1. CIVIL ENGINEERING DESIGN

1.1 GENERAL

The civil engineering structures required in a low-head hydropower plant consist of

- Powerhouse
- Power Canal
- Retaining Walls
- Slopes
- Emergency Relieving Structures
- Dewatering

The design in respect of civil engineering point of view is given in the following sections.

1.2 POWERHOUSE

1.2.1 GENERAL

As compared to high-head hydropower plants, the low-head powerhouse is larger due to the higher discharges flowing through the turbines. Therefore the design in normally dealt into three following main:

- Hydraulic Design
- Static Design
- Structural Design

1.2.2 POWERHOUSE HYDRAULIC DESIGN

1.2.2.1 GENERAL

The dimensions of the powerhouse in horizontal plan may be categorised such as length along the longitudinal plant axis and perpendicular to the flow direction and width along the flow direction. The length of the powerhouse is determined by the number and spacing of the units. The following are important aspects to note in this regard:

- Spacing of the entrance flume, the width of the trash rack panel plus the width of the intermediate pier.
- Width of the scroll case including the width of separating wall.
- Width of draft tube including width of the dividing wall.
- Diameter of the generator plus clearance for maintenance

In case of vertical setting and when the discharge is more and head is less, the width of the scroll case is decisive in finalization of powerhouse length. The intake totally depends upon careful design of trash rack because head loss in trash rack is significant than other loss such as friction, slot, etc. Therefore, proper designing of scroll case and trash rack has to be carried out.

The width of the powerhouse will be governed by the width of structures and equipment to be accommodated side by side in the direction of flow, increased by the required operating clearance, or by the length of water passage ways, necessary to ensure favourable hydraulic conditions. The following controls the width of the powerhouse:

- Installation of trash rack, either vertical or inclined.
- Cleaning of trash rack, manually or mechanical.
- Removal of trash rack by trough or hopper.
- Stop log groove and installation requirement.
- Head gates groove and operation requirement
- In case of bulb turbines the stop log groove should be at a safe distance from the nose of the bulb so that removal of bulb can be done during installation and maintenance.

- In case of pit type turbine, installation of pit.
- Arrangement of superstructure.
- Hydraulic requirement of the draft tube.
- Downstream Stop log groove and installation requirement.

It is important to note that the width of the powerhouse mainly depends upon the length of the draft tube, which is a prime requirement from hydraulic point of view. Therefore its design needs special attention. The design of trash racks and impact of stop log groove has been discussed in mechanical engineering design.

1.2.2.2 INTAKE

The flow velocity in the entrance flume should not exceed the limit value defined by the expression.

$$v = f'' \sqrt{2gH}$$

$$f = 0.12$$

This formula gives the following

Table 1-1 Intake Velocity Vs Head

H (m)	6	8	10	15
v (m/s)	1.3	1.5	1.7	2.1

The flow velocity of 0.9 to 1.2 m/s just behind the rack is acceptable in low-head plants of high capacity. If however, the cross-section of the entrance flume decreases gradually toward the scroll case, velocities 1.5 to 2 times the above value may be adopted.

However, in literature the constant f varies between 0.12 and 0.17. Therefore:

$$v = 0.12 \text{ to } 0.17 \sqrt{2gH}$$

To avoid vortices and eddies the entrance velocity at the open flume should be

$$v < 0.075 \sqrt{2gH}$$

Therefore the free cross-section at entrance flume may be

$$A = Q/v$$

Where

A	=	Area of flume at entrance without taking in to account trash rack
Q	=	Turbine discharge
v	=	Velocity of water before trash rack

1.2.2.3 DESIGN OF SPIRAL CASE

1.2.2.3.1 GENERAL

Dimension and the shape of the scroll case are closely related to the design of the turbine. Therefore, the turbine manufacturers based on their own design and likely after extensive model studies usually provide the design. However, for preliminary design at feasibility level this approach can be used. Before addressing the design formulae for scroll case, first of all some words about scroll case are necessary.

The spiral case can be such as:

- Full spiral case as shown in Figure (a) below.

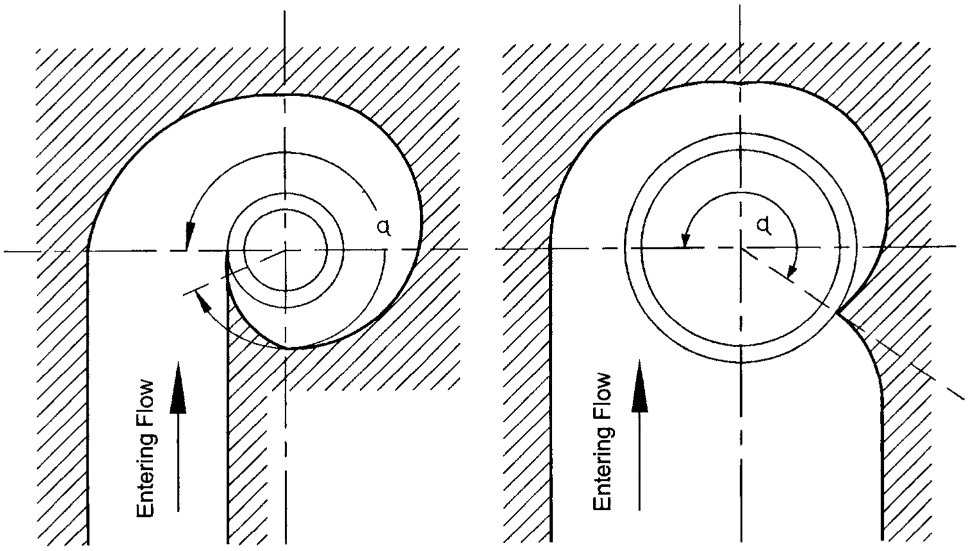


Figure 1-1 a) Full Spiral Case. b) Partial Spiral Case.

A full spiral case means that a spiral conduit entirely enclosing the turbine with a nose angle of 360^0. However, a spiral with nose angle between 320^0 and 340^0 is also possible. The full spiral case ensures the most suitable flow condition. However its application in low-head power plants is not recommendable due to the large width required as compared to partial spiral case. The full spiral case increases the overall length of the powerhouse to uneconomic levels.

- Partial Spiral Cases shown above in Figure (b).

Partial spiral case has a nose angle less than 320^0. This type of spiral case is usually employed in low-head power plants. The nose angle in low-head power plants varies between 230^0 and 285^0.

The bottom slab, roof and walls of the spiral case are of reinforced concrete. The width of the spiral case and the dividing wall usually vary between

$2.75\ D_3$ and $3.50\ D_3$

However, Morozow recommend that total width of the spiral case at power plant equipped with Kaplan turbines

$2.75\ D_3$ and $3.00\ D_3$

The width means the sum of principal and opposite radii of curvatures $R_0 + R_{180}$ as shown in Figure below.

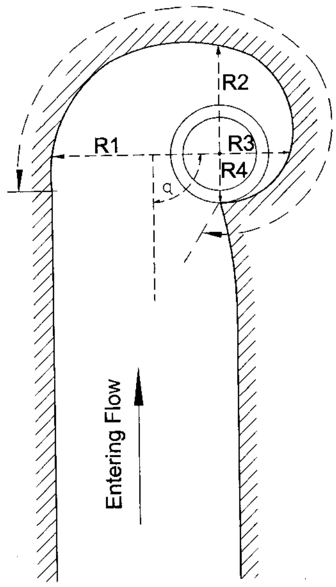


Figure 1-2 Radii of Spiral Case

1.2.2.3.2 DESIGN OF SPIRAL CASE

The design of the spiral case is governed by the flow requirements. Let us assume such as

- Spiral case of constant height
- Friction effect neglected
- Free flow in bends is defined by a velocity distribution along the radii normal to the streamlines characterised by the constant moment of momentum.

The discharge in the spiral case is given below if considering that the discharge is evenly distributed into the turbine.

$$q = \varphi \; Q/360$$

The notation of the following figure is used

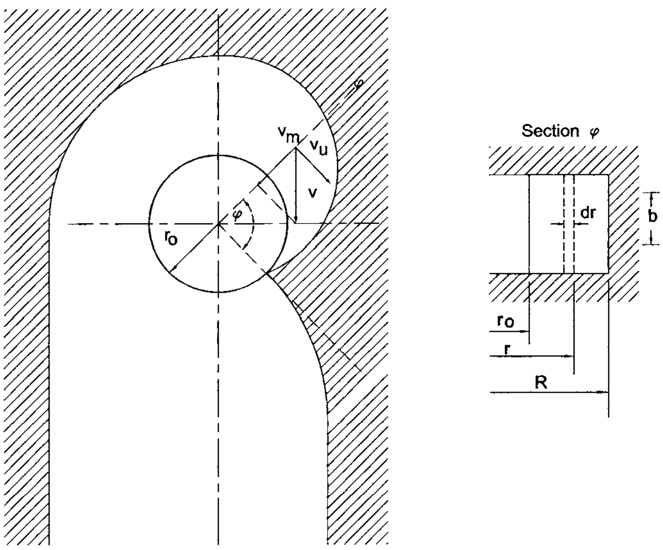


Figure 1-3 Design notation for spiral case

The discharge through the elementary width dr pertaining to angle φ is

$dq = v_u\, b\, dr = K\, b\, dr / r$

Where

v_u = Tangential velocity component in the spiral case.
K = Vortex

The discharge through the entire section can be achieved by integrating above equation between r_o and R and will be

$q = K\, b\, Ln\, R/r_o$

But normally a spiral case having section shown below is being used due to less head loss. The trapezoidal transitional part may be very steep, enclosing an angle of 12 to 15 with the vertical.

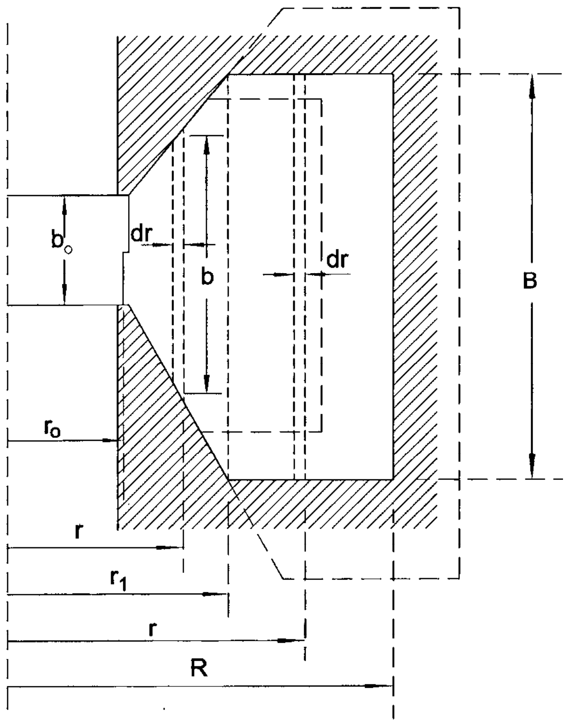


Figure 1-4 Trapezoidal Section spiral case.

The height of the section varies along the length of the spiral case and can be calculated. The rectangle of varying height B and unknown width joins the entrance mantle of height b_o by a transition trapeze section of known inclination. Therefore:

$b = b_o + \alpha\, (r - r_o)$
$B = b_o + \alpha\, (r_1 - r_o)$
$r_o = 1.4 \text{ to } 1.6\, D_3/2$
$b_o = 0.4\, D_3$

$\varphi\, Q/360 = (b_o + \alpha\, r_o)\, Ln\; r_1 / r_o + (B - b_o) + Ln\, R/ r_1$

or

$R = Num\, \log 0.4343 \{ (\alpha_1 q + 2.3026\, \alpha_2 \log ((B/\alpha)+ \alpha_3))) + 2.3026 \log ((B/\alpha)+ \alpha_3)) - 1 \}$

Where

α_1 = Q/360 K

α_2 = r_o - b_o

α_3 = r_o - b_o/α

α_4 = 2.3026α_2 log r_o - b_o

and according to figure below

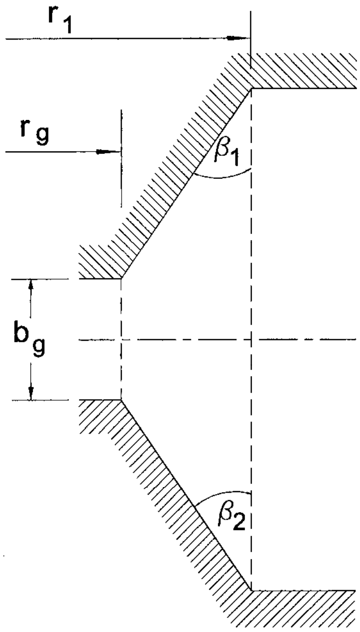


Figure 1-5 Spiral case Trapezoidal section

α = cot β_1 + cotβ_2

and

r_1 = $((B - b_o)/ \alpha) + r_o)$

The moment of momentum is obtained from turbine speed n with the help of peripheral velocity of the runner and using basic equation of turbine such as:

K = c1u D3/2

= $(30/\pi)(\eta_h g\ H/ n)$

Where

H = Rated head in meter

η_h = Hydraulic Efficiency of the Turbine = 0.93

1.2.2.4 DESIGN OF DRAFT TUBE
1.2.2.4.1 GENERAL

The term draft tube covers the whole water conduit from the exit of the turbine runner to the tailrace, where the water has a free level at atmospheric pressure. The draft tube has two basic purposes:

- To utilise the differential elevation between runner (turbine exit) and tailwater level which is known as static draft head.

- To recover part of the kinetic energy in the water leaving the turbine. This regained head is called the dynamic draft head.

There are many types of drafts tubes. The following three are the best known:

- Older design - vertical diffuser shaped draft tube
- Special design - Moody type draft tube.
- Up-to-date design - Elbow type draft tube.

Hydraulically the straight type of draft tube is the best but especially its application on bigger turbine units or even small units where excavation is difficult due to hard rock, involves high construction cost. The use of elbow type draft tube is preferable due to hydrodynamic performance to convey water coming from turbine runner in the direction of tailwater flow. The elbow type draft tube is most commonly used and its design will be discussed further. The elbow type draft tube is divided into three sections as shown in the figure below:

- Vertical section I (entrance or just after runner)
- Bend section II
- Horizontal Section III (discharge section)

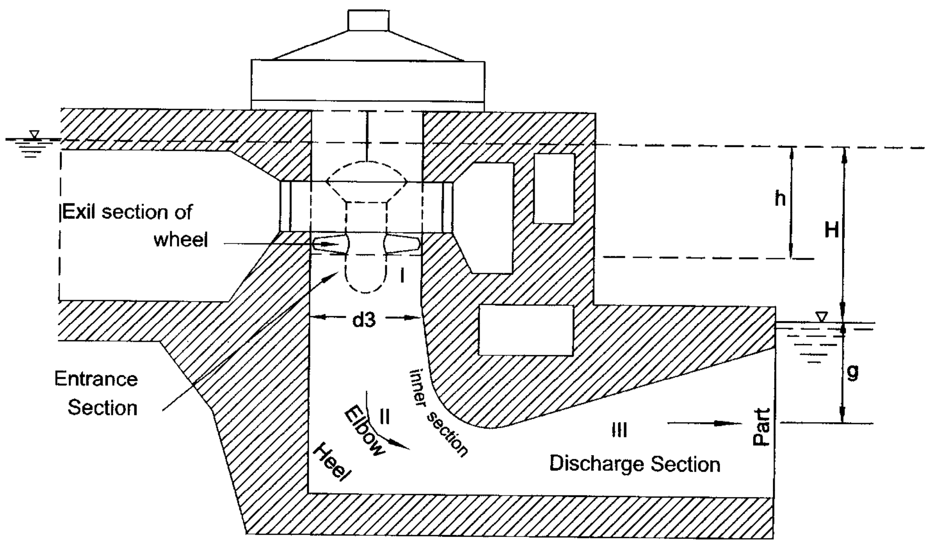


Figure 1-6 Draft tube Details

It is clear that in designing of draft tubes, the bend section is the most important because the head loss will be more due to change in flow direction. As a rule the three sections of the draft tube gradually expand like a diffuser. The most favourable shape is a curved surface on the upper or lower part of the elbow for Kaplan turbines.

