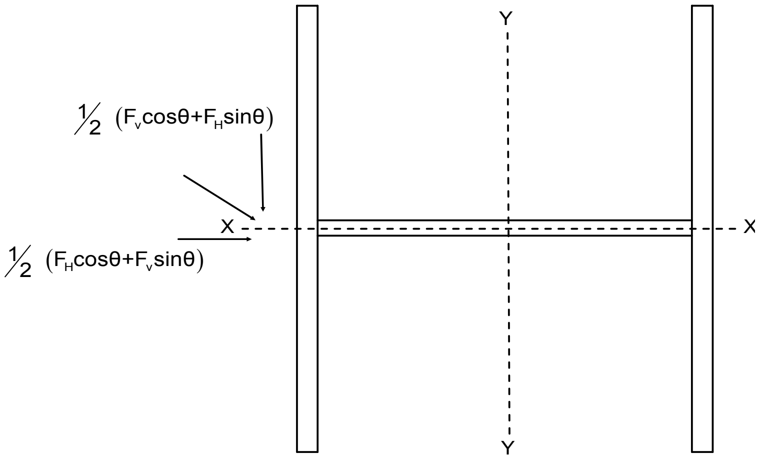


Figure 5.9 Section of anchor girder (type 1)

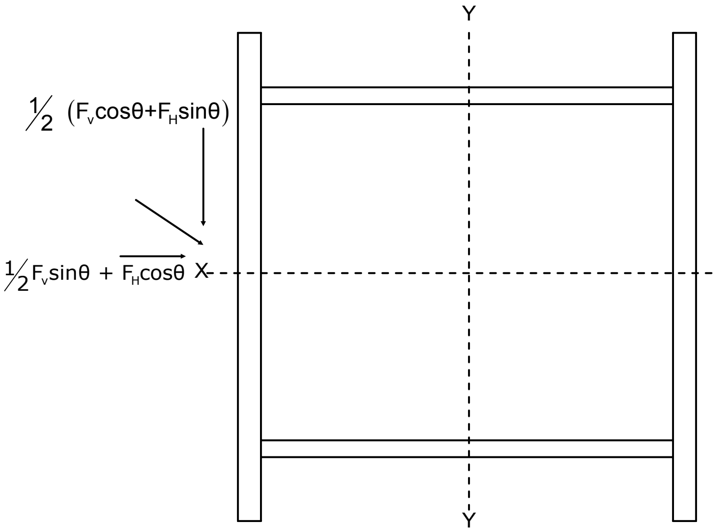


Figure 5.10 Section of anchor girder (type 2)

On anchor girder horizontal load of $\frac{1}{2}\left(F_H\cos\theta+F_v\sin\theta\right)$ and vertical force of $\frac{1}{2}\left(F_v\cos\theta+F_H\sin\theta\right)$ Is applied, where, $\theta=$ angle of tilt and this section is checked at section A-A and at section B-B. These sections are checked for combined effect of torsion and bending moment due to torsion and bending moment. Finally design intermediate stiffeners are designed. All stresses are mentioned in appendix A.

5.9 DESIGN OF HORIZONTAL ANCHORS

Horizontal anchors are designed for loading condition having one gate open and adjacent gate closed. As shown in fig.5.11, moment @ compression side is taken and total tension is found out. The anchor rods are then designed for these tension values. Anchor length of horizontal girder is based on bond stress of concrete.

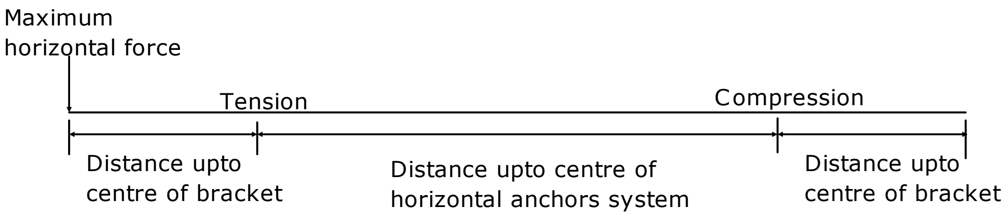


Figure 5.11 Analysis of horizontal anchors

5.10 DESIGN OF VERTICAL ANCHORS

Same method is adopted for design of vertical anchors as in horizontal anchors with one gate open and adjacent gate closed. The value is solved in the manner, similar to horizontal anchor, as shown in fig.5.13. All stresses are mentioned in appendix A.

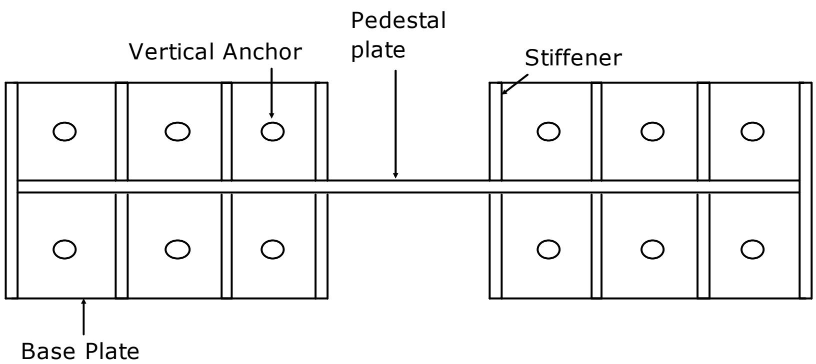


Figure 5.12 Plan of base plate with vertical anchors and pedestal plate

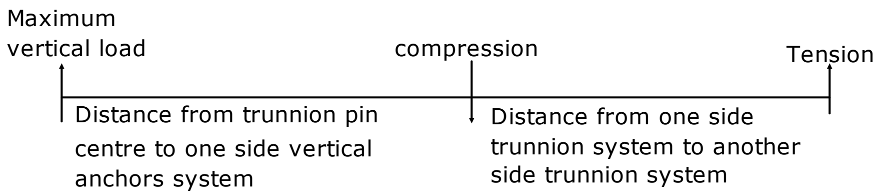


Figure 5.13 Analysis of vertical anchors

This is followed by calculation of anchorage length based on bond stress of concrete.

5.11 DESIGN OF PEDESTAL PLATE AND ITS BOLTS

Pedestal plate is designed for tension as well as compression obtained in vertical anchors system. Its bolts are designed for maximum vertical force obtained same from the vertical anchors.

5.12 DSIGN OF BASE PLATE

The dimension of the base plate is initially found out from the dimension of the anchor girder. The thickness of the plate is checked for the bending moment caused by the upward pressure found by load/area of plate using M=fz, the thickness of the plate is found.

5.13 PROGRAM INTRODUCTION

Radial gate is designed as a steel structure subjected to hydrostatic pressure, earthquake etc. as discussed in chapter 4.

For analysis and design of various components of gates, program in C++ and MS excel work book are prepared. The listings of programs prepared are as follows.

C++ program	MS excel work sheet
1. Gate frl design. cpp	1.Trunnion. xls
2. Gate hfl design. cpp	2. Pier.xls
3. Gate eq design. cpp	3. Pier footing. xls
4. Gate open frl .cpp	4. Connection detail.xls
5. Gate open hfl .cpp	

The data required and output obtained from above programs is explained below.

1. Program : Gate frl design .cpp
   Input : All RLs, assumed section sizes of various members, permissible stresses.
   Function : This program is calculate dead load of structure, hydrostatic forces on gate for FRL condition, and checks the feasibility of assumed sections. If the stresses are not with permissible limits, section is revised, and again program is run.
   Output : Complete design of all components of gate for FRL condition.
2. Program : Gate hfl design .cpp
   Input : Similar to above program, percentage increase in permissible stresses of steel and bolts or welding. The sections from above program acts as input for this program.
   Function : similar to above program, but for HFL condition.
   Output : complete design of all components of gate for HFL condition.
3. Program : Gate eq design. cpp
   Input : EQzone, various parameter related to same, percentage increase in permissible stresses
   Function : The programs calculates Earthquake forces on the structure, and also calculate hydrodynamic pressure. The sections are then checked for these loads.
   Output : Final sections are obtained in this program.
4. Program : Gate open frl .cpp
   Input : above data, interval of gate opening at which forces are required to be calculated.
   Output : Horizontal and vertical forces at trunnion level are obtained, at desired intervals.
5. Program : Gate open hfl.cpp
   Function : Same as above program, but for HFL condition.

1. Trunnion.xls

This MS excel work sheet gives the design of trunnion assembly, anchor girder, horizontal anchors, vertical anchors pedestal plate, base plate.

2. Pier.xls

   This MS excel work sheet the designs of pier as column subjected to biaxial bending.

3. Pier footing. Xls

   This MS excel work sheet gives the design of footing for pier, as open foundation.

The overall flowchart of various program in sequence of their use is shown below.