1.2.2.4.2 DRAFT TUBE DESIGN

By using the notation given in Figure 1-6 and Bernoulli's theorem gives that

$$P/\gamma + c_3^2/2g + h_s + y = P_o/\gamma + c_4^2/2g + y + \delta h$$

By simplifying and rearranging we get:

$$P/\gamma = P_o/\gamma - h_s - (c_3^2 - c_4^2)/2g + \delta h$$

From Figure 1-7, which illustrates the pressure distribution within the draft tube in case of vortex free meridian flow, the dynamic draft head will be:

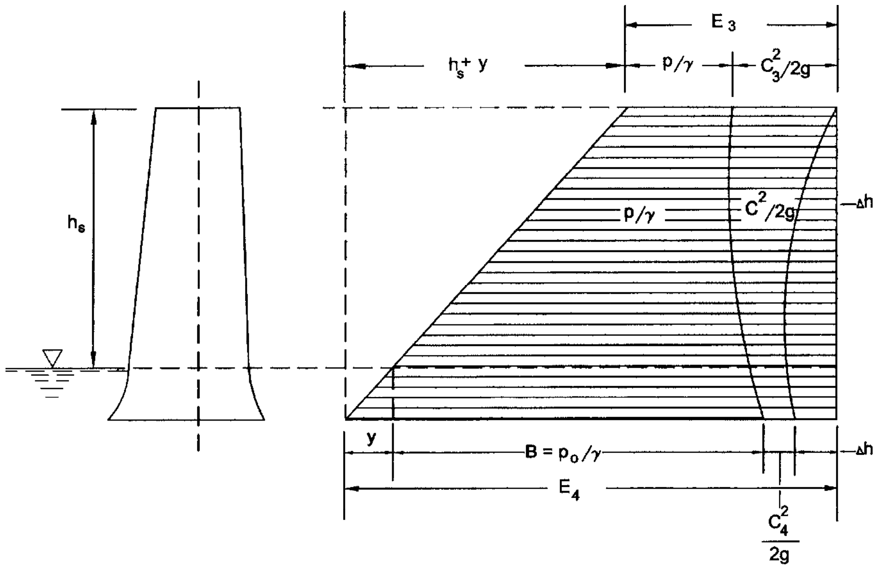


Figure 1-7 Pressure Distribution Within the Draft Tube in Case of Vortex Free Meridian Flow.

$$h_s = (c_3^2 - c_4^2)/2g + \delta h$$

If efficiency of the draft tube is η_d the dynamic draft head is

$$h_s = \eta_d (c_3^2 - c_4^2)/2g + \delta h$$

where

$(c_3^2 - c_4^2)/2g$ is the energy converted into useful work by the diffuser or draft tube.

$h_s = \eta_d = ((c_3^2 - c_4^2) - 2g\ \delta h) / (c_3^2 - c_4^2)$

By the proper shaping of the diffuser the exit velocity c_4 may be decelerated between 1 and 2 m/s and the efficiency of the draft tube may be between 75 and 80 % and even 85 %.

The enlargement in the draft pipe may be in a length L

$$\tan /2 = D_4 - D_3 /2L$$

Where

D_4 = Diameter at draft tube exit.
D_3 = Diameter at turbine runner

The kinetic head at the draft tube entrance may be:

$$c_3^2 /2g = \beta H$$

Where β is a coefficient and depend upon turbine specific speed if

$n_s = 300 - 400$ $\beta = 0.12 - 0.20$ Francis turbines
$n_s > 400$ $\beta = 0.30 - 0.40$ Kaplan turbines

From above equation we can establish that

$P/\gamma = P_o/\gamma - h_s - h_d$

It shows that when P/γ_o diminishes the result is an increase in static draft head. If the turbine is placed too high above tailwater level, cavitation may occur as a result of very low value of P/γ_o. Cavitation at the boundaries of the liquid or within the liquid can be observed when absolute pressure reduces to the saturated vapour pressure pertaining to the prevailing water temperature. Therefore to eliminate the danger of cavitation the following should be satisfied.

$P/\gamma > P_v/\gamma$

Thoma has given following relationship:

$h_s \leq B - \sigma H$

Where

B = Barometric pressure head (P_o/γ). Its value decreases with the altitude at a rate 0.11 m for every 100 meter of altitude. At sea level B= 10.3 m. For plain low to medium altitude areas where low-head power plants are located B = 10 may be used in the course of design. However for higher altitudes B= 0.95(10.3 - 0.11 E) should be used.

H = Total head

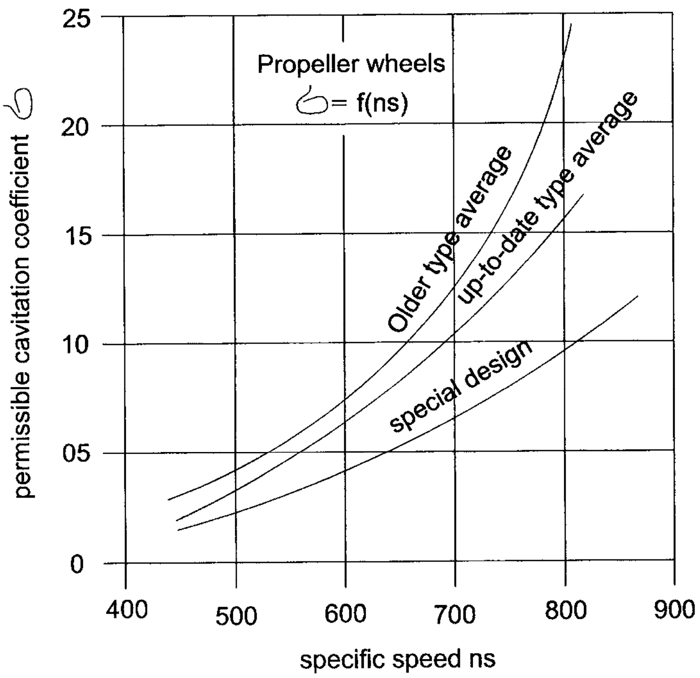


Figure 1-8 Cavitation coefficient plotted against specific speed - Kaplan Turbine.

σ = Coefficient is given by Thoma and depends upon the specific speed of the turbine. Its value increases with the increase in specific speed. It seems from this equation that the higher the specific speed, the lower the permissible draft head, in other wards, turbines with high specific speed have to be located deeper, leading to a shorter difference to the tailwater level. The value of σ for Kaplan turbine fall normally in the following range:

n_s	450	600	750	900
σ	0.43	0.65	0.95	1.50

For more information see Figure 1- 8 above.

1.2.2.5 HYDRAULIC DESIGN BASED ON EXPERIENCE

The turbine manufacturers have suggested different designs for different turbines, which are based on their experience and tests with hydraulic models. These designs are based on turbine diameter and could be changed proportionately. All these designs are given in figures below and are for Kaplan, bulb, pit type, and Francis turbines.

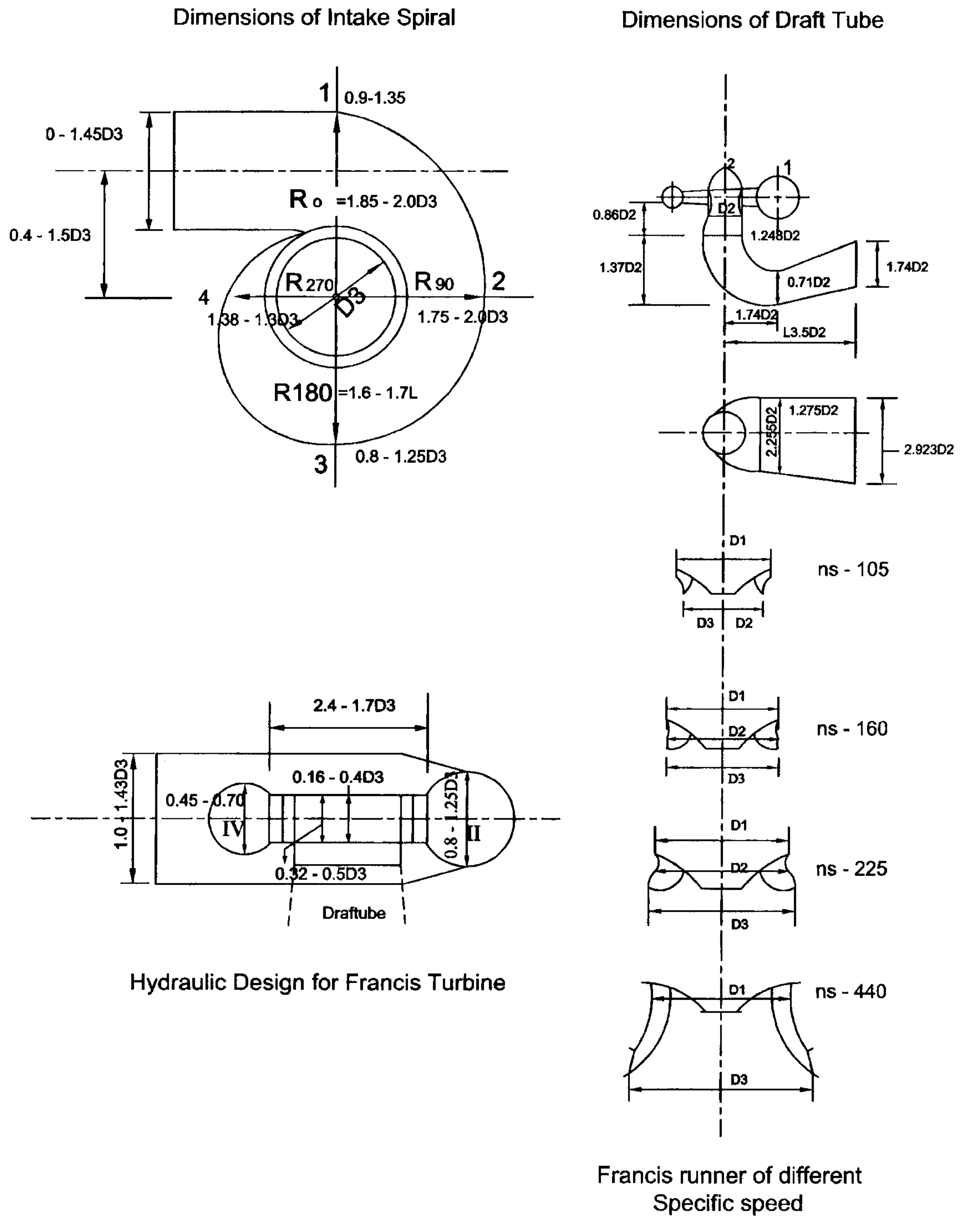


Figure 1-9 Hydraulic Design for Francis Turbine.

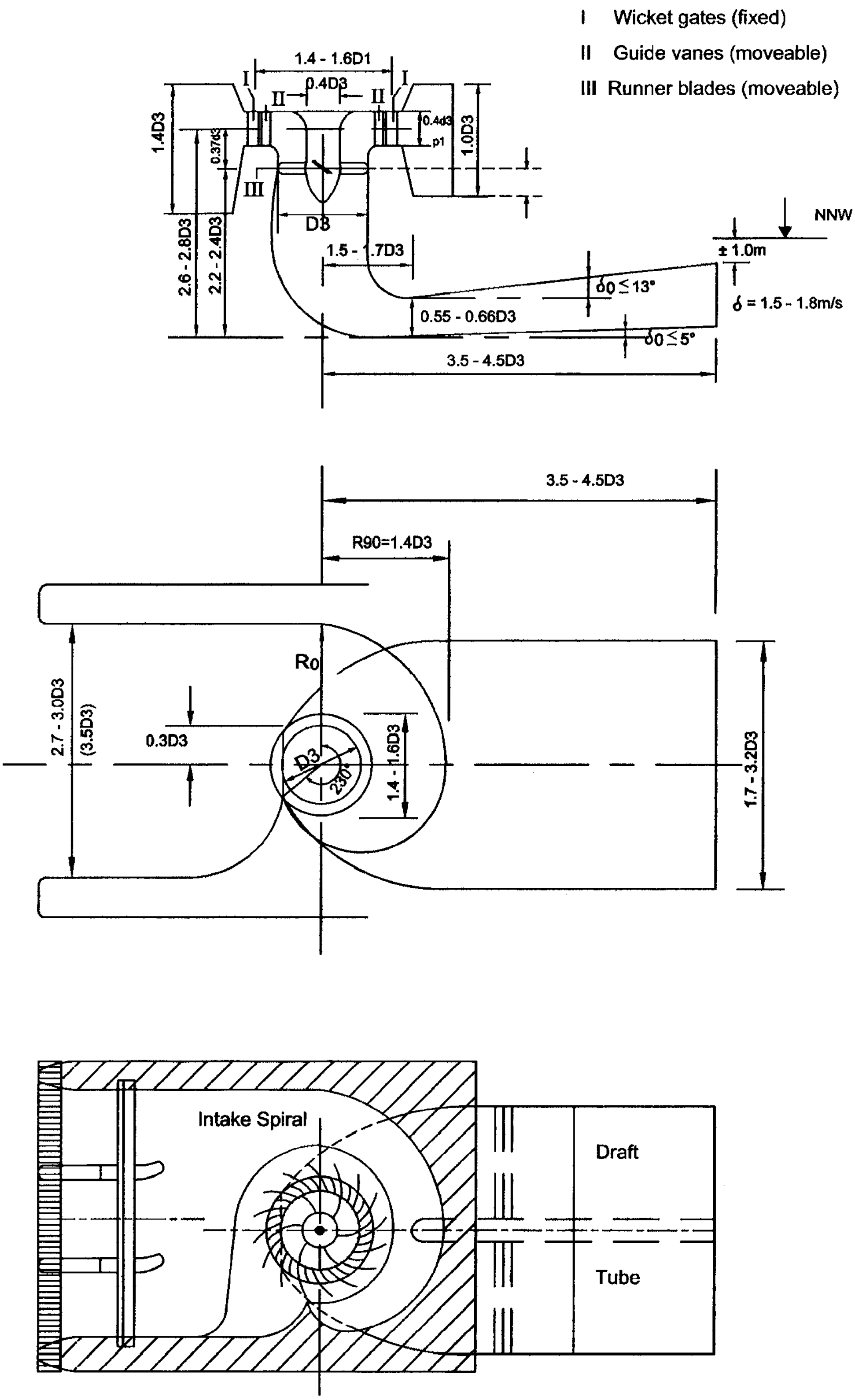
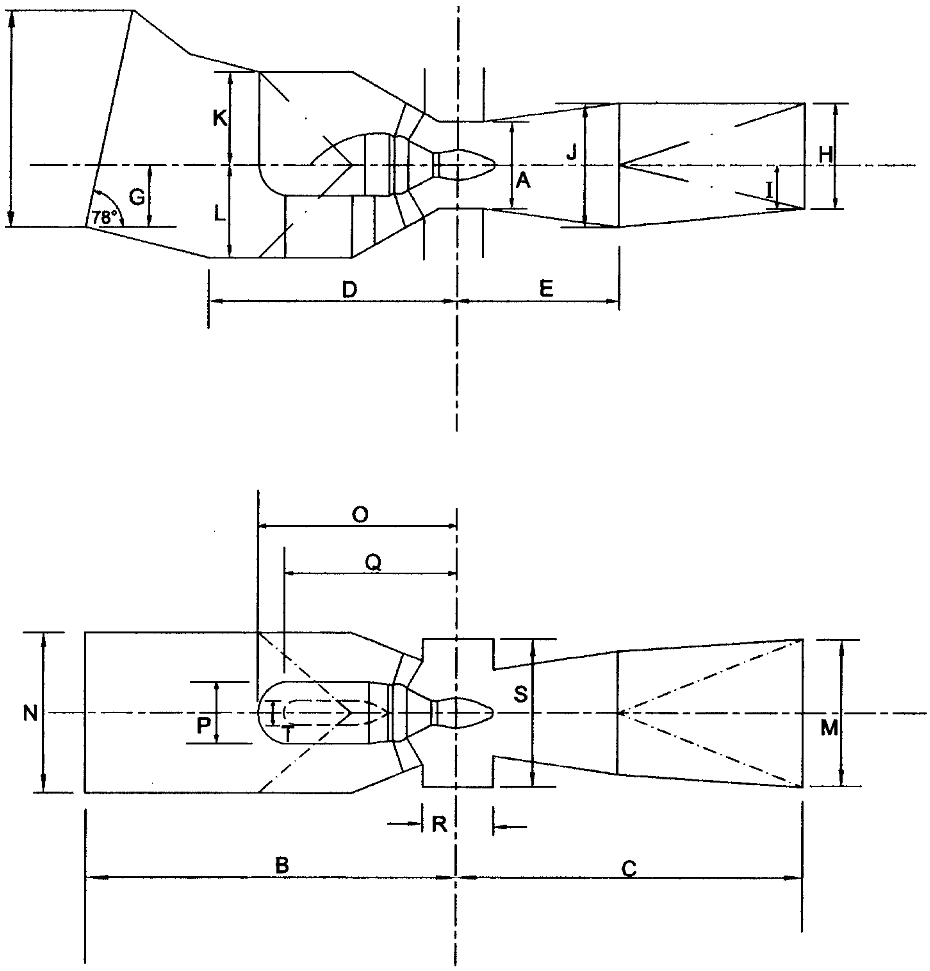
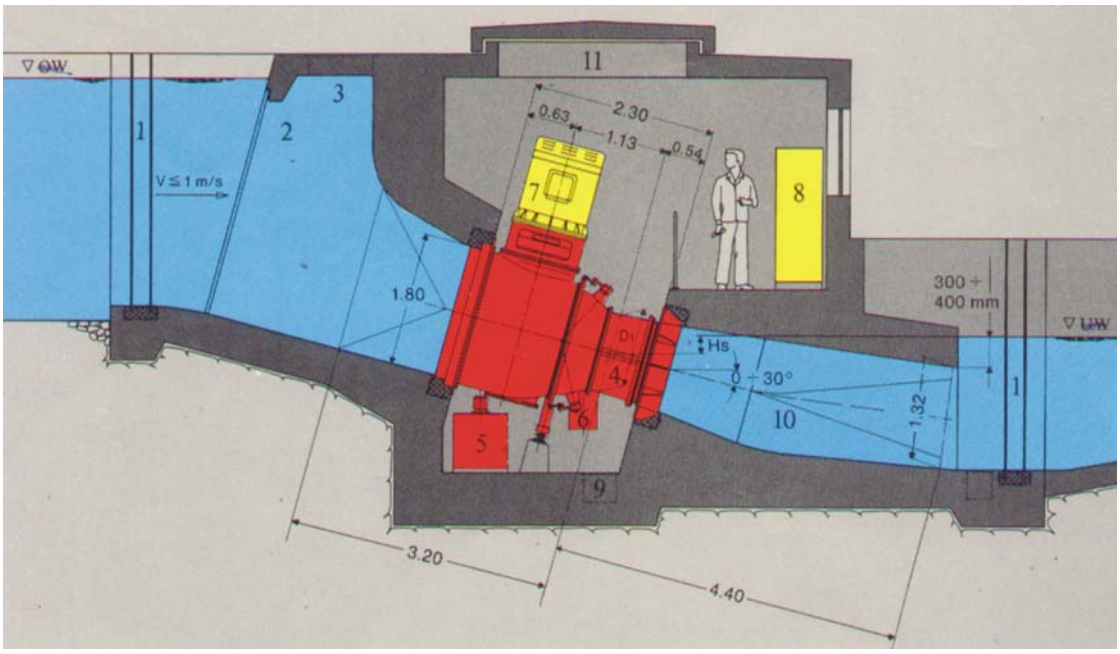


Figure 1-10 Hydraulic Design for Kaplan Turbine.

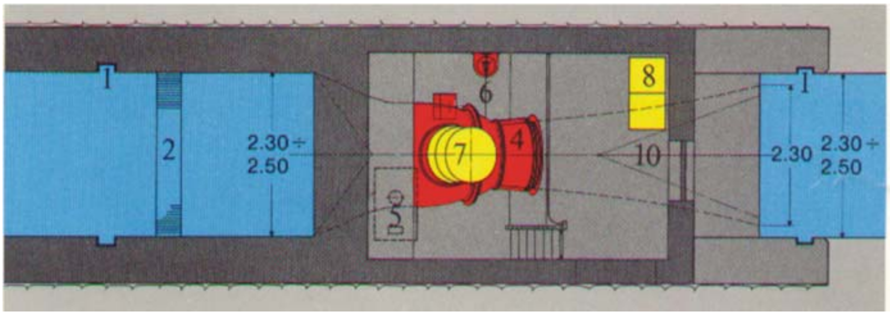


Alt	1	2	3	4		1	2	3	4
A	5600	5200	4800	4600	L	6200	5750	5300	5100
B	19300	19000	18800	18700	M	10400	9600	8900	8500
C	25200	23400	21600	20700	N	10400	9600	8900	8500
D	-	-	-	-	O	11400	11300	11300	11300
E	11780	10900	10100	9670	P	4200	4000	4000	4000
F	15300	14200	13100	12600	Q	10900	10900	10900	10800
G	6200	5750	5300	5100	R	5000	4550	4300	4100
H	6700	8100	7400	7100	S	10400	9500	8900	8500
I	4000	3720	3430	3250	T	1600	1600	1600	1600
J	8000	7430	6360	6570	U	150.5	1510	151.5	152.0
K	6200	9750	5300	5100					

Figure 1-11 Hydraulic Design for Pit type Turbine.



Installation of a Bevel Gear Bulb Turbine with Vertical Generator…



1. Stoplogs
2. Intake screen
3. Access
4. Turbine
5. Oil supplies
6. Closing weight
7. Generator
8. Turbine governor and switchgear
9. Drainage sump
10. Draft tube
11. Access opening

All dimensions are referred to $D_1 = 1$.

Dimensions given are not binding.

…or with Horizontal Generator

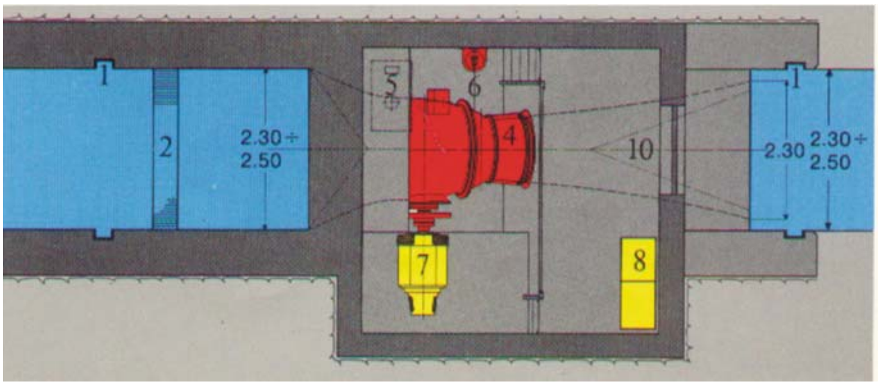
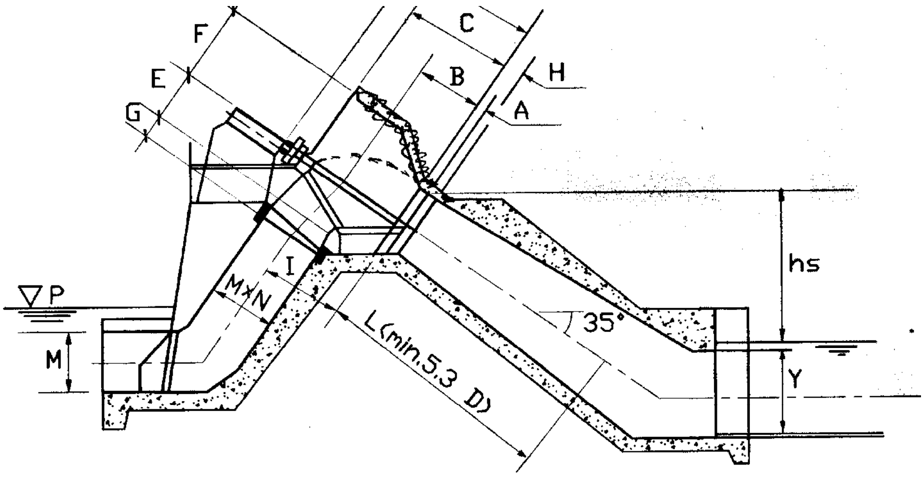
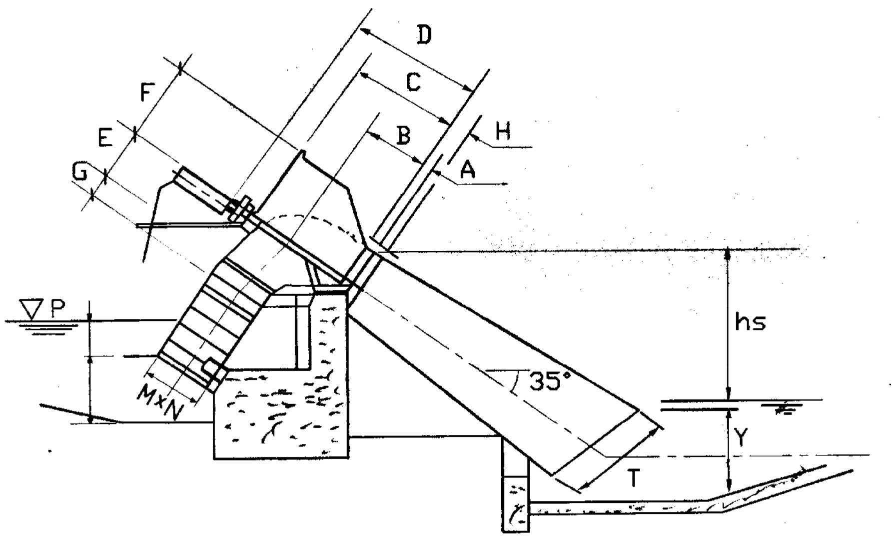


Figure 1-12 Hydraulic Design for Bevel gear type Turbine.





Type of runner hub	405		500		640		Type of runner hub	405		500		640	
runner	**1180**	**1320**	**1500**	**1700**	**1900**	**2120**	**runner**	**1180**	**1320**	**1500**	**1700**	**1900**	**2120**
A	249	278	316	358	400	446	K	400	400	445	445	540	540
B	1981	2216	2518	2854	3190	3559	L (mini)	6254	6996	7950	9010	10070	11236
C	3159	3554	4016	4552	5087	5676	M	1770	1980	2250	2550	2850	3180
D	4850	5220	6000	6530	7410	8210	N	2403	2688	3056	3462	3869	4317
E	1415	1585	1800	2404	2280	2545	P	To be defined in each case					
F	885	990	1125	1275	1425	1590	T	2189	2449	2783	3154	3525	3934
G	-	-	-	1300	1450	1620	U	956	1070	1216	1378	1540	1718
H	-	-	-	500	550	600	X	2155	2410	2740	3100	3470	3870
I	2718	3040	3455	3915	4375	4882	Y	2230	2500	2835	3215	3590	4010
J	483	540	614	696	778	868	Above dimensions (in mm) are approximate						

Figure 1-13 Hydraulic Design for Mini Hydropower Plants.

1.2.3 STATIC DESIGN OF POWERHOUSE

1.2.3.1 GENERAL

The general configuration of the low-head powerhouse is mainly influenced by the soil-mechanical and hydro-geological conditions existing in the project area. Usually low-head powerhouse structures are founded on deep deposits of silty fine river sand of the Indus Plain with uniform grain size feature. The stability analysis has to be carried out for pure static requirement as well as for hydro-geological requirements.

The stability analysis regarding pure static requirements comprise:

- Stability against sliding.
- Stability against soil rupture.
- Determination of point of the resultant of acting force on the base including the eccentricity and soil stresses.
- Stability against uplift.

The stability analysis regarding hydro-geological aspects considers that the initial boundary conditions may change with time due to the hydro-mechanical processes affecting the stability of the structure. These may include:

- Safety against suffusion,
- Internal erosion and
- Piping

1.2.3.2 STATIC REQUIREMENT

The stability analysis regarding pure static requirements comprise:

- Stability against sliding.
- Stability against soil rupture.
- Determination of point of the resultant of acting force on the base including the eccentricity and soil stresses.
- Stability against uplift.

1.2.3.2.1 STATIC INVESTIGATION

The following main conditions have to be analysed:

- Horizontal Sliding

The condition against sliding requires that the horizontal component of the acting forces must be less than the horizontal resisting friction forces.

- Position of Resultant Force

The condition concerning position of the resultant force is that it must be in the middle half of the base for earthquake and in the middle third for all other loading conditions. This work also includes the computation of the maximum and minimum stress under the foundation, with which the distribution of the soil reactions can be established.

- Floatation

For the floatation condition, the total vertical component of the downward resultant force must be larger than the uplift forces.

- Safety Against Soil Rupture

The soil rupture condition requires that the ultimate stress must be higher than the available stress under the foundation, with a safety factor of

⇒ 1.2 for earthquake
⇒ 1.5 for all others loading conditions

1.2.3.2.2 DETERMINATION OF STATIC LOADS

The acting forces shown in the free body diagram (Figure: 1-14) will be calculated. The corresponding formulae are presented in the others sections.

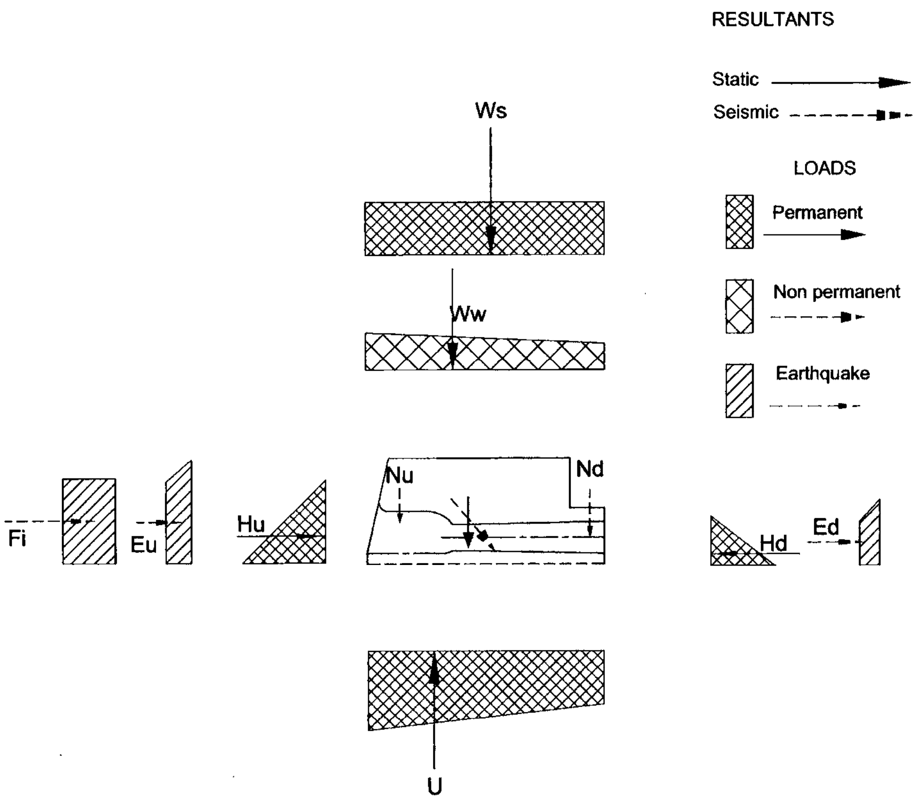


Figure 1-14 Loading Diagram for stability analysis

1.2.3.2.2.1 SELF WEIGHT

The weight of the concrete structure will be calculated only. The other weight such as weight of electro-mechanical equipment and removable parts should not be considered. These loads act vertically downward through the centre of gravity of the powerhouse structure.

The computation of the weight of a unit block of concrete may be carried out by dividing the structure into geometrical bodies i.e. parallelepipeds, truncated cones, truncated pyramids, cylinders, etc. whose centres of gravity can be directly determined. The total weight of the unit block may be determined as follows:

- Calculate the weight by considering whole structure as one geometrical body assuming that there is no cavity or open space.
- Positive value may be assigned to the geometrical bodies filled with concrete.
- Water filled cavities may also be assigned a positive value
- Negative value may be assigned to the cavities.
- Unit weight of concrete is 24 kN/m^3 and water 10 kN/m^3.
- The sum of all positive and negative partial weights gives the total weight of the structure.

This procedure may enable an easy control and computation of different loading combinations, as in the case of water filled or empty cavities (draft tubes, intake, etc.).

The weight of concrete and water and their centres of gravity may be calculated separately. All other loads, the factors of safety against sliding, floatation and position of the resultant and stresses may then be calculated by computer.

1.2.3.2.2.2 UPLIFT

This may be calculated assuming that the cut-off at upstream end of powerhouse structure is inoperative (as maximum adverse case) and a uniform hydraulic gradient exists across the structure. The uplift pressure in meters of water at any point under the foundation should be taken as the difference in water level between the piezometric line and the foundation. The uplift force is determined from the area of the trapezoid such as:

$$A = (a + b) / 2 \times L$$
$$U = A \times B \times Y_W$$

where

L = Length of powerhouse in flow direction
B = Width of unit bay (one unit or two units as a static unit)

It is acting at a point which exist from A at

$$Y = l/3\ (a + 2b) / (a + b)$$

where

a = hydrostatic pressure at A
b = hydrostatic pressure at B

1.2.3.2.2.3 WATER LOADS UPSTREAM

The upstream water pushes the powerhouse toward downstream with a load that assumes a triangular distribution. This triangulation will form in such a way that load is zero at the top of the water surface and become maximum at the powerhouse foundation level and is equal to the depth of water in front of the powerhouse structure. This may be calculated by using this formula:

$$Hu = (Y_w * h^2) \times L/2$$

where

Y_W = unit weight of water (10 kN/m^3).
h = height of water above the powerhouse foundation
L = width of unit bay or block

1.2.3.2.2.4 WATER LOADS DOWNSTREAM

These loads push the powerhouse structure toward upstream and will be calculated in the same way as the upstream water loads.

1.2.3.2.2.5 WATER LOADS WITHIN THE INTAKE AND DRAFT TUBE

The weight of water contained in the intake and draft tube of the powerhouse will act vertically downward and become as self weight of the structure. These will act through the centre of gravity of the waterway.

1.2.3.2.3 DETERMINATION OF DYNAMIC LOADS

1.2.3.2.3.1 SELF WEIGHT

These may be calculated as the product of the design earthquake acceleration and the self weight of the structure and the water. These act horizontally.

1.2.3.2.3.2 WATER LOADS UPSTREAM

The water upstream of the powerhouse structure will generate loads which will always act in the same direction as the earthquake inertia load. These may be calculated with the help of Westergaard's formula such as:

$$Eu = 0.583 \times wh^2 \times a/g$$

where

h = height of water above the base of the foundation
a/g = earthquake coefficient

Its acting point is 0.4h from the base.

1.2.3.2.3.3 WATER LOADS DOWNSTREAM

The water downstream of the powerhouse will generate an earthquake suction load in the same way as upstream. These will always act in the same direction as the earthquake load and calculated in the same way as upstream water loads.