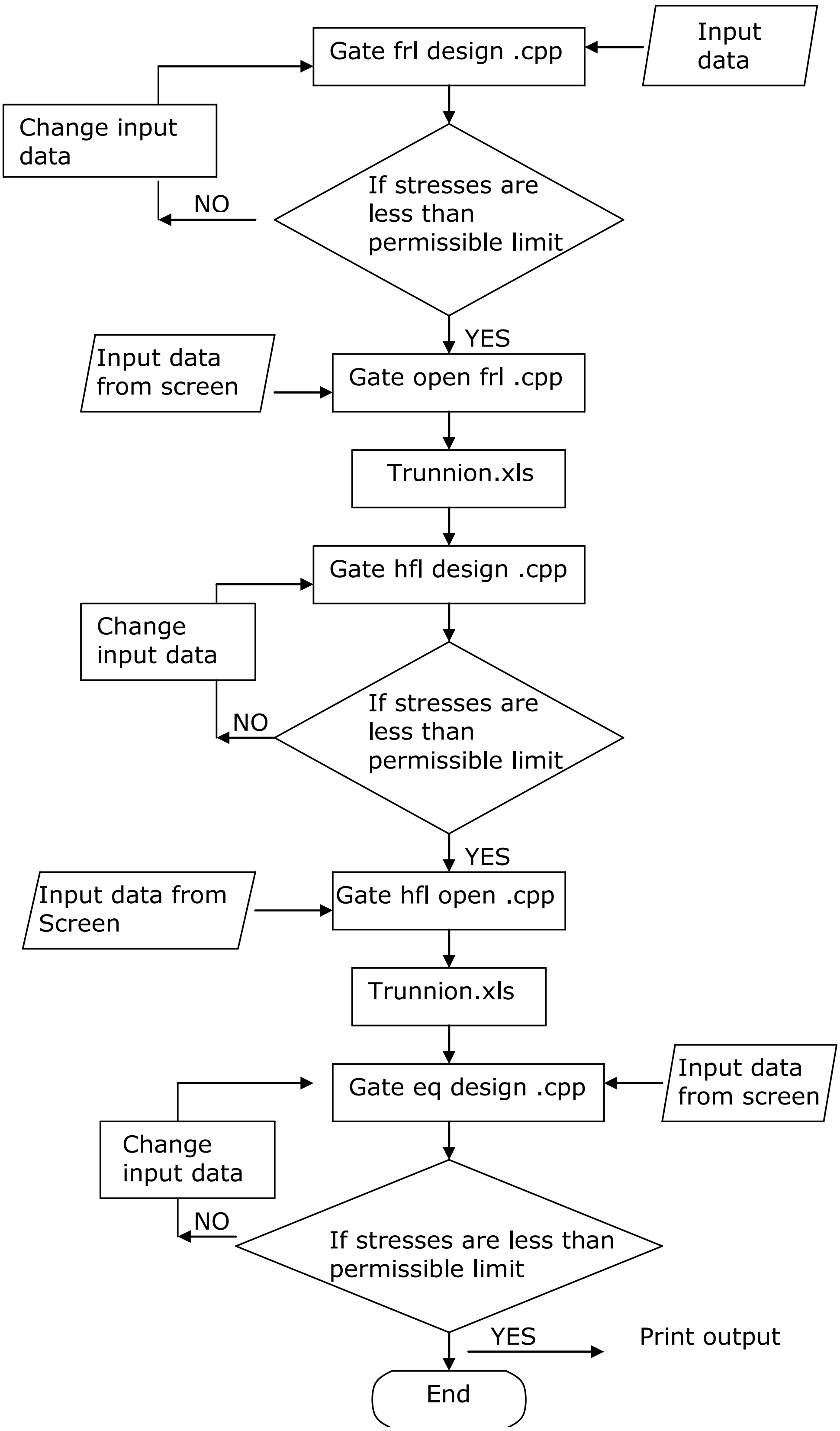
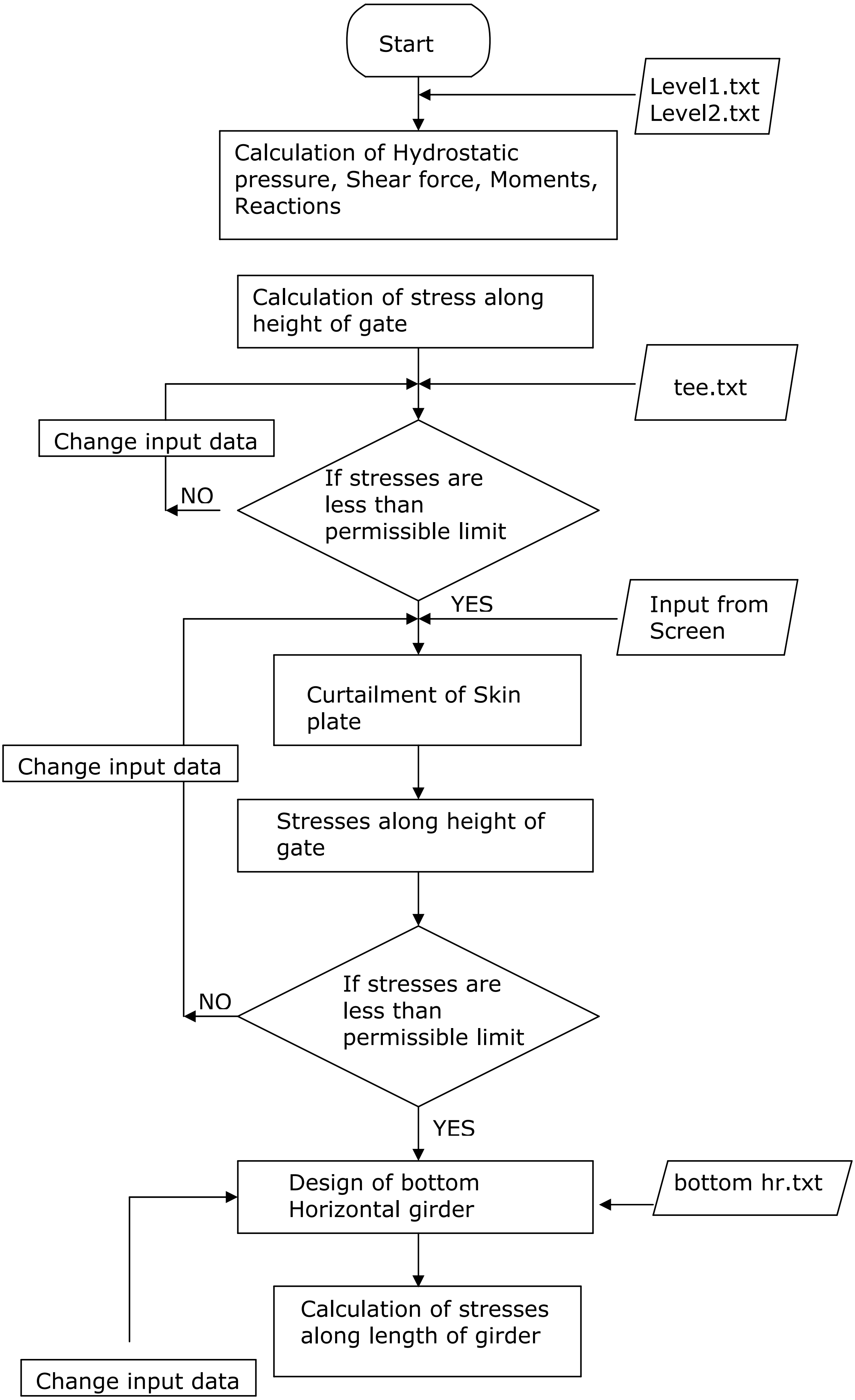
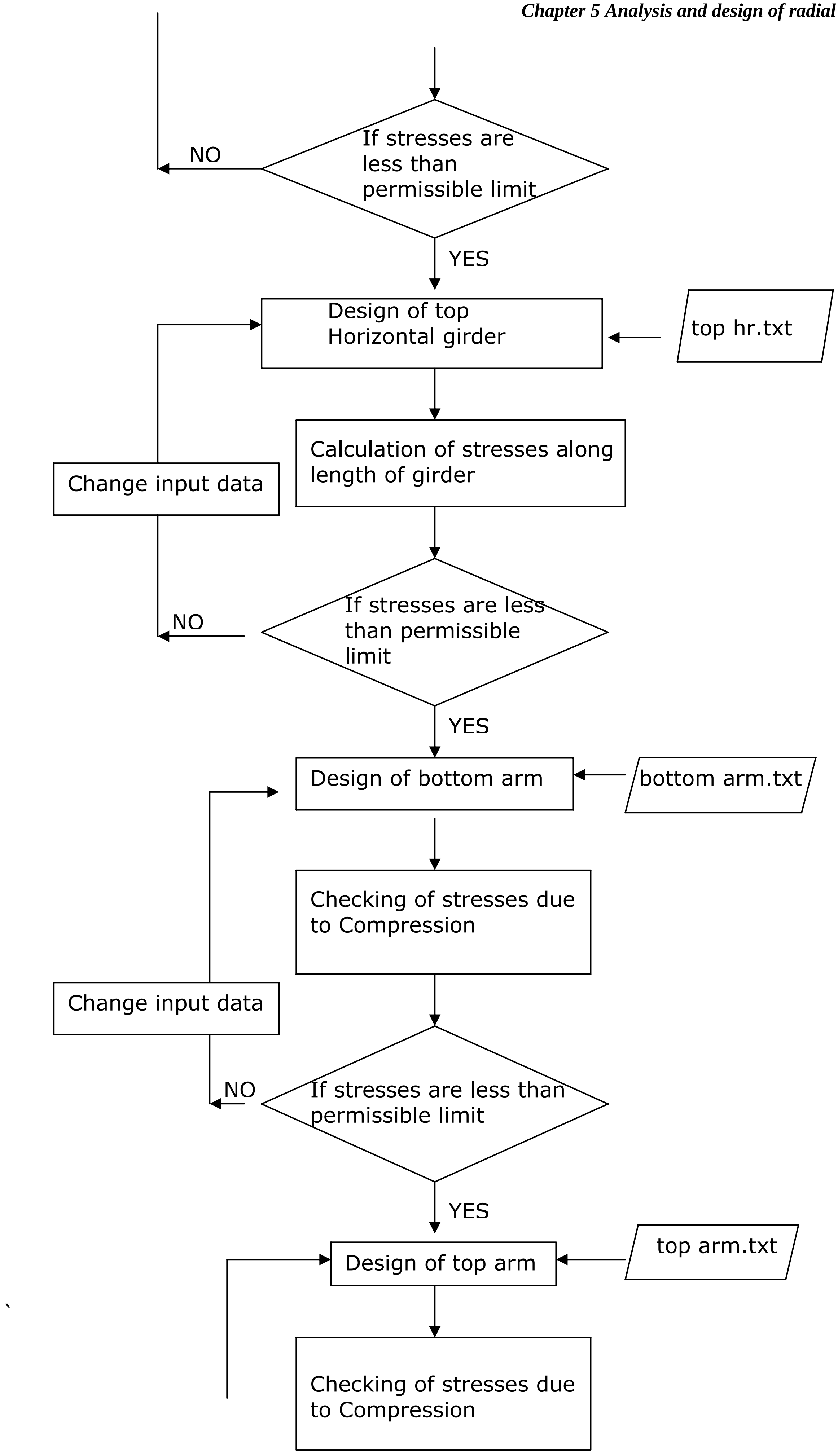


Figure 5.14 Flow chart of overall design of radial gate


Start
Level1.txt
Level2.txt
Calculation of Hydrostatic pressure, Shear force, Moments, Reactions
Calculation of stress along height of gate
tee.txt
Change input data
NO
If stresses are less than permissible limit
YES
Input from Screen
Curtailment of Skin plate
Change input data
Stresses along height of gate
NO
If stresses are less than permissible limit
YES
Design of bottom Horizontal girder
bottom hr.txt
Calculation of stresses along length of girder
Change input data


If stresses are less than permissible limit
NO
YES
Design of top Horizontal girder
top hr.txt
Calculation of stresses along length of girder
Change input data
If stresses are less than permissible limit
NO
YES
Design of bottom arm
bottom arm.txt
Checking of stresses due to Compression
Change input data
If stresses are less than permissible limit
NO
YES
Design of top arm
top arm.txt
Checking of stresses due to Compression

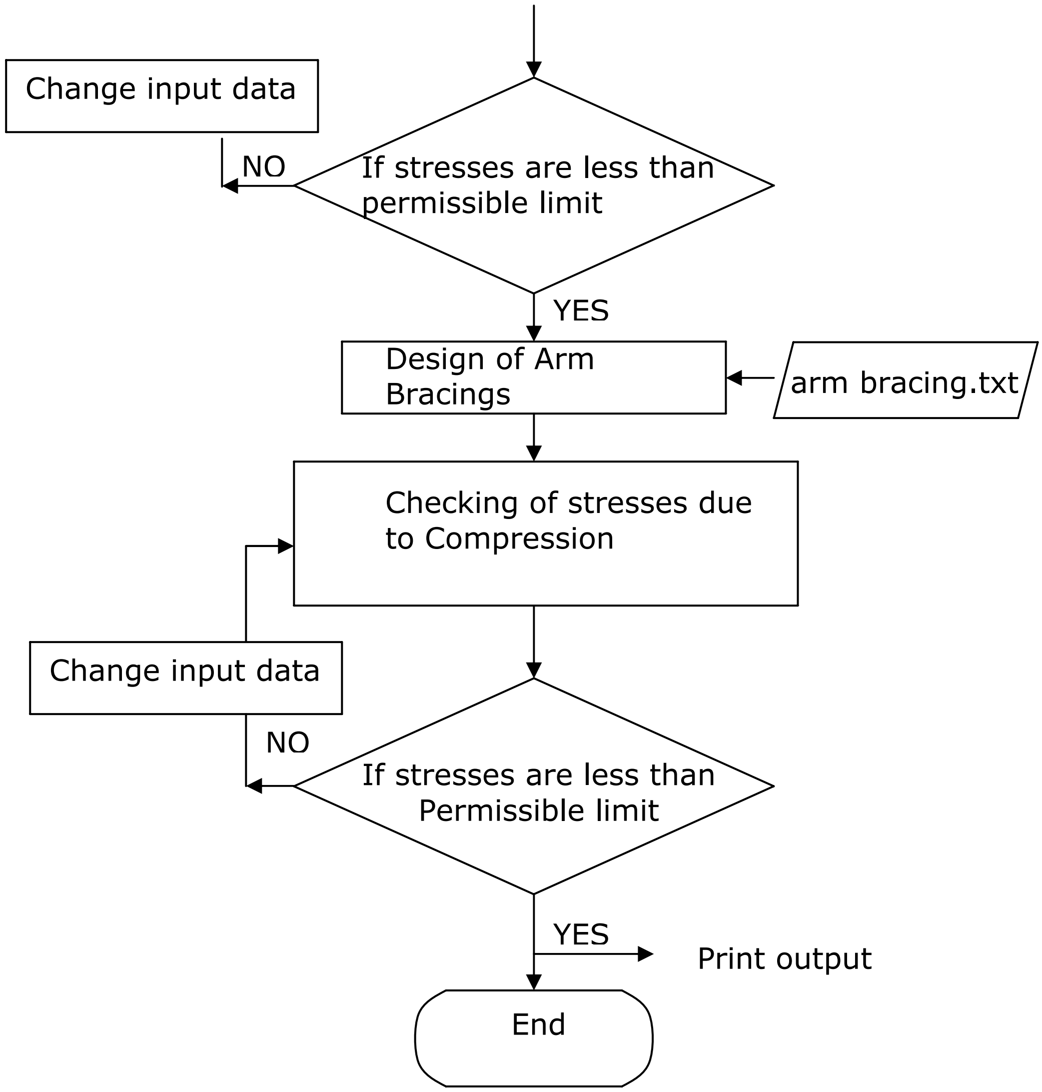


Figure 5.15 Flow chart of individual program

5.14 GIVEN DATA OF THE PROBLEM

Size of gate	=9.14m x 6.1m
HFL	=26.60m
FRL	=25.60m
Crest RL	=19.50
Foundation RL	=12.15
Trunnion RL	=23.10m
Earthquake zone	=5
Importance factor	=1.5

5.14.1 Reference figures

The sketches for various components designed by the program are drawn here for reference