1.2.3.2.4 LOADING CONDITIONS

The following are the important loading conditions which have to be investigated.

Construction Condition
The water on upstream and downstream sides is not in contact with the structure during the construction time.

Normal Condition
The intake and draft tube are running and water levels may be maximum or maximum upstream and minimum on the downstream side.

Earthquake Condition
The increments in dead and live loads have been applied on normal loading case with the assumption that the earthquake occurs during normal operation of Powerhouse. An earthquake acceleration factor for the project area may be used.

Above normal condition
The powerhouse is in operation. The change in water levels occurs because of wind and load rejection.

Repair Condition
It is assumed that the upstream and downstream stoplogs have been lowered and draft tube is dewatered and being repaired.

1.2.3.2.5 CALCULATION PROCEDURE

For all safety conditions the calculations were made as here under:

1.2.3.2.5.1 HORIZONTAL SLIDING

The condition against sliding requires that the horizontal component of the acting forces must be less than the horizontal resisting friction forces.

$$\eta_a = \frac{\tan\theta \times (\sum V - U) + A \times C}{\sum H} < \eta_r$$

where

η_r	=	required factor of safety
$\sum H$	=	sum of horizontal components of acting forces
$\sum V$	=	sum of all vertical forces
U	=	uplift force
f	=	frictional coefficient
η_a	=	available factor of safety
η_r	=	1.3 for earthquake
η_r	=	1.5 for other conditions
$\varnothing$	=	? for normal conditions
$\varnothing$	=	? for earthquake condition
A	=	area of base
C	=	Cohesion (c = 0, or ?)

1.2.3.2.5.2 POSITION OF RESULTANT FORCE

The condition concerning position of the resultant force is that it must be in the middle half of the base for earthquake and in the middle third for all other loading conditions. This work also includes the computation of the maximum and minimum stress under the foundation. With these minimum and maximum stresses under the foundation, the distribution of the soil reactions can be established.

Hence the criteria used is that the eccentricity of the resultant should not exceed one sixth of the base width in the normal case and one quarter in all other cases.

- Determination of position of the acting force

$$X = \frac{\sum M}{\sum V}$$

where

$\sum M$	=	sum of moments of all acting forces about A (kN-m)
$\sum V$	=	sum of vertical components of all acting forces (kN)

- Determination of the eccentricity

$$e = x - B/2$$

where

e	=	eccentricity
B	=	width of structure

- Determination of maximum/minimum stress under the foundation

$$\varphi_{\substack{min \\ max}} = \frac{N}{A} + \frac{M}{W_X}$$

Replacing the above variables in the above general formula with:

N	=	$\sum V$
A	=	B x L
M	=	e x V (moment with respect to the centre of the base)
W_X	=	BL/6 (internal moment relating to the centre of the base)

$$\varphi \; \begin{matrix} min= \\ max \end{matrix} \; \frac{V}{B \times L} \; [L \pm 6 \times e]$$

1.2.3.2.5.3 FLOATATION

For the floatation condition, the total vertical component of the downward resultant force must be larger than the uplift forces.

Required safety factor against floatation = 1.10

$$\eta_a = \frac{\text{Sum of vertical Components of the Forces}}{\text{Uplift}}$$

1.2.3.2.5.4 SAFETY AGAINST SOIL RUPTURE

The soil rupture condition requires that the ultimate stress must be higher than the available stress under the foundation, with a safety factor of

- 1.2 for earthquake
- 1.5 for all others loading conditions

The safety factor against soil-rupture can be calculated by this relationship:

$$av = \frac{V_b}{\sum V}$$

Where

V_b = is the theoretical ultimate load, when soil rupture occurs. The rupture occurs when the ultimate stress value of the soil is achieved.

$\sum V$ = sum of vertical forces acting on the structure (including the uplift).

The ultimate load for soil rupture can be computed with the following formula recommended by German Standard DIN-4017

$$V_b = A' \times \varphi_f$$

where

A' = area to be calculated
A' = a' x b'
a' = a - 2 ey
b' = b - 2 ex

For the unit block

A' = a' x b'
a' = 1 m
b' = B - 2 x e

$$\varphi_f = \underbrace{C \,.\, Nc \,.\, Xc \,.\, Vc}_{I} + \underbrace{\gamma 1 \,.\, d \,.\, Nd \,.\, Xd \,.\, V d}_{II} + \underbrace{\gamma 2 \,.\, b \,.\, Nb \,.\, Xb \,.\, Vb}_{III}$$

where

Ist Term gives the influence of cohesion
IInd Term gives the influence of foundation depth

IIIrd Term gives the influence of foundation width

where

φ_f	=	average ultimate stress in the soil
C	=	Cohesion
d	=	Depth of the foundation surface
N_c	=	Coefficient of influence from Cohesion
N_d	=	Coefficient of influence done by foundation depth
N_b	=	Coefficient of influence done by foundation width
		(Other details: see German Standard DIN 4017-T.1)

1.2.3.2.6 SAFETY FACTOR

The following safety factors are used as base. The calculated safety factor must be more than as given below.

Sr. No.	Loading Condition	Sliding	Floatation
1.	Construction	1.5	
2.	Normal	1.5	1.5
3.	Above Normal	1.5	1.1
4.	Earthquake	1.3	1.1
5.	Repair	1.5	1.5

1.2.3.2.7 COMPUTER PROGRAM FOR STATIC STABILITY ANALYSIS

The program has been developed by GTZ for the stability analysis of the low-head powerhouse using the above-described method, allowing the possibility to consider different loading conditions. The program has been written in FORTRAN-IV programming language and runs on any IBM-compatible personal computer on DOS (Disk Operating System).

The main objective of the program is to provide a computational efficient and easy to use tool for the computation of stability of powerhouse using internationally accepted engineering practice. The details about the used this programme can be found in "**Guide Lines Powerhouse**" for Powerhouse Stability Analysis Programme.

1.2.3.2.8 EXAMPLE-STATIC STABILITY ANALYSIS OF GUDDU POWERHOUSE DATA

Fig. 1-14 shows a cross-section of the powerhouse with normal and earthquake loads. The loads have been described by the abbreviations marked on Figure.

a) Water level combinations

For normal, earthquake, repair and construction conditions upstream water level 77.90 m (designed water level of the barrage) and downstream water level 72.50 m have been used because most of the time except the months of June to September these levels have been observed, against a discharge of 1055 m^3/sec.

The sudden blackout and wind will increase the water levels on upstream side therefore above normal loading case with upstream water level 79.0 and downstream water level 72.50 has been adopted. To check the stability against floatation the upstream water level 77.85 and downstream water level 78.00 have been used.

b) The following loading conditions have been used.

- **Construction Condition**

The water on upstream and downstream sides is not in contact with the structure during the construction time.

- **Normal Condition**

The draft tube is running.

- **Earthquake Condition**

The increments in dead and live loads have been applied on normal loading case with the assumption that the earthquake occurs during normal operation of Powerhouse. An earthquake acceleration factor of 0.1 g has been used.

- **Above normal condition**

The powerhouse is in operation. The change in water levels occurs because of wind and load rejection.

- **Repair Condition**

It is assumed that the upstream and downstream stoplogs have been lowered and one draft tube is under repair.

CALCULATION OF FORCES AND STABILITY REQUIREMENTS

For the above mentioned loading conditions the analysis has been made to investigate the tendencies of instability by using a computer program “ POWERHOUSE” written by GTZ.

The following results were achieved.

- **Horizontal Sliding**

For each loading condition factor of safety for stability against horizontal sliding has been determined. The minimum factor of safety achieved is 1.79 from earthquake condition. The powerhouse is stable against sliding with ample factor of safety.

- **Overstressing**

The maximum stress obtained from the analysis is 30.13 t/m^2 in earthquake loading condition. Considering flat foundation and surcharge load the stresses are not so high.

- **Floatation**

For above normal condition where maximum uplift exists, the floating factor obtained shows that powerhouse is stable against floatation.

- **Overturning**

The resultant of forces in each case is within middle third of base width of powerhouse foundation. This indicates that structure is safe against overturning.

1.2.3.3 HYDRO-GEOLOGICAL REQUIREMENT

1.2.3.3.1 GENERAL

Besides the static requirement for safety, the seepage below a structure on deep beds of sand, is the most endangering factor where a complete cut-off cannot be realised. Internal erosion may occur as consequence of suffusion followed by an increase of K-value i.e. seepage velocity. The internal erosion means the displacement and removal of bigger particles (grains) of the soil texture. With the removal of fines from downstream site of the structure the exit gradient of seepage flow may increase which leads to piping. If the exit gradient is higher than critical, then the soil particles are washed away creating pipes advancing from downstream side to upstream site. The water flows directly through these pipes to downstream side washing away the material under the structure. As a consequence the collapse of the structure occurs. It is essential to safeguard the structure against failure by piping and uplift pressure. Since the last century numerous investigations of existing weirs, especially failure of weirs, have brought great experience for such structures. The experience in Pakistan and Egypt with weirs on fine sand gives a realistic basis to determine the values to use in the design concerning the hydraulic gradient. This is a decisive parameter for dimensioning of a structure on sandy soils with differential water level on upstream and downstream sides.

It is stated that the most unfavourable condition of subsoil is a fine or silty-fine sand. There are formulae based on more than 100 years of experimental data for establishing safety against piping. The most important approaches are:

- Bleigh's Theory
- Lane's weighted Creep Theory
- Flownet

1.2.3.3.2 BLEIGH'S THEORY

Bleigh assumed for the analysis of the problem that the percolation water follows a path along the underface of the structure resting on permeable foundation. This may include a cut-off in the form of a sheet pile or a grout curtain wall. He considered that

"The line of creep will be forced to flow around these obstructions and may not as might be imagined take the line of least resistance. The line of creep may be measured down one side of the vertical obstruction and up the other side."

The added length of creep will thus be twice the depth of the curtain wall. Bleigh also recommended that the ratio of total length of creep (L), and the head (H) for a given sub-soil grade should not be less than a certain value for the safety of the work, such as:

$$L = C_B * H$$

where

C_B = Bleigh's Coefficient
= Percolation Coefficient

According to Bleigh the percolation coefficient for different soils should be less than the values given below in Table:1 - 2.

Table 1-2 Bleigh's Coefficient

Sr. No.	Soil Classification	i	C_B
1	Silty Sand	0.056	18
2	Fine Sand	0.067	15
3	Coarse Sand	0.083	12
4	Gravel's	0.110	9
5	Boulder with Gravel and Sand		4 to 6

Where

i = Hydraulic Gradient = H/L

The length of creep according to Bleigh is as per Figure below:

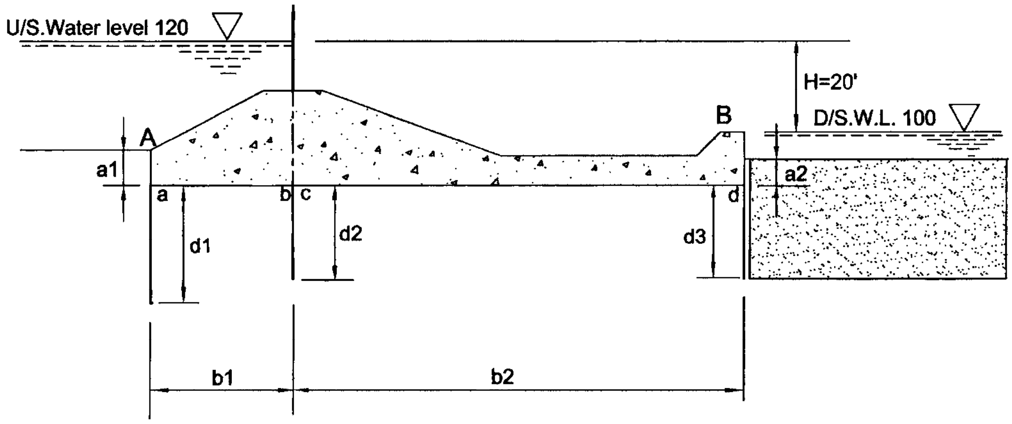


Figure 1-15 Hydraulic Structure with Three Sheet Piles

$$L_B = a_1 + 2d_1 + b_1 + 2d_2 + b_2 + 2d_3 + a_2$$
$$L_B = a_1 + b_1 + b_2 + a_2 + 2\,(d_1 + 2d_2 + 2d_3)$$
$$L_B = \sum a + \sum b + 2 \sum d$$

Bleigh assumed that the loss of head per unit length of percolation for both horizontally or vertically up or down the path along the creep line is the same and can be calculated by drawing the creep cumulatively as abscissa and head as ordinate such as shown in the figure below:

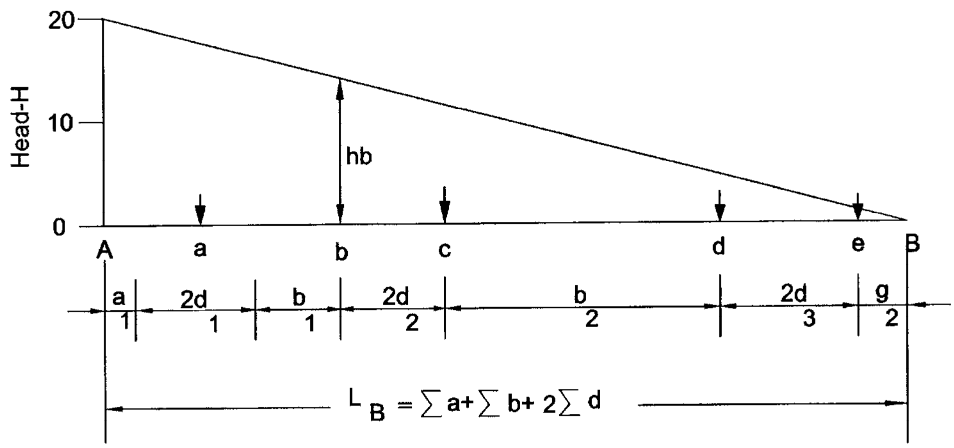


Figure 1-16 Pressure Head by Bleigh's Theory

The pressure head h at any point b is given by

$$h_b / H = (2d_2 + b_2 + 2d_3 + a_2)/ L_B$$
$$h_b = (H/ L_B)\,(2d_2 + b_2 + 2d_3 + a_2)$$

Similarly at c, d

$$h_c = (H/ L_B)\,(b_2 + 2d_3 + a_2)$$
$$h_d = (H/ L_B)\,(2d_3 + a_2)$$

1.2.3.3.3 LANE'S WEIGHTED CREEP THEORY

Lane evolved weighted creep theory, which in fact is a modification of Bleigh's theory, only the vertical creep is given weightage of 3 to 1 over the horizontal creep ratio 1:1, as recommended by Bleigh. The weighted creep relation can be written with reference to Figure: 1-15 as.

$$L_W = 2\sum d + (1/3)\sum b$$

Where

$2\sum d$ = The sum of all the creep lengths along both faces of the vertical cut-off and sloping contacts at an angle greater than 45^0.

$\sum b$ = Sum of all horizontal and sloping contacts of less than 45^0.

For the safety of structure according to Lane.

$$L_W = C_W * H$$

where

C_W = Weighted Creep coefficient depending upon the soil in the foundation

The values of Lane's weighted creep coefficient are given in Table: 1 - 3.

Table 1-3 Lane's weighted Creep Coefficient

Sr. No.	Soil Classification	C_W
1	Silty Sand	8.5
2	Fine Sand	7.0
3	Medium Sand	6.0
4	Coarse Sand	5.0
5	Fine Gravels	4.0
6	Medium Gravels	3.5
7	Coarse Gravel Including Pebbles	3.0
8	Boulders with some cobbles and Gravels	2.5

To determine the uplift pressure under a hydraulic structure according to Lane's weighted creep theory, the abscissa will be plotted according to the weighted creep, such that in the summation one third the length of each horizontal creep plus lengths along either face of a cut-off are plotted and the head as ordinate (Figure: 1-17). The hypotenuse of the triangle will give the hydraulic gradient and pressure head at any point under the structure.

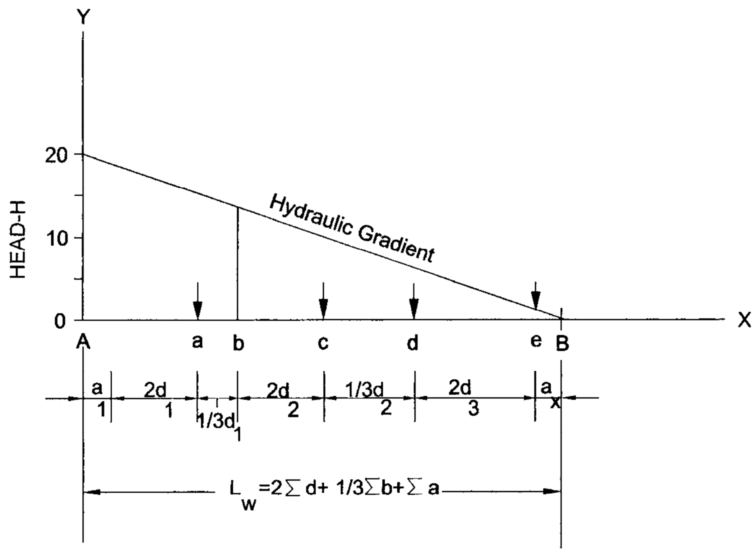


Figure 1-17 Hydraulic Gradient According to Lane's Weighted Creep Theory.

1.2.3.3.4 FLOWNET

Flow net is a net work (Figure: 1-18) of flow lines or stream lines and equipotential lines intersecting each other orthogonally and is used in seepage analysis to calculate:

- The quantity of seepage water.
- The seepage pressure.

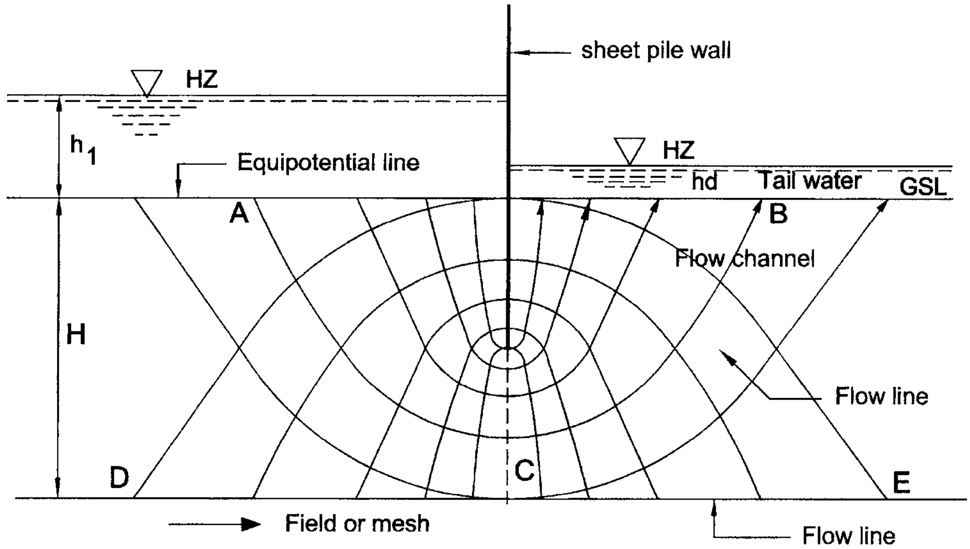


Figure 1-18 Flow net for a sheet pile wall.

Flow line is a path which water particle follows during seepage through the material. Flow lines are also called stream lines and can be of infinite number. Equipotential line is of equal head or pressure and can be of infinite number in a flow net. These lines intersect flow lines at right angle.

- **Seepage Quantity**

From Figure: 1-19 consider the flow field ABCD with a unit depth perpendicular to the paper. Let

N_f = number of flow channels in a flow net.
N_d = number of equipotential drops in a flow channels.

Head loss from AB to BC = dh

where
$dh = h_t / N_d$

From Darcy's law

$d_q = KiA = KA\, dh / dh = K (h_t\ N_d\ / b)\, a$

Therefore total discharge

$q = (d_q)(N_f) = K h_t (a/b) (N_f / N_d)$

For a square net work a = b then a/b=1

Therefore discharge per unit length is

$$q = K h_t (N_f / N_d)$$

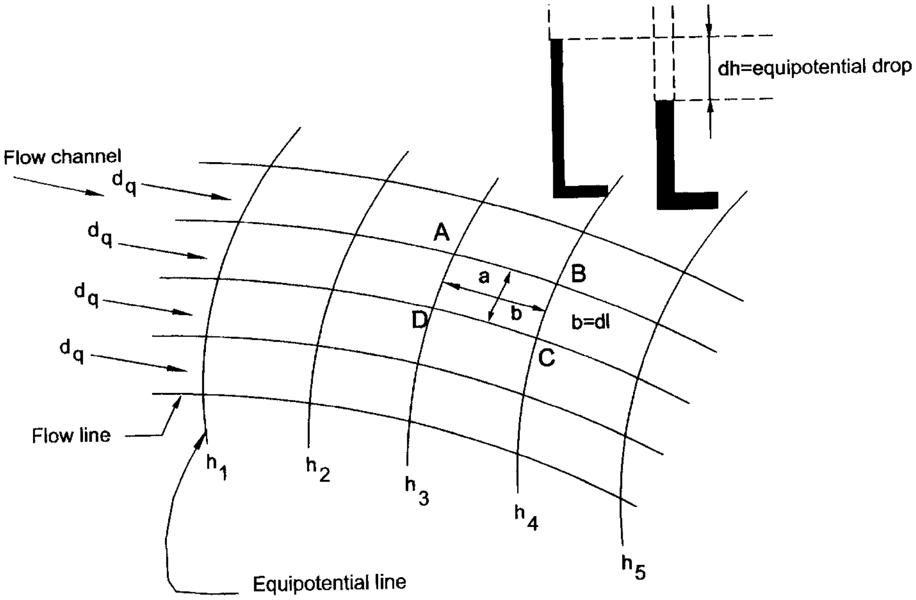


Figure 1-19 Flow net definitions

- **Seepage Pressure**

Total seepage force on AD Figure: 1-17 when a = b for a square field

$$a^2 \gamma_w h_4$$

Total seepage force on BC

$$a^2 \gamma_w h_5$$

The differential force acting on the field ABCD

$$a^2 \gamma_w (h_4 - h_5) = a^2 \gamma_w dh$$
$$a^3 \ (dh/a) \gamma_w = a^3 \ i \gamma_w$$

where

dh/a = i = hydraulic gradient
a^3 = the volume of the soil element ABCD

When the water flows downward, the seepage pressure cause an increase in inter granular pressure. When water flows upwards however, the inter-granular pressure is reduced which may cause unstable conditions to the structure. The seepage force on upstream of a dam increases stability while at the downstream it generates unstable conditions leading to heave, boiling or piping.

1.2.3.3.5 EXAMPLE-STABILITY ANALYSIS AGAINST PIPING OF GUDDU POWERHOUSE

The soil strata 20 m before the upstream apron, below the upstream apron, powerhouse structure, downstream apron and 20 m beyond the downstream up to 14.50 m level has been divided into 436 small elements as shown in Fig. 1-20.

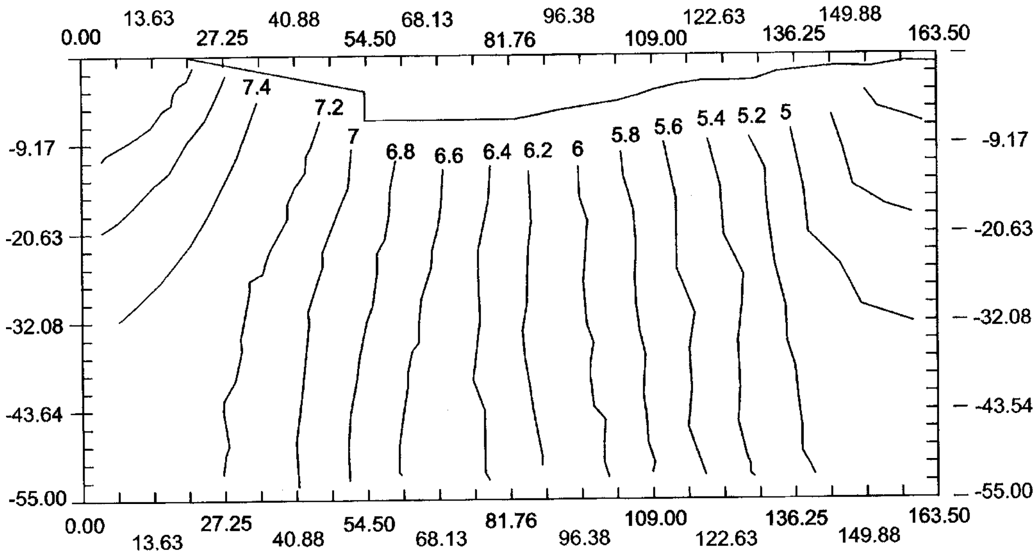


Figure 1-20 Flownet under the Foundation of Guddu Powerhouse.

The total number of nodes is 245. The input data used is the number of elements, number of nodes, the model information, the x & y co-ordinates of nodes, the water head at upstream nodes before powerhouse structure, and the permeability of subsoil in x and y direction. Because the soil is homogenous in this case so the permeability in x-direction is the same as in y-direction. The output results are

- Static pressure at every node
- Flow of water at every node
- Velocity of flow in x-direction for every element
- Velocity of flow in y-direction for every element.

Results and Discussion

The output of the computer program based on flow networks developed by GTZ(was used) provides pressure head and discharge at each node and seepage water velocity through each element. Equipotential lines below powerhouse structure has been drawn and shown in Figure above. The results show that the pressure head decreases with increase in seepage path. The water entered to the subsoil from upstream side. (node Nos.1,23 and 38) is 0.05 l/s approximately, which is equal to the water coming out of the soil at downstream side of the structure (node 215, 230 and 245). This indicates that groundwater level is already above the foundation level of the powerhouse and there is no water charging in the ground reservoir.

The horizontal and vertical velocities of water at the exit (downstream side of the downstream apron) are 0.005 and - 0.003 cm/s. The negative sign of the vertical velocity indicates the water is coming out of the soil. From these velocities of water the exit gradient has been calculated by adopting K value of 0.001 m/s. The exit gradient is 1/17, which is safe considering the washing out of the fine particles of the soil. Because for fine sands the maximum limiting exit grout is 1/7. The results indicate that the structure is safe against pinging.

1.2.4 POWERHOUSE STRUCTURAL DESIGN

1.2.4.1 GENERAL

The structural analysis of the powerhouse has to be done considering two main parts such as

- Superstructure
- Substructure

The substructure is the part of the powerhouse which is beneath the entrance flume, spiral case and draft tube. It β may be based on foundation slab. The parts of the superstructure are the machine hall, generator floor, service bay, etc. Therefore structural design should be done for

- Substructure as one structure.
- Frame of the draft tube.
- Frame of the entrance flume.
- Structural design of spiral case.
- Generator floor.
- Drainage shaft, Ventilation shaft, elevator shaft, second storey floor, etc.
- Machine hall.

The powerhouse may have foundation such as

1. Caisson
2. Pile
3. Mat

The mat foundation is the type, which is most commonly used. The pile foundation is rarely used and caisson occasionally. The design of the substructure is considered such as

- Elastic structure on elastic foundation.
- Rigid structure on elastic foundation.

These structures should be designed for temperature and shrinkage also.

1.2.4.2 MACHINE HALL

Machine hall is usually a rectangular building whose function is to provide the protection from weather to the equipment installed and personnel who staff it during erection and maintenance. It is therefore designed around the plant and its leading dimension should not be fixed until the physical size and spacing of the generating sets, ancillary equipment and arrangement of cable have been finalised.

The length, width and height are mainly determined by the overall dimension and arrangement of the turbine and generator. The length of the building is dependent on the number of units together with their physical dimensions and design. The width of the machine hall depends upon the physical dimension and arrangement of the turbo-generator set and space required for transport of equipment during erection and maintenance, especially in low-head plants.

The height of the machine hall upto the roof is determined by the height of the crane rail and depth of the crane bridge, which depends upon the capacity and height to which the crane hook must rise. The rise totally depends upon the part of maximum dimension that has to be lifted. In case of turbines with vertical axis the limiting items may be the turbine runner along with shaft and top cover. In horizontal setting turbine runner is the controlling dimension. However, in bulb unit the size of the generator rotor may be the decisive one. Therefore, the machine hall in case of machines with horizontal axis is less lofty than in case of machine with vertical axis.

The floor levels are generally determined in relation to the access road, flood level and turbine runner setting and arrangement.

The machine hall will be enclosed either by load-bearing walls or by curtain walls filling the opening between the reinforced concrete columns. The roof may be of wood or steel truss or reinforced concrete structure interacting with the walls/ column as a frame. In Pakistan reinforced concrete walls, column, beam and slab is the preferred option due to less cost and high temperature. Therefore these will be designed as frame structure. The columns are supporting bracket for crane rail.

The acting loads may include:

- Dead weight
- Live load
- Wind load
- Snow load
- Crane load, etc.

The other loads can be calculated on the basis of standard method. However, crane load must include the weight of the crane bridge, the trolley and the dynamic effect of the braking in addition to the maximum carrying capacity of the crane. The manufacturer will supply the weight of the crane bridge and the trolley and the maximum capacity of the crane hook can also be determined from the weight of the heaviest part to be lifted. Braking load will be calculated as given below and will be 2 to 10 % of the weight of the moving mass. The braking the of crane bridge will apply horizontal longitudinal force to the frame at the level of the rail and braking of trolley will transmit a force to the frame which act perpendicular to the rail in flow direction. The braking load may be

$$P = ma = G\, v/g\, t \qquad \text{(tons)}$$

Where

P	=	Braking load, tons	
v	=	Velocity of the crane bridge or trolley,	m/sec
G	=	Weight of the moving mass, tons	
g	=	Acceleration due to gravity, m/s^2	
t	=	Braking time, sec	

1.2.4.3 DRAFT TUBE

The draft tube may be divided into two parts such as:

- Upper part near the throat. The splitter pier do not extend upto this part and therefore, foundation slab, roof slab and separation wall may become a frame.
- The part where splitter pier exists. In this part foundation slab, the separation wall, the splitter pier and roof of the draft tube form a multi-bay framed structures at right angle to the direction of flow. The foundation slab may be treated as a continuous beam if the ratio of the slab depth to the thickness of the vertical element is great.

These frames should be analysed for all type of loading conditions such as repair, sudden closure of turbine, uplift pressure at maximum, ground pressure, loads induced by the machine hall, etc. The loads acting in horizontal and vertical direction may also be treated separately.

1.3 POWER CANAL
It is the most important element of a power plant. The power canal may be constructed by excavating the bed or by filling the embankment or partly by filling and partly by excavation. The power canal in low-head plants are always open channels and may be

- Earthen canal
- Lined canal
  - ⇒ Brick lining
  - ⇒ Concrete lining
  - ⇒ Asphalt lining
  - ⇒ Stone pitching
  - ⇒ etc.

The design of the earthen canal will be made by using well known Lacey's formulae such as

Wetted perimeter	P	$= 2.67\ Q^{1/2}$
Slope	I	$= (f^{5/3}\ Q^{-1/6})\ /1788$
Depth	h	= 0.47 (Q/f)1/3
Regime velocity	V	$= \{ m/(f\ \nu^{2/3}\ g^{4/3}) \}^{1/4}\ gm\ i^{1/2}$

Where

m	=	Hydraulic radius
ν	=	Kinematic viscosity
f	=	silt factor
Q	=	Design discharge

The lined canal may be designed with well known Manning's or Strickler formula such as

$$V = (1.486 / n)\ m^{2/3}\ i^{1/2}$$

Where

m	=	Hydraulic radius
i	=	Bed slope
n	=	Manning's coefficient depending upon type of lining

For more details please refer to the course material on "Open Channel Hydraulics" prepared by GTZ for WAPDA Engineering Academy, Faisalabad.

1.4 RETAINING WALL
1.4.1 GENERAL
In low-head power plants retaining walls are provided on upstream and downstream of the powerhouse. These retaining walls are used to bound the intake and outlet slabs on both sides. These walls are also used to provide a smooth connection of the earthen embankment of headrace and tailrace with the powerhouse, which is concrete structure. Retaining walls are of so many type such as:

- Gravity retaining wall
- Cantilever walls
- Counterforts walls
- Buttresses
- Etc

In designing of retaining wall two type of designs are important to be carried out such as:

1. Stability Analysis
2. Structural Analysis

In stability analysis of a retaining wall lateral earth pressure is an important parameter. The two theories such as

- Coulomb's Earth Pressure Theory
- Rankine Earth Pressure Theory

developed in eighteenth and nineteenth centuries are still in use in their original form and in some modified form. We will discuss these theories in detail here.

1.4.2 COULOMB'S EARTH PRESSURE THEORY

The basic assumptions in Coulomb's theory are as follows:

- Soil is isotropic, homogeneous on both sides and below the retaining wall, has internal friction (ϕ) and may be cohesive (c' > 0) or non cohesive (c' = 0).

- The rupture surface is a plane surface and the backfill surface is planer (it may slope but is not irregular shaped, ß = constant).

- The friction resistance is distributed uniformly along the rupture surface and the soil to-soil friction coefficient is a function of tan (ϕ).

- The failure wedge is a rigid body undergoing translation

- There is wall friction, i.e. as the failure wedge moves with respect to the back-face of the wall a friction force is developed between soil and wall. This friction angle is usually termed as δ.

- Analysis is carried out on a 2-dimensional basis, which assumes an infinitely long retaining wall.

Notation adopted is as indicated in Figure below.