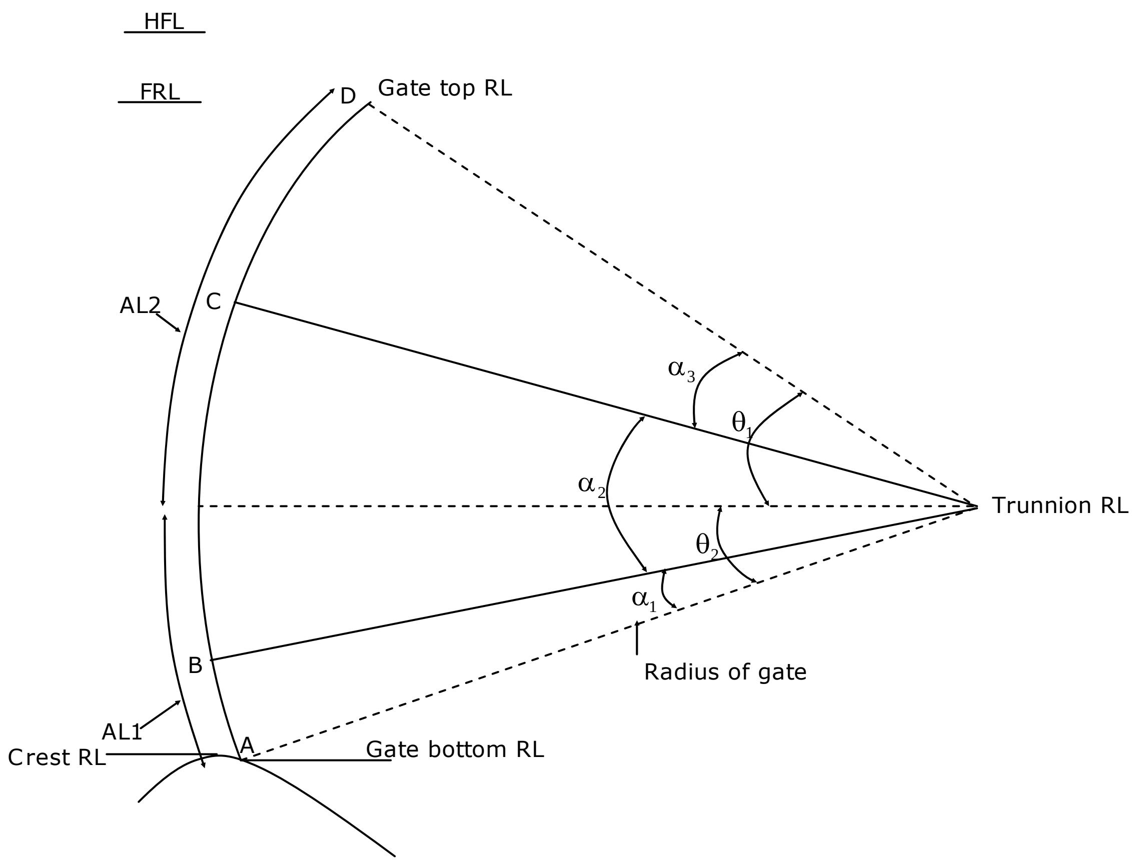


Figure 5.16 Profile of gate

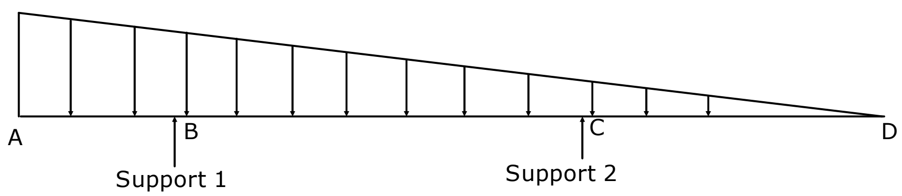


Figure 5.17 Load diagram for FRL condition

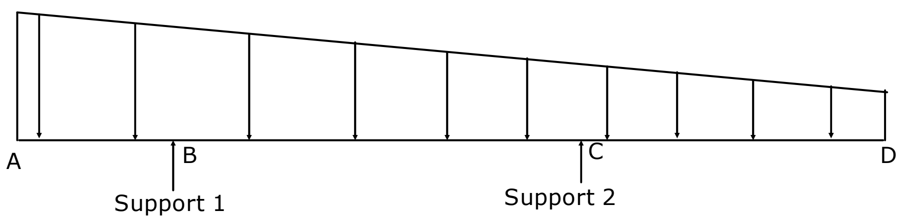


Figure 5.18 Load diagram for HFL condition

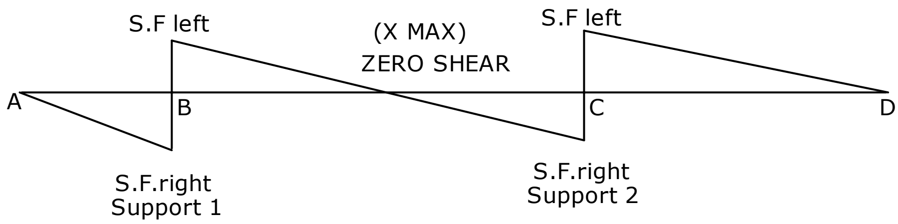


Figure 5.19 Shear force diagram along height of gate

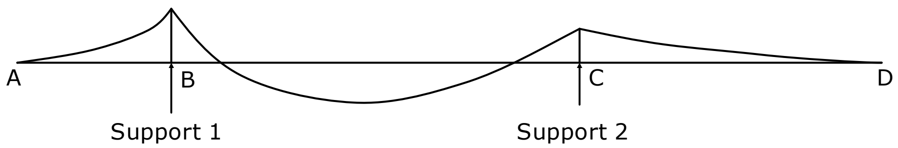


Figure 5.20 Bending moment diagram along height of gate

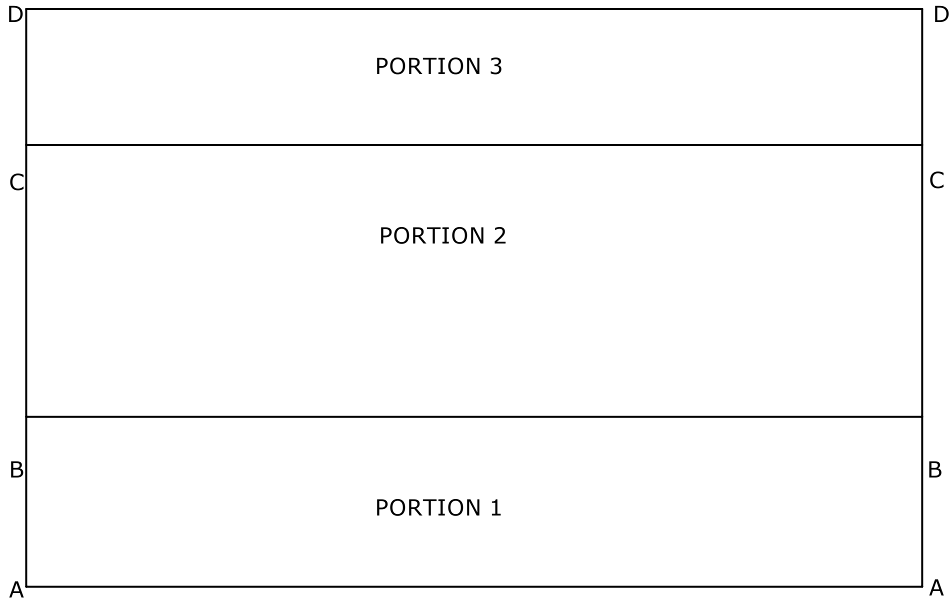


Figure 5.21 Curtailment of skin plate

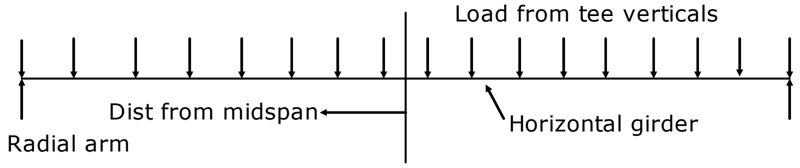


Figure 5.22 Horizontal girder

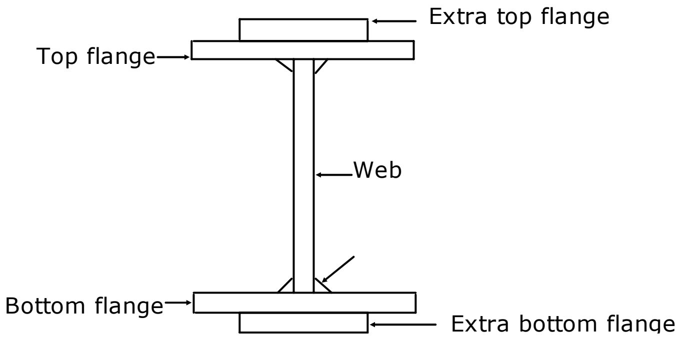


Figure 5.23 Section of horizontal girder

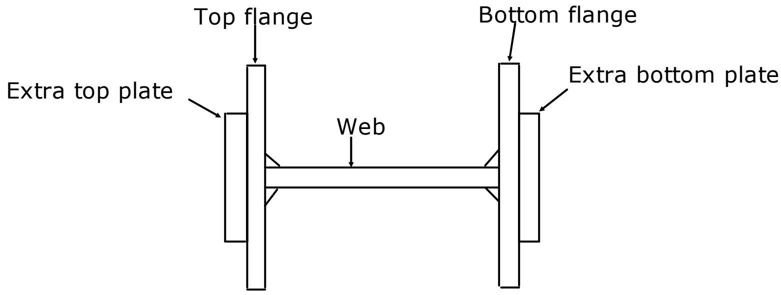


Figure 5.24 Section of radial arm

6. ANALYSIS AND DESIGN OF PIERS

6.1 PIER

The section of pier is designed so as to accommodate the gate, bridges etc. The loads acting on pier are calculated as mentioned in chapter 4. Thereafter, the stability of pier in overturning and sliding is checked. For checking the stability in Overturning, provision of IS 456-2000 clause 20.1, is followed which is follow.The stability of a structure as a whole against overturning shall be ensured so that the restoring moment shall be not less than the sum of 1.2 times the maximum overturning moment due to the characteristic dead load.

The stability in overturning is checked as per provision of IS 456-2000, clause 20.2, which is states.

The structure shall have a factor against sliding of not less than 1.4 under the most adverse combination of the applied characteristic forces The pier is designed for load combination as mentioned below,

(1) Dead load + live load
(2) Dead load $\pm$ Earthquake load
(3) Dead load + live load $\pm$ Earthquake load
(4) Dead load + Wind load
(5) Dead load + live load $\pm$ wind load

The load combination at point 3 is critical as compared to other. For size of pier 12.73m x 2.44m wind load, as per provision of IRC 6 -2000 is 41.49 kN and earthquake force is 191.85 kN. So generally taken into consideration only first combination third combination.

Once the stability criteria are satisfies, the pier is designed as column subjected to biaxial bending. The pier is design by working stress method and stresses along x and Y direction is worked out. The using provision of IS 456-2000.
$\sigma_{cbx} + \sigma_{cby} - \sigma_c \geq 0.35 \times \left(\sigma_c + \sigma_{cbx} + \sigma_{cby}\right)$. If this equation is satisfy then section is cracked section i.e. RCC member. In wall type pier nominal steel would be sufficient to take the stresses. The equation is given in book “Design of RCC” by Dean Peabody for finding the steel required as consider pier as biaxial member

$$k^3+3\times\left(\frac{e_{xx}}{t}-\frac{1}{2}\right)\times k^2+3\times p\times\left[\frac{e_{xx}}{t}\times(2\times m-1)+\frac{a}{t}\right]\times k=3\times p\times\left\{m\times\left[2\times\left(\frac{a}{t}\right)^2+\frac{e_{xx}}{t}\right]+\frac{d_1}{t}\left[\frac{a}{t}-\frac{e_{xx}}{t}\right]\right\} \tag{6.1}$$

Where,

t = thickness of pier,

l = length of pier,

e_{xx} = eccentricity in x-x direction,

m = modular ratio,

d_1 = effective cover,

$p = \dfrac{\text{Area of steel}}{\text{Area of concrete}}$

k, a and d_2 are defined in fig.

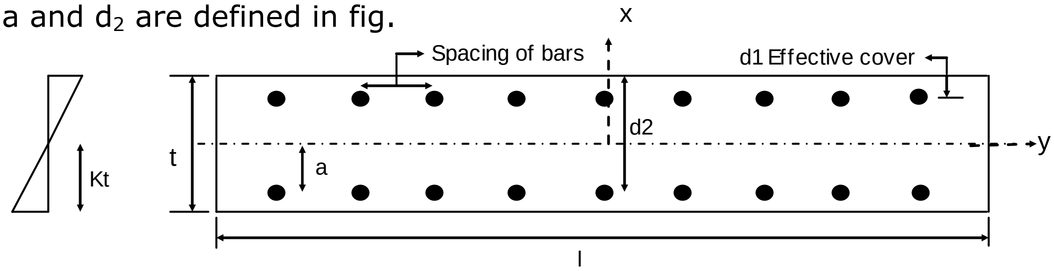


Figure 6.1 Reinforcement detailing and notation as per book dean Peabody

Stress in concrete is given by

$$\sigma_c=\frac{N\times k\times 2}{l\times t\times\left[k^2+m\times p\times(2\times k-1)-p\times\left(k-\frac{d_1}{t}\right)\right]} \tag{6.2}$$

Stress in steel is given by

$$\sigma_s=\sigma_c\times m\times\left(\frac{d_2-k\times t}{k\times t}\right) \tag{6.3}$$

Where,

N = normal force,

First of all assume all dimension of pier also assume area of steel and from equation 6.1 find value of k with trial and error. After this find the stress in steel and concrete which must less than permissible stresses. Find steel required from trial and error method.

Footing of the pier is normally anchored in the spillway itself, in case of rock, whereas in case permeable foundation, generally raft foundation is designed. In this project only isolated footing with uniaxial moment is designed looking to the time frame allotted for the project.

7.1 GENERAL

As the aim of the project is to prepare a program for analysis and design of radial gate and piers and to validate the same in the parametric studies and check the results with readily available commercial software. An effort is made to generate the radial gate in STAAD PRO and results of this same are compare.

7.2 INTRODUCTION OF STAAD PRO 2005

STAAD.Pro 2005 is the most popular structural engineering software product for 3D model generation, analysis and multi-material design. It has an intuitive, user-friendly GUI, visualization tools, powerful analysis and design facilities and seamless integration to several other modeling and design software products. The software is fully compatible with all Windows operating systems but is optimized for Windows XP. The Graphical User Interface (GUI) of STAAD.Pro include model generation, structural analysis and design, result verification, and report generation.

7.3 MODELING OF RADIAL GATE IN STAAD PRO 2005

As above stated comparison with C++ program with STAAD PRO 2005. The data considered is same as that of program prepared, the data is mentioned below.

Given data

Gate size 9.5 x 6.4

FRL	= 25.60m
Crest level	= 19.20m
Trunnion level	= 23.10m
Radius of gate	= 7.315m
No of Tee verticals	= 16 no.

Skin plate thickness = 12mm and 10mm

The radial gate is modeled as a spatial steel frame. The skin plate is idealized as rectangular shell element, resting on tee verticals, which in turn rests on horizontal girders. The model generated shown in fig. 7.1. The hydrostatic pressure is applied default trapezoidal pressure command of STAAD PRO as shown in fig. 7.4 and 7.5. The Bending moment diagram for in horizontal girders is shown in fig. 7.7 and 7.8. The bottom girder shows higher values of bending moment resisting greater hydrostatic pressure from bottom. The shear force diagram shown in fig. 7.10. The End reaction in trunnion girder shown in fig. 7.9.