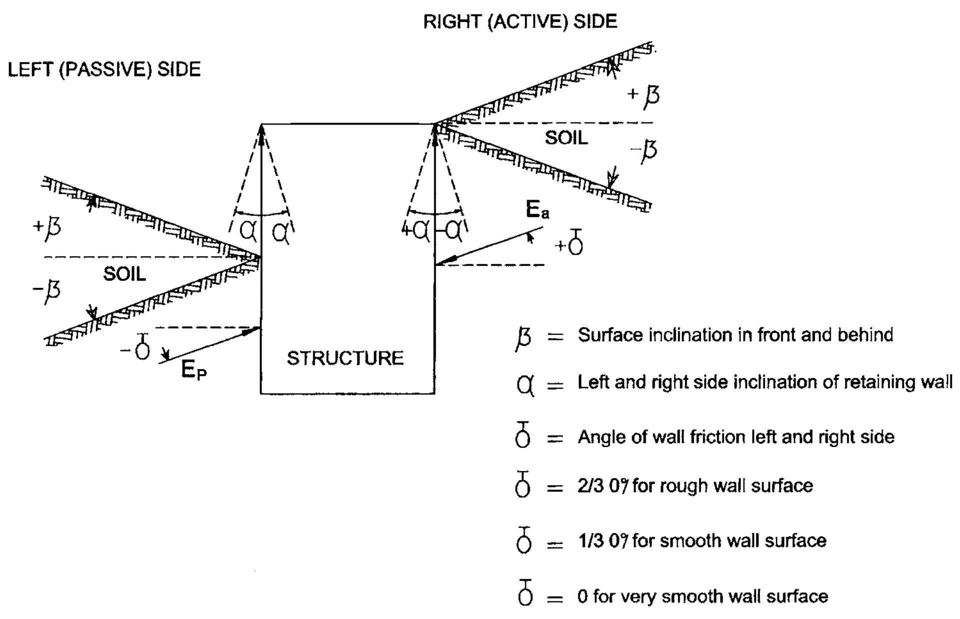


Figure 1-21 Notation Used in Retaining Wall Stability Analysis.

The procedures adopted follow the DIN norms, and this description closely follows the book "Grundbau Taschenbuch", Part 1, 4th edition, 1990. The basic equations used are described in the following paragraphs.

1.4.2.1 NORMAL CONDITIONS

Normal conditions in this case mean the loading conditions without lateral forces due to seismic loading. The individual components of the earth pressures are described in the following paragraphs.

1.4.2.1.1 ACTIVE EARTH PRESSURE COMPONENTS

1.4.2.1.1.1 EARTH PRESSURE

The active earth pressure for non-cohesive soil is determined as:

$$E_a = \gamma\left(\frac{h^2}{2}\right) x\ K_a$$

Where the coefficient K_a is determined as

$$K_a = \frac{\mathrm{Cos}^2(\phi+\alpha)}{\mathrm{Cos}^2\alpha * \left[1+\sqrt{\frac{\mathrm{Sin}(\phi+\delta)*\mathrm{Sin}(\phi-\beta)}{\mathrm{Cos}(\alpha-\delta)*\mathrm{Cos}(\alpha+\beta)}}\right]^2}\ x\ \frac{1}{\mathrm{Cos}(\alpha-\delta)}$$

The horizontal component E_{ah} of the active soil pressure is determined as:

$$E_{ah} = E_a * \mathrm{Cos}(\delta-\alpha) = \gamma\left(\frac{h^2}{2}\right) x\ K_{ah}$$

with

$$K_{ah} = \frac{\mathrm{Cos}^2(\phi+\alpha)}{\mathrm{Cos}^2\alpha * \left[1+\sqrt{\frac{\mathrm{Sin}(\phi+\delta)*\mathrm{Sin}(\phi-\beta)}{\mathrm{Cos}(\alpha-\delta)*\mathrm{Cos}(\alpha+\beta)}}\right]^2}$$

The vertical component E_{av} of the soil pressure is determined as:

$$E_{av} = E_{ah}\tan(\delta-\alpha) = E_a\,\mathrm{Sin}(\delta-\alpha)$$

1.4.2.1.1.2 VERTICAL LOADING

In case of a uniform vertical loading q the assumption is that there is no change in the plane of failure. Therefore, the corresponding equations are:

$$K_a(q) = \left(\frac{\mathrm{Cos}(\alpha).\mathrm{Cos}(\beta)}{\mathrm{Cos}(\alpha+\beta)}\right).K_a$$

$$E_a(q) = q.h.K_a(q)$$

and the horizontal component $E_{ah}(q)$

$$K_{ah}(q) = \left(\frac{Cos(\alpha).Cos(\beta)}{Cos(\alpha+\beta)}\right).K_{ah}$$

$$E_{ah}(q) = q.h.K_{ah}(q)$$

1.4.2.1.1.3 CONSIDERATION OF COHESION

Cohesion has no effect on the direction of the plane of failure as long as the surface and the earth pressure remain horizontal. With any wall inclination and without load applied on the ground surface, on basis of *Coulomb*'s theory:

$$E_a(c) = 2.c.h.K_{ah}(c)$$

where

$$K_{ah}(c) = \frac{Cos(\phi).Cos(\beta).Cos(\delta-\alpha).(1-\tan(\alpha).\tan(\beta))}{1+Sin(\phi+\delta-\alpha-\beta)}$$

1.4.2.1.2 PASSIVE EARTH PRESSURE COMPONENTS

In case of submerged soil due to groundwater, the effective soil pressure E'_a is used instead of E_a, which implies subtracting the water pressure. The equations used are the same, only that the soil density adopted corresponds to that of submerged soil.

1.4.2.1.2.1 EARTH PRESSURE

In case of passive soil pressure, which indicates the resistance of the soil to be displaced by the retaining wall the governing equations are similar to the active case.

$$E_p = \gamma\left(\frac{h^2}{2}\right) x\ K_p$$

$$K_p = \frac{Cos^2(\alpha-\phi)}{Cos^2\alpha * \left[1-\sqrt{\frac{Sin(\phi-\delta)*Sin(\phi+\beta)}{Cos(\alpha-\delta)*Cos(\alpha+\beta)}}\right]^2}\ x\ \frac{1}{Cos(\alpha-\delta)}$$

The coefficient K_p is determined as

The horizontal component E_{ph} of the active soil pressure is determined as:

$$E_{ph} = E_a * Cos(\delta-\alpha) = \gamma\left(\frac{h^2}{2}\right) x\ K_{ph}$$

with

$$K_{ph} = \frac{Cos^2(\phi+\alpha)}{Cos^2\alpha * \left[1+\sqrt{\frac{Sin(\phi+\delta)*Sin(\phi-\beta)}{Cos(\alpha-\delta)*Cos(\alpha+\beta)}}\right]^2}$$

The vertical component E_{pv} of the soil pressure is determined as:

$$E_{pv} = E_{ph}\tan(\delta-\alpha) = E_p\,Sin(\delta-\alpha)$$

1.4.2.1.2.2 VERTICAL LOADING

In case of a uniform vertical loading q the assumption is that there is no change in the plane of failure. Therefore, the corresponding equation is:

$$K_p(q) = \left(\frac{Cos(\alpha).Cos(\beta)}{Cos(\alpha+\beta)}\right).K_p$$

$$E_p(q) = q.h.K_p(q)$$

and the horizontal component $E_{ph}(q)$

$$K_{ph}(q) = \left(\frac{Cos(\alpha).Cos(\beta)}{Cos(\alpha+\beta)}\right).K_{ph}$$

$$E_{ph}(q) = q.h.K_{ph}(q)$$

1.4.2.1.2.3 CONSIDERATION OF COHESION

Similarly, for passive earth pressure:

$$E_p(c) = 2.c.h.K_{ph}(c)$$

Where

$$K_{ah}(c) = \frac{Cos(\phi).Cos(\beta).Cos(\delta-\alpha).(1-tan(\alpha).tan(\beta))}{1+Sin(\phi+\delta-\alpha-\beta)}$$

1.4.2.1.3 TOTAL HORIZONTAL FORCES

The total horizontal earth forces include earth pressure, horizontal component of vertical loads and cohesion. The vertical components of the earth pressure, which may create stabilising force and moments, are neglected.

1.4.2.1.3.1 ACTIVE FORCES

These are calculated as follows:

$$E_{ah} = \gamma.\frac{h^2}{2}.K_{ah} + q.h.K_{ah}(q) - 2.c.h.K_{ah}(c)$$

1.4.2.1.3.2 PASSIVE FORCES

These are calculated as follows:

$$E_{ph} = \gamma.\frac{h^2}{2}.K_{ph} + q.h.K_{ph}(q)6 + 2.c.h\ cdot.K_{ph}(c)$$

According to DIN 1054, the passive earth pressure should be taken into account in the stability analysis of retaining walls under following conditions:

- A displacement of the wall to make passive earth pressure effective does not represent any risk.
- The soil should be at least medium dense.
- In case of cohesive soils, these should be stiff.

When these conditions apply, DIN 1054 also recommends:

a) Include only up to 50% of total passive forces for loading case 1.
b) Neglect total passive forces for loading case 2 and 3.

The present version of the computer program includes above given recommendations.

1.4.2.2 EARTHQUAKE LOADING
The accepted practice to consider the effect of earthquake on active and passive soil pressure is in accordance to *Coulomb*'s theory. Basically this implies to assume additional forces due to earthquake action.

The vertical acceleration due to earthquake is neglected.

The calculations are made using Krey's method, which assumes that the slope of the reference planes of active and passive earth pressure as well as the slope of the surface of the terrain should be taken with reference to the new direction of the forces.

In case of both active and passive earth pressures, the system is rotated in accordance to the horizontal acceleration factor as follows:

$$\varepsilon_h = \left|\tan \Delta\alpha\right| = \left|\tan \Delta\beta\right|$$

1.4.2.2.1 ACTIVE EARTH PRESSURE
Figure 1-22 schematically shows the rotated system for the active earth pressure. For the active soil pressure considering earthquake loading, in accordance to the direction of angles adopted, the fictitious displacement lead to the new angles are:

$$\alpha - \Delta\alpha$$

$$\beta - \Delta\beta$$

$$K_a(e) = \frac{\mathrm{Cos}^2(\phi+\alpha-\Delta\alpha)}{\mathrm{Cos}^2(\alpha-\Delta\alpha) * \left[1 - \sqrt{\dfrac{\mathrm{Sin}(\phi+\delta) * \mathrm{Sin}(\phi-\beta+\Delta\beta)}{\mathrm{Cos}(\alpha-\Delta\alpha-\delta) * \mathrm{Cos}(\alpha-\Delta\alpha+\beta+\Delta\beta)}}\right]^2} \times \frac{1}{\mathrm{Cos}(\alpha-\Delta\alpha-\delta)}$$

$$K_{ah}(e) = \frac{\mathrm{Cos}^2(\phi+\alpha-\Delta\alpha)}{\mathrm{Cos}^2(\alpha-\Delta\alpha) * \left[1 - \sqrt{\dfrac{\mathrm{Sin}(\phi+\delta) * \mathrm{Sin}(\phi-\beta+\Delta\beta)}{\mathrm{Cos}(\alpha-\Delta\alpha-\delta) * \mathrm{Cos}(\alpha-\Delta\alpha+\beta+\Delta\beta)}}\right]^2}$$

To calculate the effect of earthquake loading the coefficient K_{ah} should be replaced by $K_{ah}(e)$. To increase safety, sometimes an additional assumption is to make $\delta = 0$.

1.4.2.2.2 PASSIVE EARTH PRESSURE
Figure 1-23 schematically shows the rotated system for the passive earth pressure. For the passive soil pressure considering earthquake loading, and in accordance to the direction of angles adopted, the fictitious displacement lead to the new angles are:

$$\alpha + \Delta\alpha$$

$$\beta - \Delta\beta$$

and

$$K_p\,(e) = \frac{\cos^2(\alpha+\Delta\alpha-\phi)}{\cos^2(\alpha+\Delta\alpha) * \left[1-\sqrt{\dfrac{\sin(\phi-\delta) * \sin(\phi+\beta-\Delta\beta)}{\cos(\alpha+\Delta\alpha-\delta) * \cos(\alpha+\Delta\alpha+\beta-\Delta\beta)}}\right]^2} \times \frac{1}{\cos(\alpha+\Delta\alpha-\delta)}$$

$$K_{ph}\,(e) = \frac{\cos^2(\alpha+\Delta\alpha-\phi)}{\cos^2(\alpha+\Delta\alpha) * \left[1-\sqrt{\dfrac{\sin(\phi-\delta) * \sin(\phi+\beta-\Delta\beta)}{\cos(\alpha+\Delta\alpha-\delta) * \cos(\alpha+\Delta\alpha+\beta-\Delta\beta)}}\right]^2}$$

To calculate the effect of earthquake loading the coefficient Kph should be replaced by Kph(e). However, according to recommendations given in DIN 1054 the normal practice is to neglect passive earth pressures to increase safety against earthquakes.

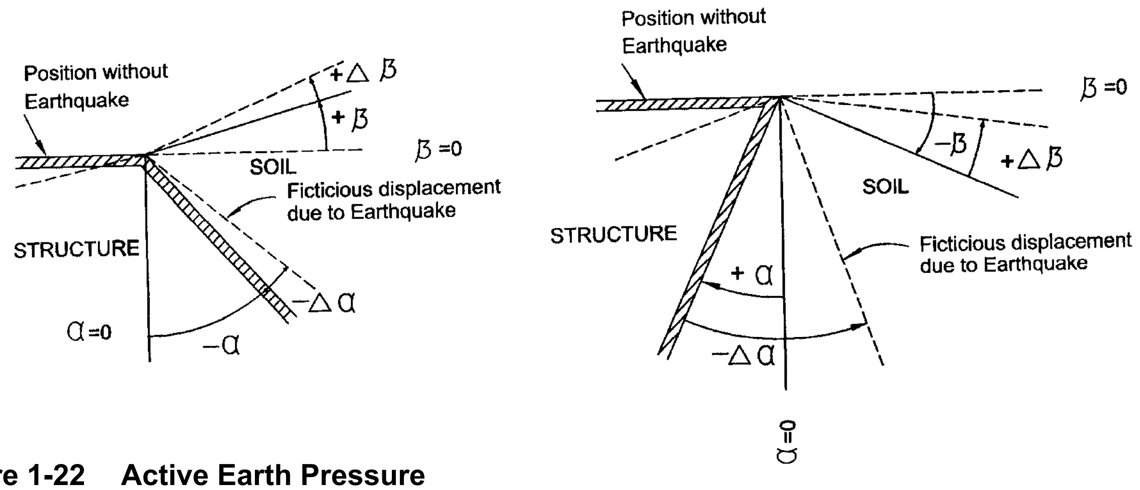


Figure 1-22 Active Earth Pressure

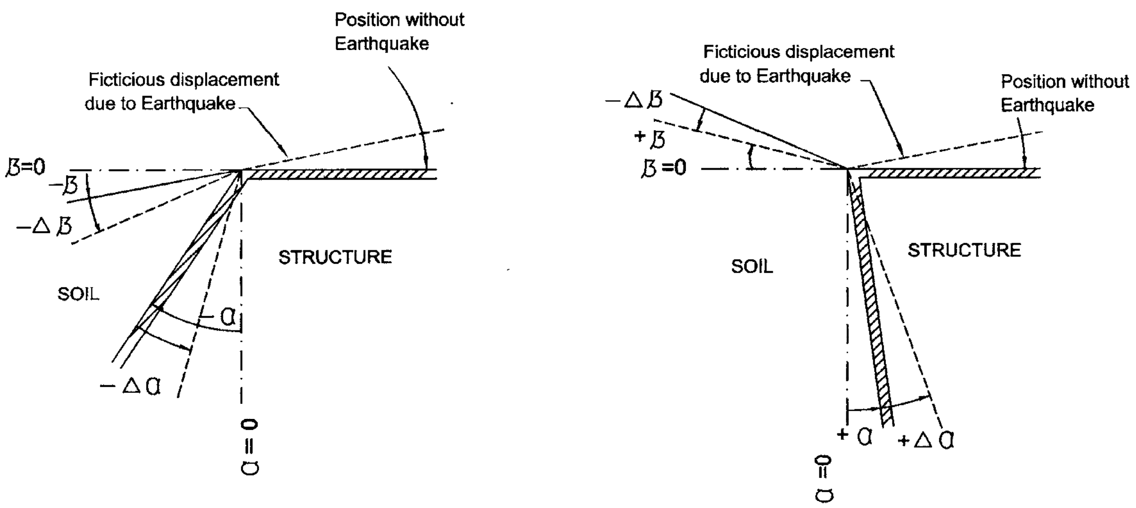


Figure 1-23 Passive Earth Pressure

1.4.3 RANKINE'S EARTH PRESSURE THEORY

In 1857 Rankine introduced this theory and developed it for purely non-cohesive soils(C=0) but later on in 1915 Bell extended this theory to C-ϕ soils as well. The original assumptions made by Rankine are:

- Soil is non-cohesive dry, isotropic and homogeneous.
- Backfill is horizontal
- Wall is vertical
- Failure is a plain strain problem

Taking a unit length of an infinitely long wall as shown below

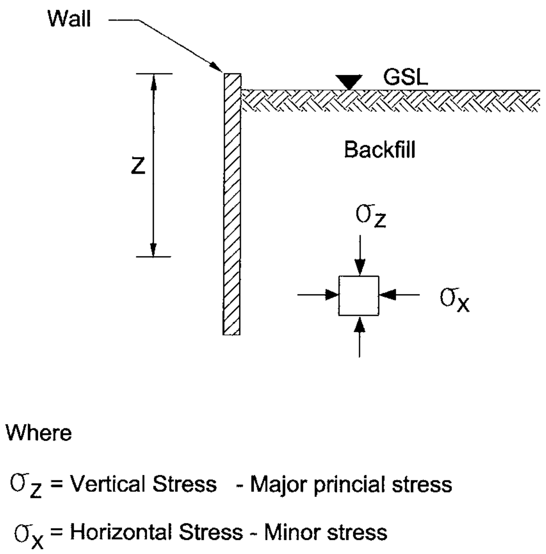


Figure 1-24 Soil Element-Rankine Theory

Considering Mohr Circle as shown below

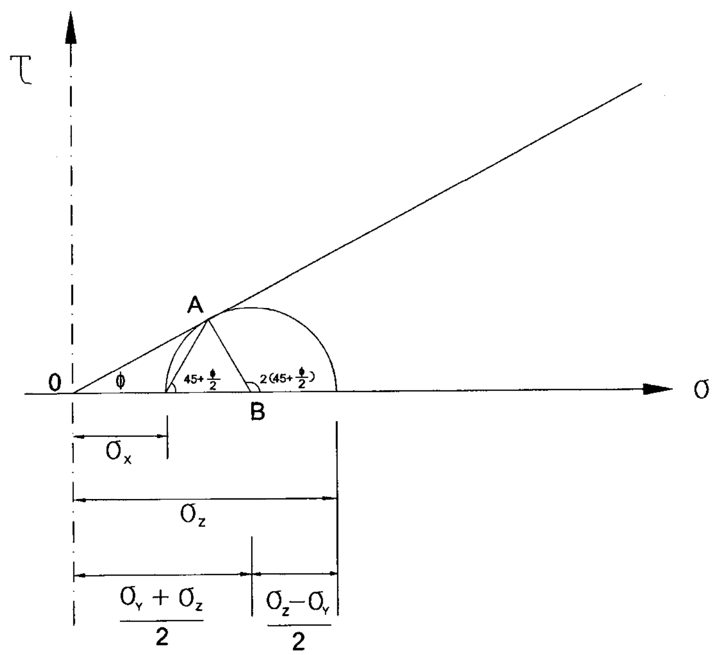


Figure 1-25 Mohr Circle-Rankine Theory For C = O

From OAB

$$\text{Sin}\,\phi = \frac{AB}{OB} = \frac{\frac{1}{2}(\sigma_z - \sigma_x)}{\frac{1}{2}(\sigma_z + \sigma_x)}$$

$$\text{Sin}\,\phi(\sigma_z + \sigma_x) = (\sigma_z - \sigma_x)$$

or

σ_z (1 - Sin ϕ) = σ_x (1 + Sin ϕ)

Then

$$K_a = \frac{\sigma_x}{\sigma_z} = \frac{(1-\text{Sin}\,\phi)}{(1+\text{Sin}\,\phi)} = \tan^2\left(45-\frac{\phi}{2}\right)$$

σ_x = active earth pressure = $\sigma_z K_a$

σ_a = γ Z $\tan^2$ (45 - ϕ/2)

Similarly

K_p = $\tan^2$ (45 + ϕ /2)

σ_p = γ Z $\tan^2$ (45 + ϕ/2)

For backfill at sloping angel of β as shown in Figure below

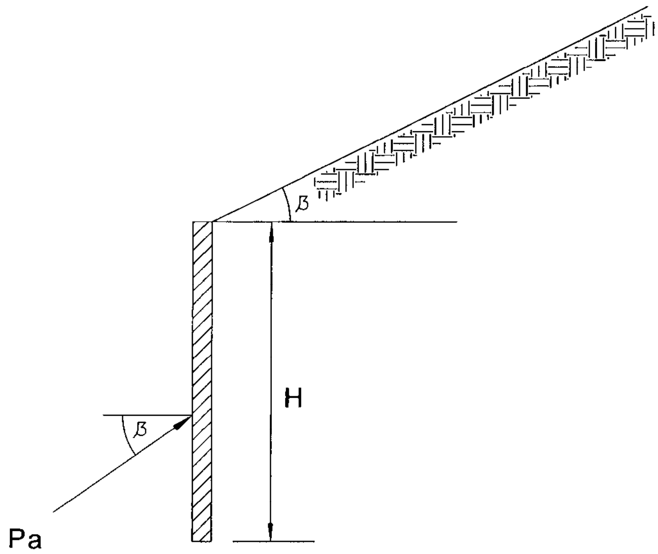


Figure 1-26 Retaining wall with sloping backfill

$$K_a = \frac{\text{Cos}\,\beta - \sqrt{(\text{Cos}^2\beta - \text{Cos}^2\phi)}}{\text{Cos}\,\beta + \sqrt{(\text{Cos}^2\beta - \text{Cos}^2\phi)}}$$

$$K_p = \frac{\text{Cos}\,\beta + \sqrt{(\text{Cos}^2\beta - \text{Cos}^2\phi)}}{\text{Cos}\,\beta - \sqrt{(\text{Cos}^2\beta - \text{Cos}^2\phi)}}$$

Bell Modification in Rankine theory for c- ϕ soils
He suggested the active earth pressure such as:

$\sigma_a = K_a \gamma Z - 2C \sqrt{K_a}$
$\sigma_a = K_p \gamma Z - 2C \sqrt{K_p}$

This theory should be used for analysis of cantilever and counterfort retaining walls.

1.4.4 COMPUTER PROGRAMME FOR RETAINING WALL STABILITY ANALYSIS
1.4.4.1 GENERAL
The program has been developed by GTZ for stability analysis of gravity retaining walls. The program has been written in FORTRAN-IV programming language and compiled on an IBM Compatible Computer running on DOS Disk Operating System. The programme is based on Coulomb's Theory.

The program's objective is to carry out stability analysis of gravity retaining walls subjected to various static and dynamic loads. The program allows to carry out 2-dimensional stability analysis of gravity retaining walls with practically any geometry. For this reason, in its present version the program does not optimise the geometry of the retaining wall.

The minimum required factors of safety in accordance to the indicated stability tests are given in Table below.

Table 1-4 Factor of Safety for Gravity Retaining Wall

Sr. No.	Description	LC 1	LC 2	LC 3
1	Sliding	1.5	1.35	1.2
2	Soil rupture	2.0	1.5	1.3
3	Slope rupture	1.4	1.3	1.2
4	Flotation	1.1	.1.	1.05
5	Hydraulic Heave	3.0	2.5	2.0

LC1 = Normal loading including traffic loads, wind etc.
LC2 = Case 1 + unfrequent large traffic, special loading during construction
LC3 = Case 2 + Loads of low probability, i.e., earthquake, accident and failure during construction

The detail about how to use this programme can be found in "**Guide Lines Retain**" for Retaining Wall Stability Analysis Programme. However, small description about the data, the method on which it is based is given below.

1.4.4.2 STABILITY TESTS
The stability tests presently included in the computer program are described in the following paragraphs. These tests are always carried out for loading case 1 and loading case 3 (in case of earthquake loading).

1.4.4.2.1 SLIDING

$$\eta_a = \frac{\Sigma H\,(Max.\,Re\,acting\ Forces\,(T_o - E_{pr}))}{\Sigma H\,(Available\ Acting\ Forces)} < \eta_r$$

Where
P_z = Vertical traffic load
G = Self weight

G_w = Weight of water acting vertically
W_O = Uplift pressure
E_{av} = Vertical active earth pressure
P_x = Horizontal traffic load
W_1 = Lateral water pressure due to ground water(Gr W)
W_2 = Lateral water pressure due to normal water(N W)
E_{ah} = Horizontal passive earth pressure
E_{pr} = Mathematical passive earth pressure

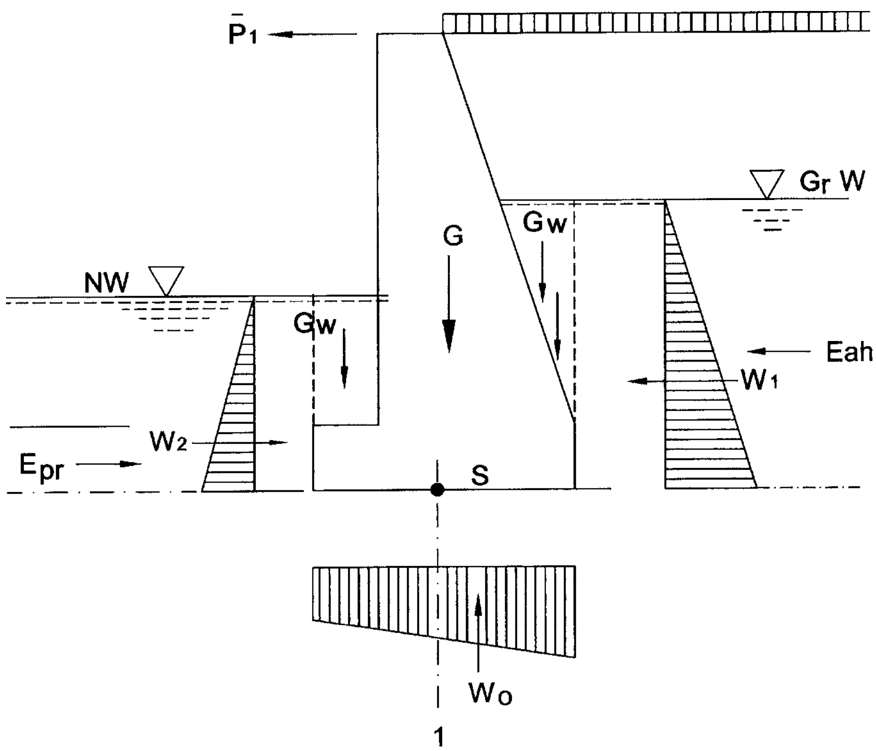


Figure 1-27 Horizontal Sliding

1.4.4.2.2 SOIL STRESSES

$$\eta = \frac{\text{Eccentricity, } e}{\text{Width of Foundation, } b} \leq \begin{cases} 1:6 & \text{Permanent Load} \\ 1:3 & \text{Total Load} \end{cases}$$

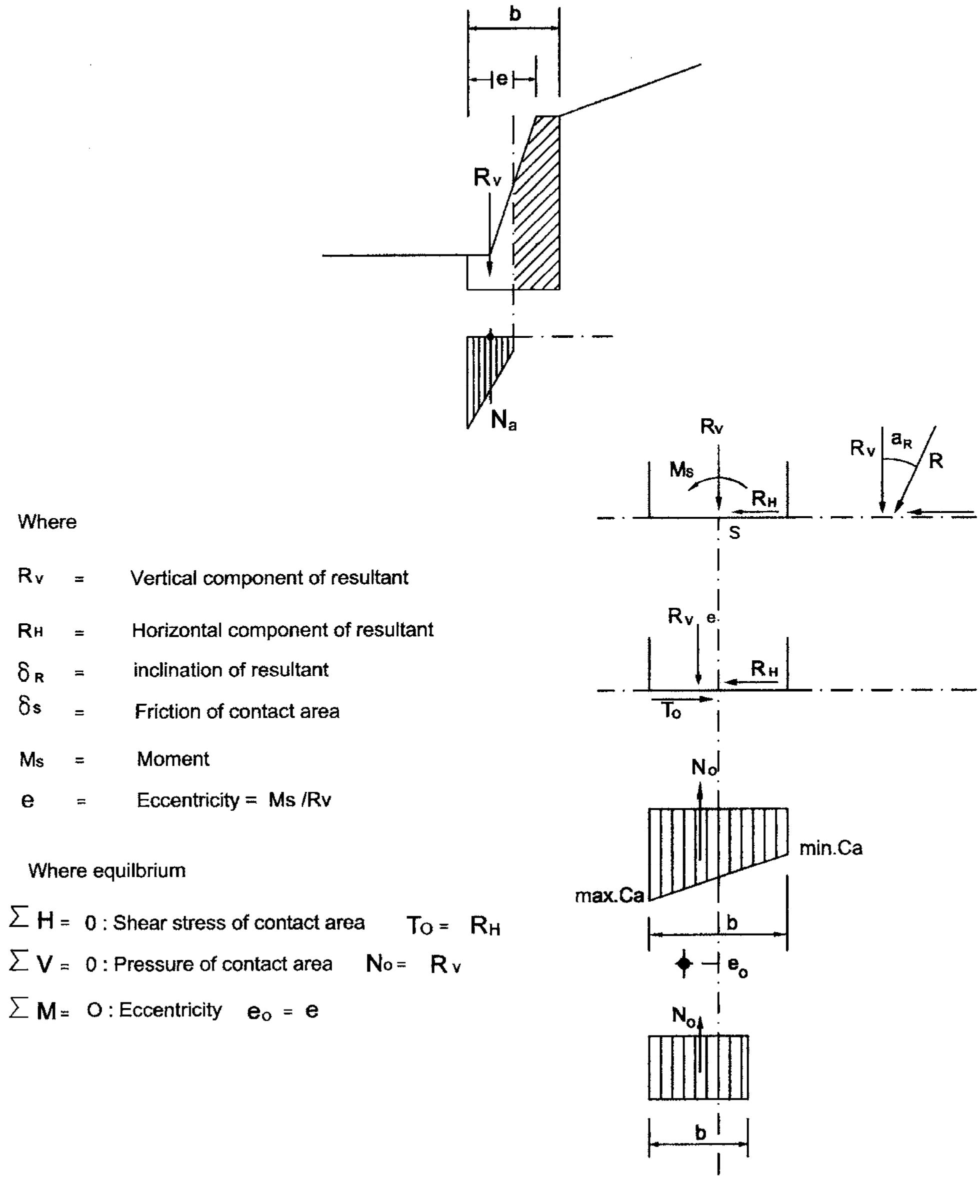


Figure 1-28 Soil Stress- Retaining Wall

Mathematical pressure of contact area

$$\sigma_0 = (Rv/a\ b)$$
$$b' = b - 2\ e$$

Stress of contact area

when $0 < e < b/6$

$$\sigma_0 = (Rv/a)^*\ b\{ 1 + (b\ e/b) \}$$

This will give maximum and minimum value of σ_0.

1.4.4.2.3 BEARING CAPACITY

Bearing capacity indicates the shear resistance of the soil on which the retaining wall is to be constructed. The equation proposed by Hansen in 1970 has been internationally accepted to check that adequate bearing capacity is available in accordance to the safety factor.

The special case of retaining walls allows simplifying Hansen's equation, reducing the number of parameters required. The load is assumed to be inclined and eccentricity applied. In any case the three main components of the equation remain, i.e. cohesion, depth and width.

$$\eta = \frac{\sigma_{of}}{\sigma_{or}}$$

$$\eta \geq 2 \text{ (loading case 1)}$$

When horizontal Surface and loading are as in Figure below.

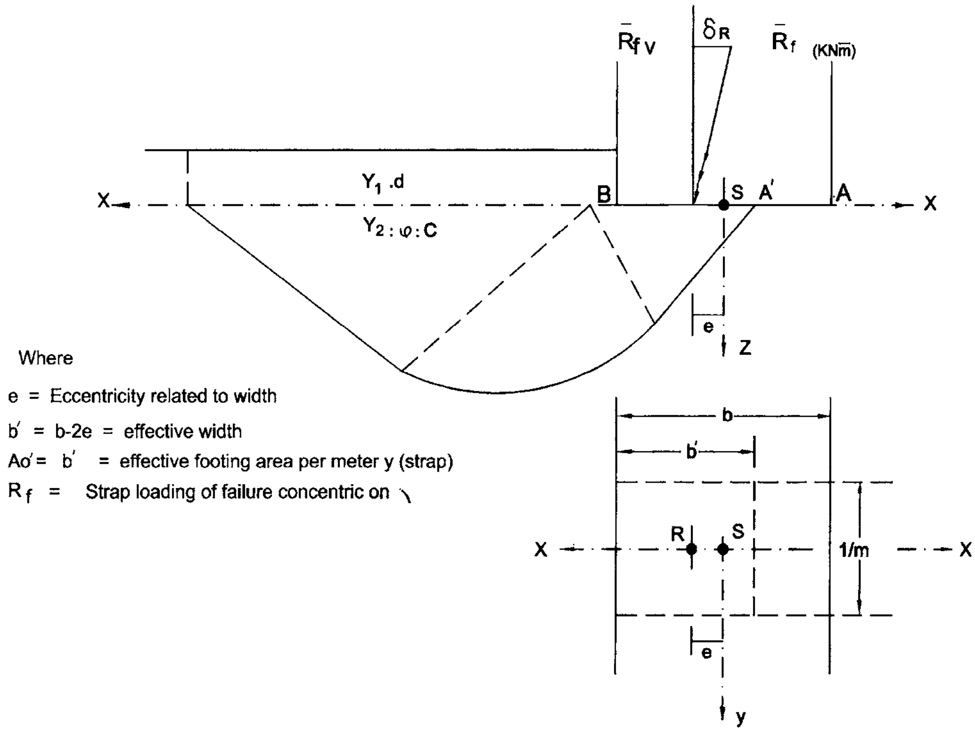


Figure 1-29 Horizontal surface - Eccentric, inclined strap loading

Then stress of soil rupture

$$\sigma_{of} = c \,.\, N_c \,.\, 1.\, K_c + V_1 \,.\, d \,.\, N_d \,.\, 1.\, K_d + V_2 \,.\, b' \,.\, N_b \,.\, 1.\, K_b$$

Bearing Capacity of soil will be

N_c	=	$(N_d - 1) \cot \phi$	CAQUOT 1956
N_d	=	$e^{n.\tan\phi} \tan^2(45^* + \phi/2)$	Prandtl 1920
N_b	=	$(N_d - 1) \tan \phi$	Meyerhof 1962

With angle of sliding area for the Rankine case

$\theta_a \quad = \theta_a' \quad = \quad (45^* + \phi/2)$
$\theta_p \quad = \theta_p' \quad = \quad (45^* - \phi/2)$

Circle	Square	Rectangular	Strap
$\phi = 0$ $V_c = 1 + 0.2 \,.\, b/a$	$\phi = 0$ $V_c = (V_d.N_d - 1)/(N_d - 1)$	$V_d = 1 + 0.2 \,.\, (b/a) \sin\phi$	$V_b = 1 + 0.3\,(b/a)$

Coefficient of inclination of loading K for non-cohesive soil c = 0, $\phi > 0$

$K_c \quad = \quad$ not applicable
$K_d \quad = \quad (1 - 0.7 \tan \delta_R)^3$
$K_b \quad = \quad (1 - 1.0 \tan \delta_R)^3$

Coefficient of inclination of loading K for un-consolidated cohesive soil c > 0, $\phi = 0$ initial strength

$K_b \quad = \quad$ not applicable
$K_d \quad = \quad 1$

$$K_c = 0.5 + 0.5\left[\frac{1 - R_{fH}}{B.1.c_u}\right]$$

Coefficient of inclination of loading K for un-consolidated cohesive soil c > 0, $\phi > 0$ initial strength

$$K_c = K_d - \frac{1 - K_d}{N_d - 1}$$

The component of failure loading have to be calculated by iteration with estimated factor of safety.