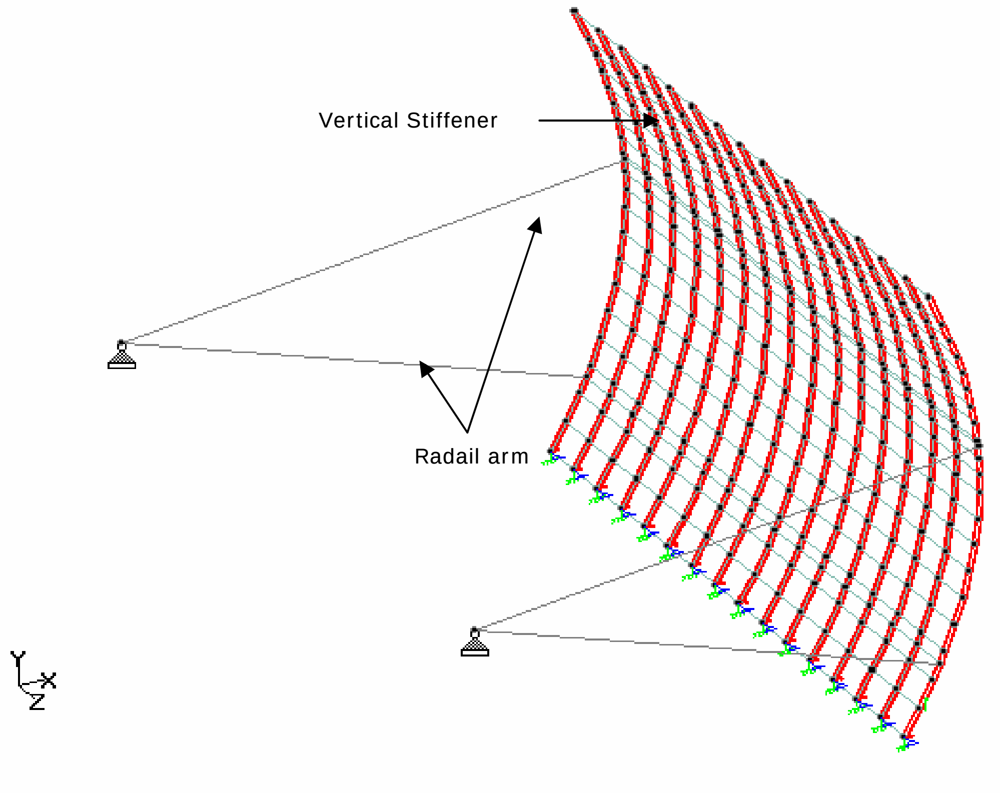


Figure 7.1 Model of radial gate with tee verticals

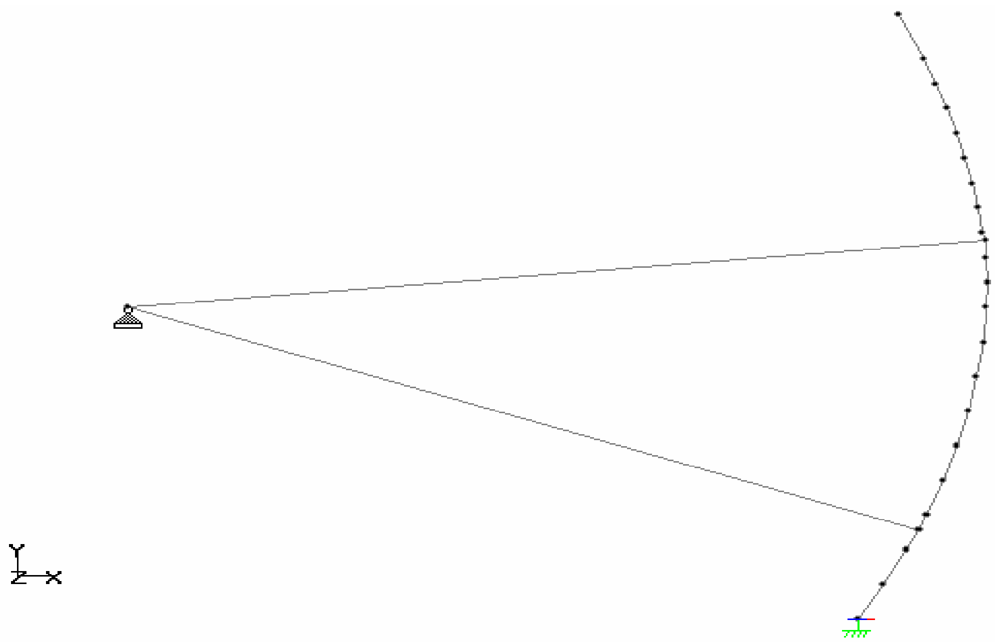


Figure 7.2 View from (+Z) direction (side elevation)

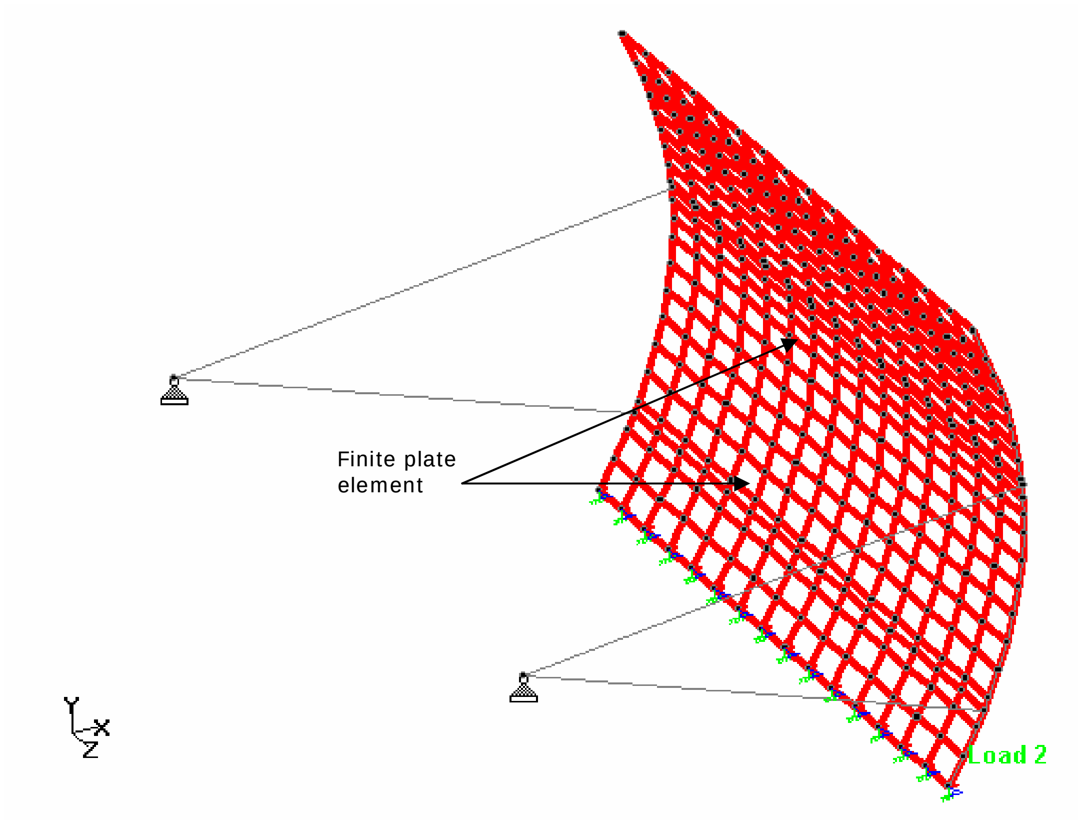


Figure 7.3 Skin plate as plate element

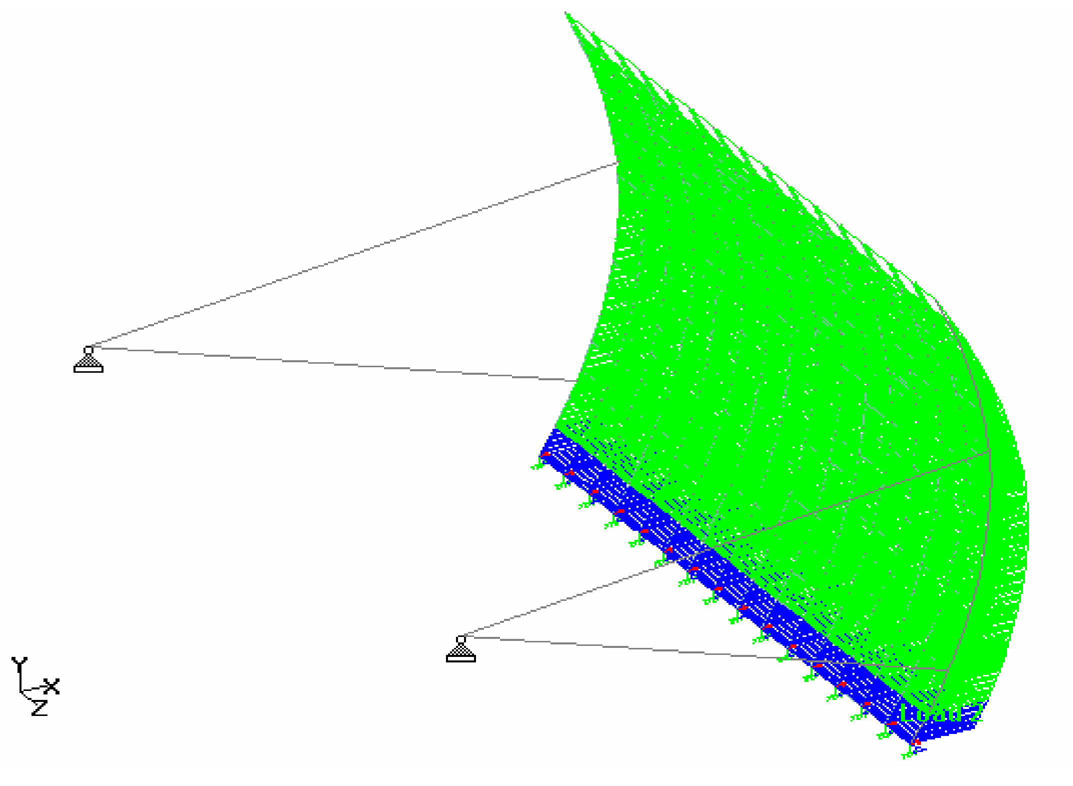


Figure 7.4 Hydrostatic pressures on radial gate

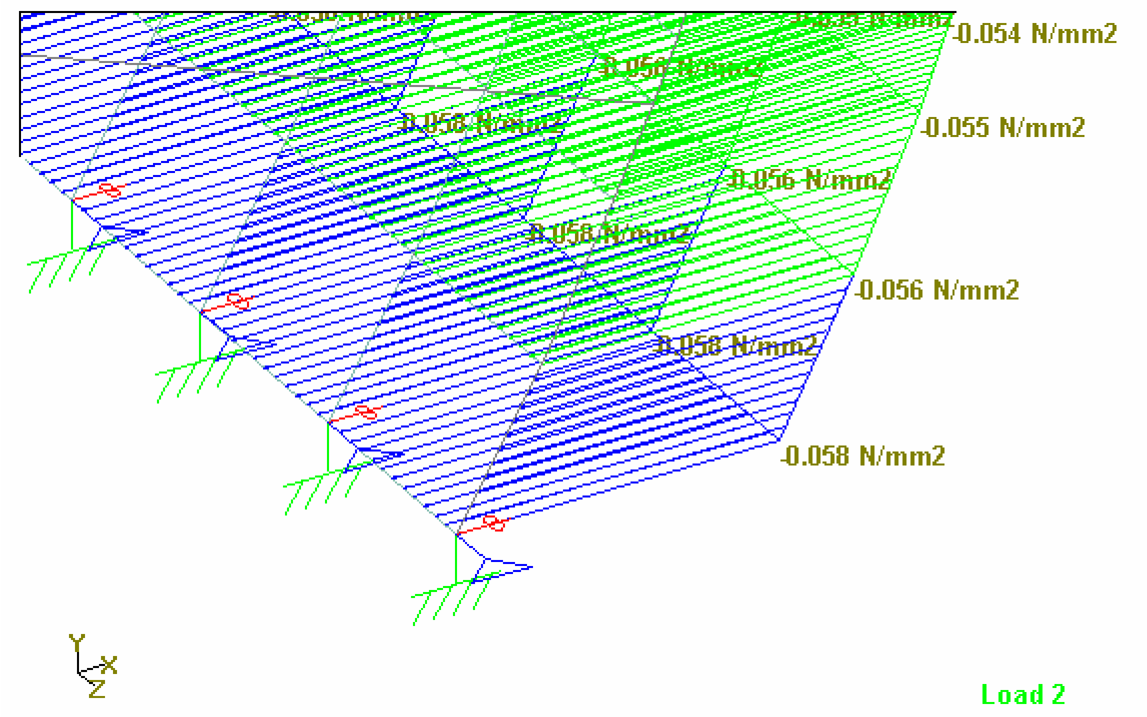


Figure 7.5 Hydrostatic pressure in plate elements

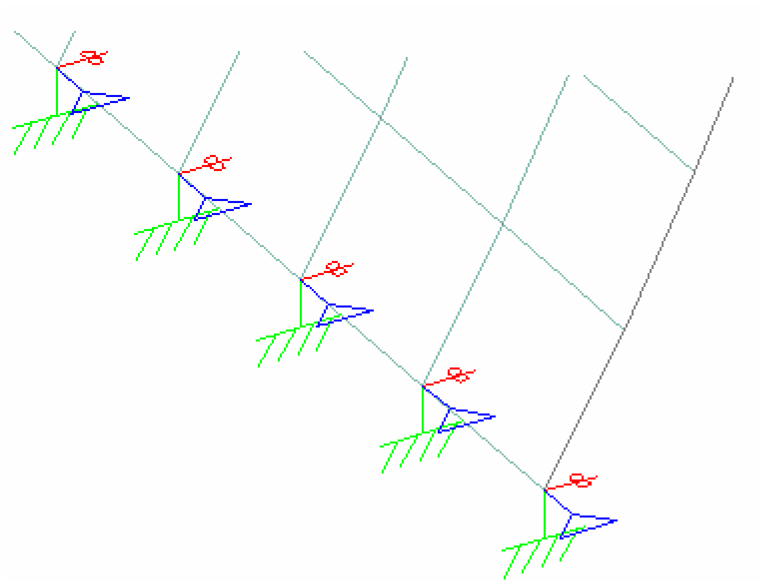


Figure 7.6 Support condition at sill level of radial gate

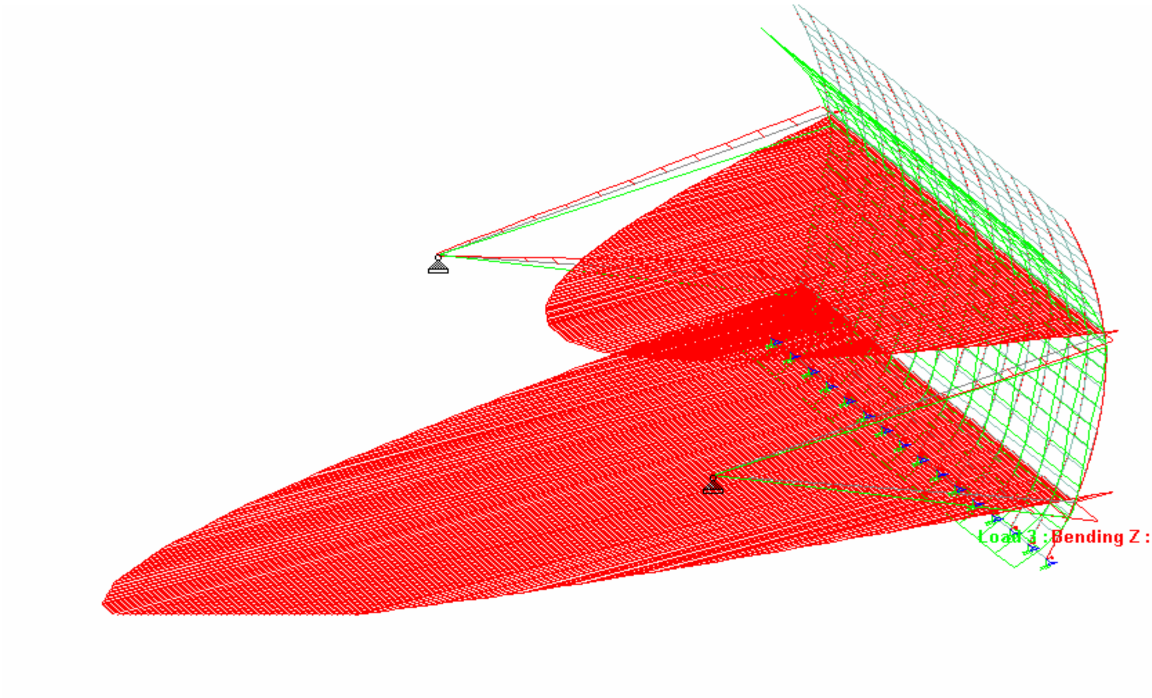


Figure 7.7 Bending moment in Z direction in top and bottom girder

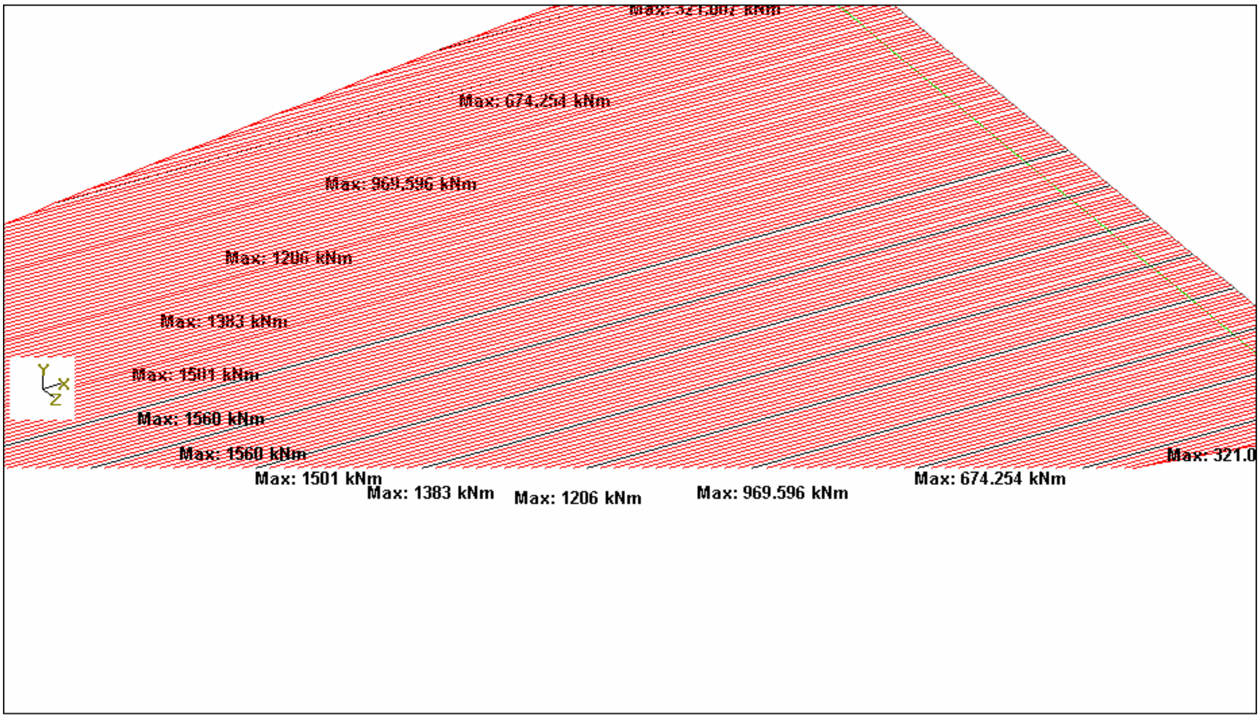


Figure 7.8 Bending moment diagram in bottom girder

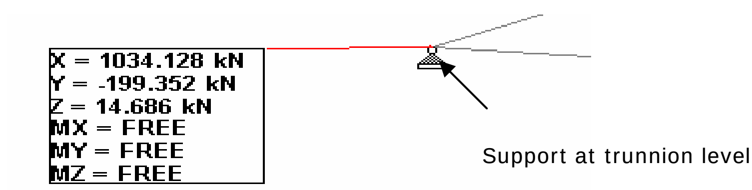


Figure 7.9 Reactions in trunnion girder

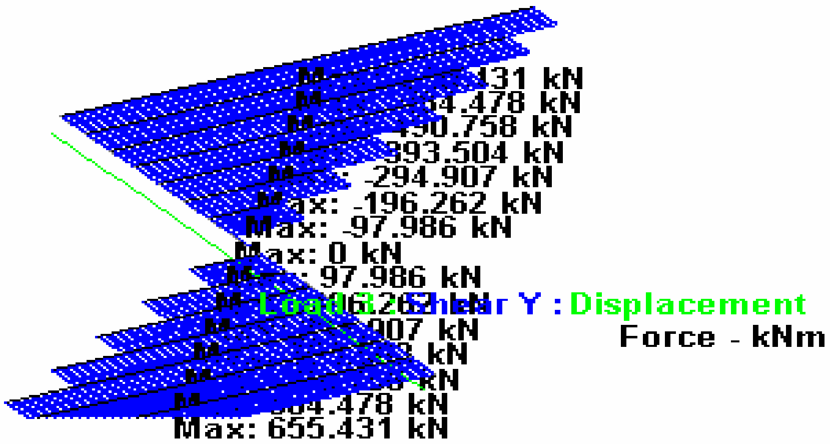


Figure 7.10 Shear force diagram in bottom girder

7.4 CASE STUDY

Table 7.1 Comparison with STAAD PRO and C++ Program

Particulars	Position	STAAD PRO 2005	C++ Program	% Variation
Bottom horizontal girder	1) Moment at centre of span (kN.m)	1560	1559.5	0.03
	2) Reaction at end of span (kN)	655.43	690.04	5
Top horizontal girder	1) Moment at centre of span (kN.m)	790	848.24	6
	2) Reaction at end of span (kN)	339.38	375.33	9.5
Trunnion	1) Horizontal Reaction (kN)	1034.12	1007	2.5
	2) Vertical Reaction (kN)	199.35	235.65	15

Table 7.2 Comparison of weight of radial gate

	STAAD PRO (Including bracings)	C++ Programming (Including bracing)	C.D.O Deptt. (Including bracing)
Weight (kN)	160	158.5	170

7.5 PARAMETRIC STUDY

Selection of various parameters viz. Radius of gate, trunnion level etc. shall be judiciously decided at the planning level. If proper values of above data is not adopted, then it may result into heavier gates and therefore effort required to lift the gate increases. Thus program of this kind helps in accurately deciding the values of data as discussed below.

(1) Radius of curvature of gate

IS 4623 1984 page 11 clause 5.3 gives the value of radius as 1H to 1.25H i.e a variation of radius for 9.15m x 6.3m from 6.3m to 7.87m for a given head of 6.3m. If 6.3m value is taken, the wt of gate comes 159.1kN whereas 7.87m is taken value of weight of gate comes out to be 158.6kN, where as optimum gate radius should be 1.2H = 7.56m. Graphs depict the results for gate sizes 9.15m x 6.3m and 12.5m x 8.3m.

Table 7.3 Weight and hoisting capacity with variation in radius of radial gate (9.15 m x 6.3m size gate)

Radius (m)	Weight of radial gate (kN)	Hoisting capacity (kN)
0.7 H	160.6	184.7
0.8 H	160.2	184.2
0.9 H	159.7	183.5
1.0 H	159.1	182.7
1.1 H	158.6	182
1.2 H	158.4	181.7
1.3 H	159.1	182.6

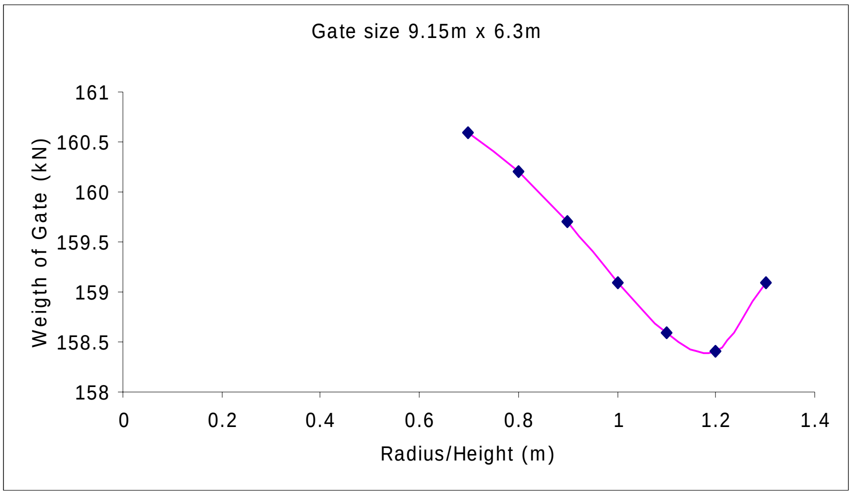


Figure 7.11 Radius/height Vs Weight of gate (9.15m x 6.3m)

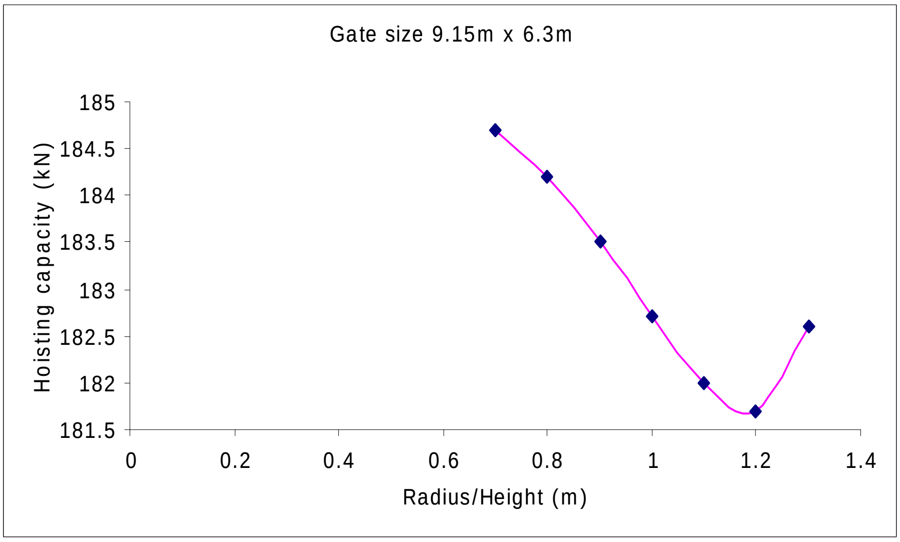


Figure 7.12 Radius/height Vs Hoisting capacities (9.15m x 6.3m)

Table 7.4 Weight and hoisting capacity with variation in radius of radial gate (12.5m x 8.3m size gate)

Radius (m)	Weight of gate (kN)	Hoisting capacity (kN)
0.7 H	257	274.1
0.8 H	256.6	273.5
0.9 H	255.9	272.6
1.0 H	255.2	271.7
1.1 H	254.6	270.7
1.2 H	254.4	270.2
1.3 H	255.1	271.6

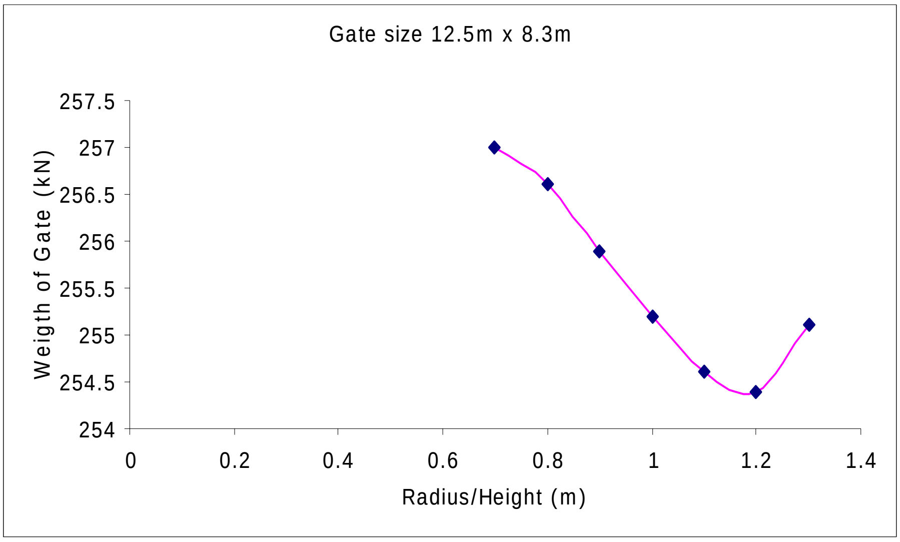


Figure 7.13 Radius/height Vs Weight of gate (12.5m x 8.3m)

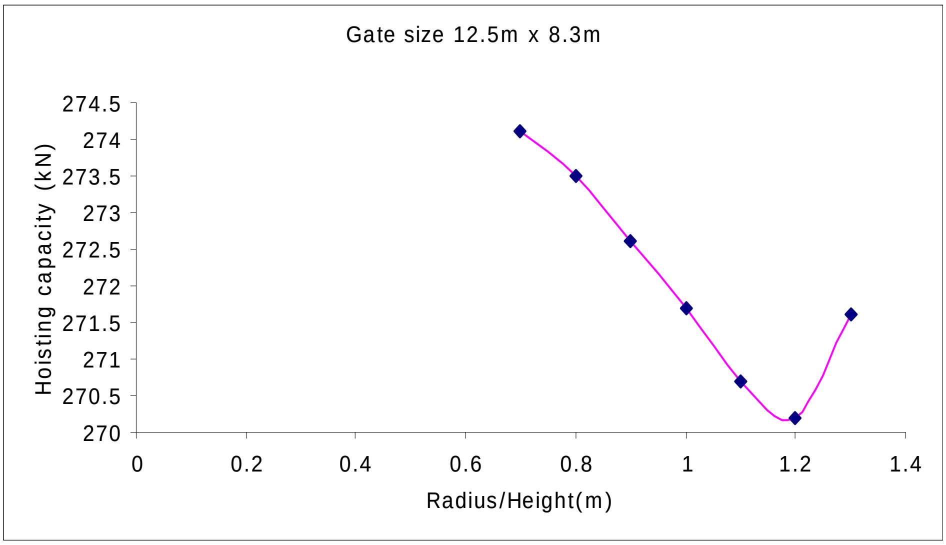


Figure 7.14 Radius/height Vs Hoisting capacities (12.5m x 8.3m)

(2) Fixing trunnion level

The location of trunnion level affects the hoisting capacity. Some times, it is not possible to place trunnion exactly the centroidal axis of the gate. Therefore, the program gives ideas of increase or decrease in hoisting capacity due to various location of trunnion. Graph shows that if trunnion is located at about 0.6H, it gives optimum weight of the radial gate. The weight of gate increases, if trunnion is placed above or below this level

Table 7.5 Weight and hoisting capacity with variation of trunnion level of radial gate (9.15m x6.3m size gate)

Trunnion position from Bottom RL	Weight of gate (kN)	Hoisting capacity (kN)
0.1 H	161	187.3
0.2 H	159.4	187
0.3 H	158.2	186.6
0.4 H	157.4	186.3
0.5 H	157	186.1
0.6 H	156.8	186
0.7 H	157.6	186.1
0.8 H	159.9	186.3
0.9 H	163.1	186.6
1 H	167.1	187

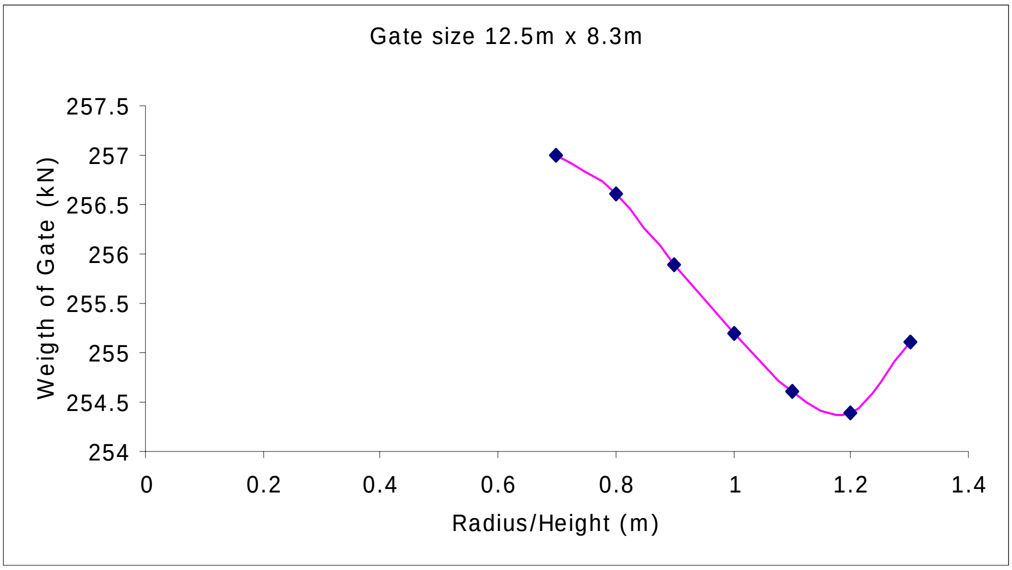


Figure 7.15 Trunnion height from bottom RL /Height Vs Weight of gate (9.15m x 6.3m)

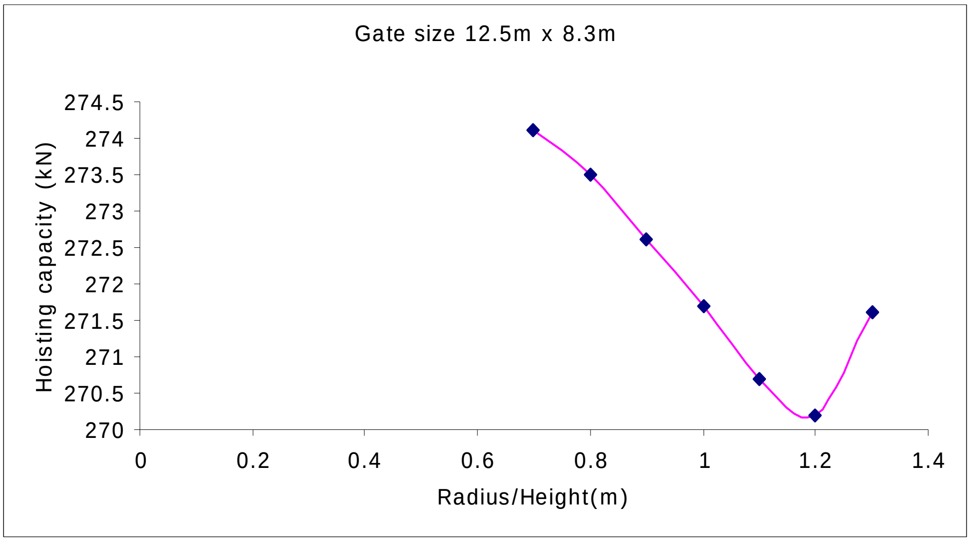


Figure 7.16 Trunnion height from bottom RL /Height Vs Hoisting capacities (9.15m x6.3m)

Table 7.6 Weight and hoisting capacity with variation of trunnion level of radial gate (12.5m x 8.3m size gate)

Trunnion position from Bottom RL	Weight of gate (kN)	Hoisting capacity (kN)
0.1 H	245.6	271.3
0.2 H	242.4	271
0.3 H	240	270.6
0.4 H	238.5	270.3
0.5 H	237.6	270.1
0.6 H	237.2	270
0.7 H	238.9	270.1
0.8 H	243.5	270.3
0.9 H	249.8	270.6
1 H	257.8	271

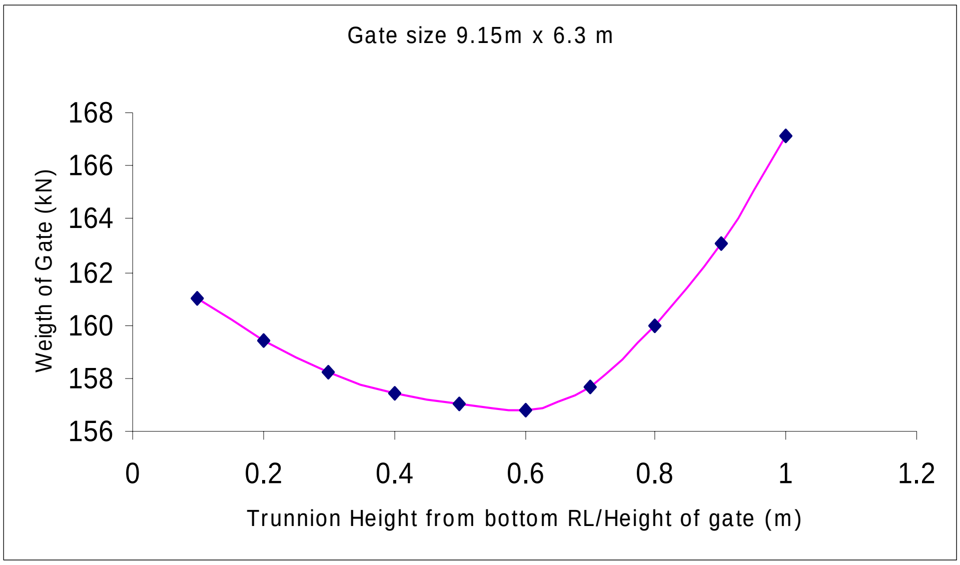


Figure 7.17 Trunnion height from bottom RL /Height Vs weight of gate (12.5m x 8.3m)

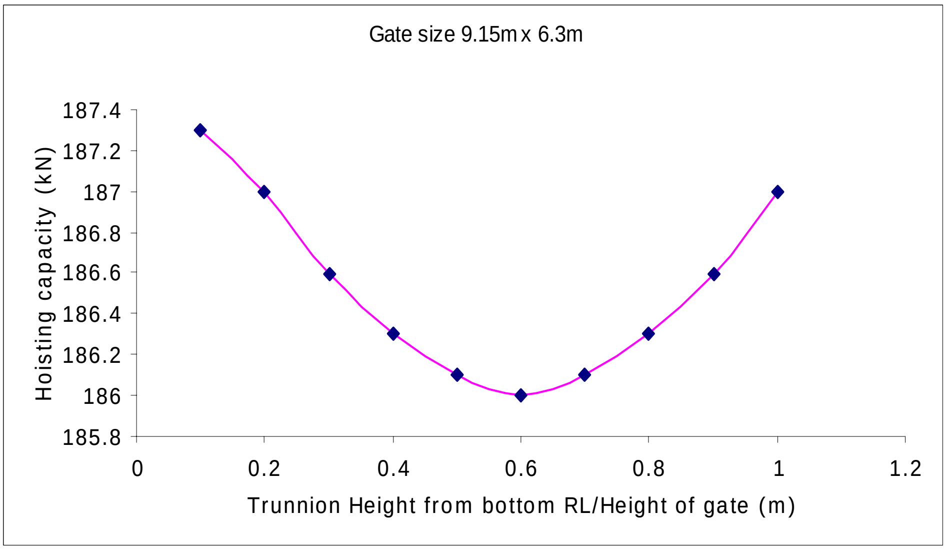


Figure 7.18 Trunnion height from bottom RL /Height Vs Hoisting capacities (12.5m x 8.3m)

(3) Comparison with different loading conditions.

Table 7.7 Comparison with different loading condition

Element	Related matter	FRL	HFL	FRL+EQ
Skin plate	Stress due to moment Zcombine(N/mm2) at point B 12mm thick	117	131	153
Bottom horizontal girder	Stress of center span (N/mm2)	106	117	113
Top horizontal girder	Stress of center span (N/mm2)	105	148	117
Bottom Arm	Compressive stress (N/mm2)	94.9	104	101
Top arm	Compressive stress (N/mm2)	51.6	52.2	57.9

From above table it is observed that the stresses in various components of the gate are governed either HFL or earthquake condition in the table considered the skin plate and radial arm is governed by earthquake condition where as the design of girder by HFL the design values are change with change in earthquake zone and /or HFL .

8. SUMMARY AND CONCLUSION

8.1 SUMMARY

The analysis and design of radial gate and its piers involves knowledge of all spheres of structural engineering. The project carried out can be summarized under following heads:

(1) Design of gate

The gate was design as the radial gate following the provision of IS 4623-1984. The composite section consisting of skin plate and tee verticals was designed to resist hydrostatic pressure. The girders supporting the skin plate and tee verticals were design as beam under flexure to resist hydrostatic pressure, resting on radial arms. Thereafter the radial arm were designed as compression member, the slenderness aspect of the radial arms were checked by provision of suitable bracings, which are to resist 2.5% of load, the radial arm carried. The design was carried for FRL, HFL, and seismic load condition.

(2) Design of pier and its footings

The pier was design as wall type pier as per provision of IS 13551-1992. The pier was analyze as column subjected to biaxial bending. The pier was design to take load of gate, bridge etc., hydrostatic load, seismic loads,

Footing was designed as isolated footing to give proper shape to the project. As such actual footings are either designed as "resisting on spillway", in case of rock foundation or "raft foundation "in case of permeable foundation.

In this project, the footing was designed as footing subjected to uniaxial moment in longitudinal direction.

As part of the project, the analytical results of the radial gate were checked with results obtained from standard software i.e STAAD PRO, for this spatial model of gate was prepared and accordingly analysis was carried out for FRL condition. The results obtained were appreciated matching.

8.2 CONCLUSIONS

After carrying out entire analysis and design of gate and studying various parametric studies, following points can be concluded for the project carried out.

(1) The design of skin plate is governed by earthquake because of additional hydrodynamic forces, the HFL condition may become critical if the difference between HFL and FRL is more than the value considered in the project

(2) The loads on bottom horizontal girder is found to be greater than that of top girder, in this project HFL condition is found to be more critical than the FRL + EQ condition

(3) IS 4623-1984. Specifies range of 0.1H to H for provision of trunnion were H= head of water, it was observed that the hoisting capacity to lift the gate is minimum at 0.6H, similarly IS 4623-1984. specifies the radius of gate to be within a range of 0.7H to 1.3H. It was found that the hoisting capacity is minimum for radius is 1.2H, where h = head of water.

(4) The pier found to be extremely stable in longitudinal direction but is found to be critical in transverse direction. When one gate is closed and other adjacent gate is open. The reinforcement is accordingly design for transverse direction

8.3 FUTURE SCOPE OF WORK

(1) Radial gate with radial arms having three members may be design and advantage of same over radial arm with two members may be studies, for higher value of water heads.

(2) Raft type foundation may be design instead of isolated footing.

(3) Other type of gate may be thought instead of radial gate.

(4) Prestress type anchoring system.

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APPENDIX A

Material and type of stress		Wet Condition		Dry Condition	
		Accessible	Inaccessible	Accessible	Inaccessible
1.	Structural steel				
	a).Direct compression and compression bending	0.45YP	0.40YP	0.55YP	0.45YP
	b).Direct tension and tension bending	0.45YP	0.40YP	0.55Yp	0.45YP
	c). shear stress	0.35YP	0.30YP	0.40YP	0.35YP
	d). Combined stress	0.60YP	0.50YP	0.75YP	0.60YP
	e).Bearing stress	0.35UTS	0.25UTS	0.75UTS	0.34UTS
2.	Bronze				
	Direct bearing stress	0.035UTS	0.030UTS	0.040UTS	0.035UTS

NOTE 1 — YP stands for minimum guaranteed yield point stress, UTS stands for ultimate tensile strength. For materials which have no definite yield point, the yield point, may be taken at 0.2 percent proof stress.

NOTE 2 — The term 'wet condition" applies to skin plates and those components of gate which may have a sustained contact with water, for example, horizontal girder and other components located on upstream side of skin plate. The term 'dry condition' applies to all components which generally do not have a sustained contact with water, for example, girders, stiffeners, etc, on downstream side of skin plate, even though there may be likelihood of their wetting due to occasional spray of water.

NOTE 3 — The term accessible applies to gates which are kept in easily accessible locations and can, therefore, be frequently inspected and maintained, for example, gates and stop logs which are stored above water level and are lowered only during operations. The term inaccessible applies to gates which are kept below water level and/or are not easily available for frequent inspection and maintenance, for example, gates kept below water level or in the bonnet space even while in the raised position or gates which on account of their frequent use are generally in water.

APPENDIX B

List of usefull website

a) http://www.google.com
b) http://www.vivismo.com
c) http://www.nicee.org
d) http://www.usarmy.org
e) http://www.nptel.iitm.ac.in