Inclined Surface
Stress of soil rupture

$$\sigma_{of} = c \,.\, N_c \,.\, V_c .\, K_c \,.\, \lambda_c + V_1 \,.\, d' \,.\, N_d \,.\, V_d \,.\, \lambda_d \,.\, K_d + V_2 \,.\, b' \,.\, N_b \,.\, V_b .\, K_b \,.\, \lambda_b$$

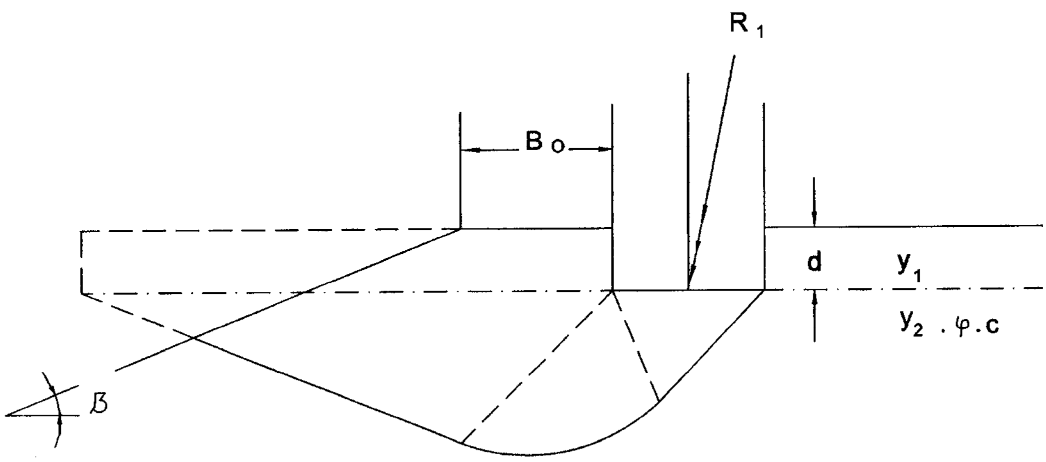


Figure 1-30 Inclined Surface

Coefficient of inclination of slope λ for $\beta < \phi$

λ_c = $(N_d \cdot e^{0.0349 \cdot \beta \cdot \tan\phi} - 1) / (N_d - 1)$
λ_d = $(1 - \tan \beta)^{1.9}$
λ_b = $(1 - 0.5 \cdot \tan \beta)^6$

Where
β in degree

Bearing capacity

- Calculation of σ_{of} with β and embedded depth $d' = d + 0.8 \cdot B_0 \cdot \tan \beta$
- Calculation of σ_{of} with $\beta = 0$, $d' = d$
- The smaller value of σ_{of} should be used.

1.4.4.2.4 HYDRAULIC HEAVE

$$\eta = \frac{Y'}{\max . f_{sv}}$$

$$\eta = \frac{i(\text{critical})}{i(\max.)}$$

$$\eta = \frac{Y'/Y_w}{H_w/\min.L}$$

Where
η > 1.5 for water front structures according to EAU
η > 1.3 recommended for earth structure due to irregularities in the soil
f_{sv} = Maximum seepage pressure

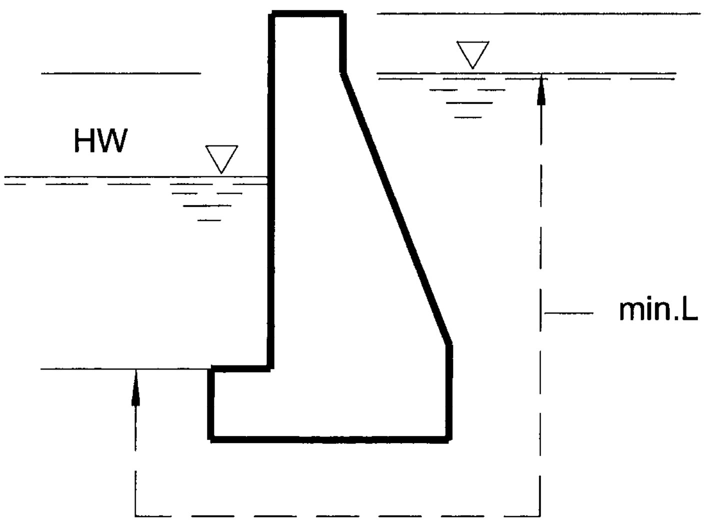


Figure 1-31 Hydraulic Heave - Retaining wall

1.4.4.2.5 FLOATATION

$$\eta = \frac{\text{self weight } G}{\text{uplift pressure } W_o} \geq 1.1$$

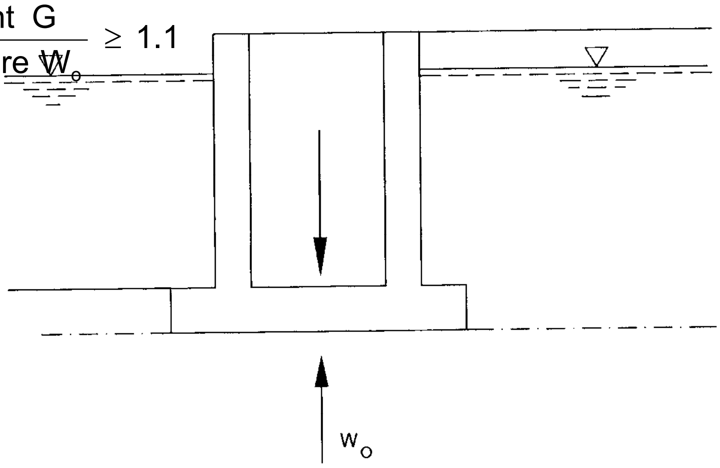


Figure 1-32 Flotation of Retaining Wall

1.4.4.3 INFORMATION REQUIREMENTS
Following are the data requirements. Some of the parameters required can be obtained from the DIN-tables.

1.4.4.3.1 GEOMETRIC DATA
Geometric data comprises the co-ordinates of wall, water and soil as well as the angles needed to calculate active and passive earth pressures.

1.4.4.3.1.1 X-Y CO-ORDINATES
Following are the required co-ordinates:

- Geometry of retaining wall
- Geometry of moist soil right/active side
- Geometry of submerged soil right/active side
- Geometry of water right/active side
- Geometry of moist soil left/passive side
- Geometry of submerged soil left/passive side
- Geometry of water left/passive side

All co-ordinate values should be given in meter, with upto 2 decimal digits.

1.4.4.3.1.2 ANGLES
The angles required in the analysis are as follows:

- α, ß, on soil/active side
- α, ß, on water/passive side

All angles should be given in degrees, according to notation in Figure 1-21.

1.4.4.3.2 SOIL CHARACTERISTICS
Following are the basic parameters required to describe the soil characteristics:

- Angle of internal friction, in degrees.
- Cohesion dimensionless

1.4.4.3.3 SPECIFIC WEIGHTS

Following are required specific weights for water, wall and soil:

- Specific weight of water
- Specific weight of retaining wall
- Specific weight of moist soil
- Specific weight of submerged soil
- Void ratio

For bearing capacity analysis:

- Specific weight of soil above foundation level
- Specific weight of soil below foundation level

Values should be given in kN/m^3.

1.4.4.3.4 LOADS

- Distributed vertical loads on both embankments, in kN/m^2.
- Horizontal earthquake acceleration factor, in %.

1.4.4.3.5 MISCELLANEOUS GEOMETRIC DATA

- Embedded depth of foundation (d), in meter.
- Horizontal distance to slope (B_0), in meter.
- Length of wall, in meter.

1.5 SLOPE STABILITY

1.5.1 GENERAL

In low-head power plant headrace and tailrace is one of the most important components. These are made by excavating the earth or by filling of embankment. The embankments of the headrace and tailrace are usually protected by stone or concrete or burnt clay brick to safe guard against erosion under normal and special operation conditions. The stability of the slope has to be checked under normal and special conditions especially when water level in the canal decreases rapidly, causing seepage flow into the canal.

Numerous methods for stability analysis of slope are available but, in general all the available methods can be divided into three categories as defined below.

1.5.2 SLIDING SURFACE METHODS

1.5.2.1 GENERAL

All these methods assume the validity of Coulomb's law of failure. These methods do not account for the load deformation characteristics of the material in question. Most of the methods currently in use fall under this category. The following steps are performed under these methods:

- Assumed failure plane is chosen
- Shear strength along assumed plane is calculated
- Disturbing moment and the resisting moment are calculated.
- Factor of safety should be computed such as

$$\text{FOS} = \frac{M_R}{M_D} = \frac{\text{Resisting moment}}{\text{Driving moment}}$$

- A number of failure planes are tried and the one with the least FOS is located. This plane is called critical plane.

Most of the engineers prefer to use these methods and a few will be given in details.

1.5.2.2 TAYLOR'S SLOPE STABILITY NUMBER METHOD.

In 1937 Taylor introduced this method and used a number of variables such as:

- Undrained cohesion of soil = c_u
- Angle of internal friction = ϕ
- Unit weight of soil = γ
- Slope height = H
- Slope angle = ι
- Factor of safety = F

According to Taylor's

$$m = c_u / F \gamma H$$

By using four out of six variables he prepared two graphs for saturated and partial saturated soil such as:

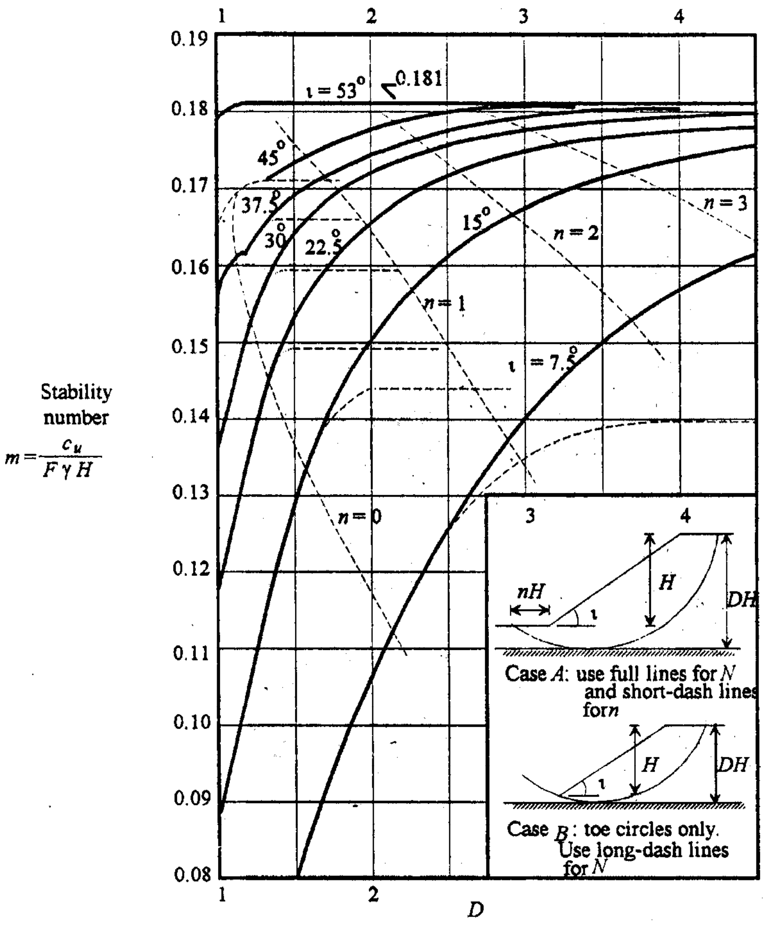


Figure 1-33 Taylor Stability Number Chart for Cohesive Soil.

This show that the toe failure will occur for all slopes steeper than 530. For slopes less than 530, however there are, three possibilities of failure depending upon DH such as:

For DH ≥ 3 Base failure with slip circle tangent to the hard stratum
For DH = 1 Base failure, toe failure, slope failure may occur depending on slope
For DH < 1 Slope failure only will occur.

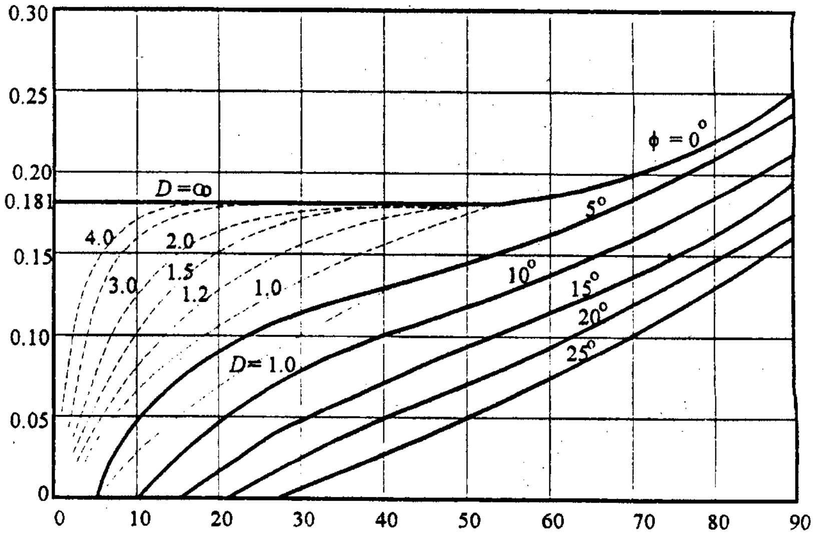


Figure 1-34 Taylor's Stability number Chart for φ greater than 0

1.5.2.3 KREY'S METHOD

A circular sliding surface is assumed. The calculation for stability against sliding of the soil wedge is based on the slices method. The basic formulae are:

$$\eta_{av} = \frac{M_R}{M_D} = \frac{\text{Resisting moment}}{\text{Driving moment}}$$

$$\eta_{av} = \frac{T_i \text{ x } R}{G_i \text{ Sin}_i \text{ x } R \text{ x } M_e}$$

where

$$T_i = \frac{G_i + c \text{ x } b \text{ x Cot} - (U + U)\, b}{\text{Cos x Cot} + \text{Sin}}$$

where

G_i	= own weight of one slice
M_e	= external moments from external disturbing forces
G_i x Sin i	= the disturbing force for each slice, from its own weight
T_i	= the resisting tangential force from friction
i	= the angle of the tangent to each slice

1.5.2.4 BISHOP'S METHOD OF ANALYSIS

The method was first described by Bishop in 1955 and a simplified version was developed further by Janbu et. al. in 1956. In this method, it is assumed that the forces acting on the sides of any slice have zero resultant in the vertical direction, as shown in Figure below. This is the method recommended by DIN 4084 and is attributed only to Bishop.

Another important feature of this method is that the safety factor is used to express the shear forces acting along the failure arc. Therefore, the equation for the safety factor takes the form:

$$T_i = N_i \tan\phi + c \, . \, l_i$$

$$W_i = N_i \, Cos\theta_i - \left[\frac{1}{\eta} . N_i \tan\phi \, Sin\theta_i + \frac{1}{\eta} \, . \, c \, . \, l_i \, . \, Sin\theta_i \right] = 0$$

$$\Sigma F_v = 0: \; -N_i \, Cos\theta_i - \frac{1}{\eta} T_i \, Sin\theta_i$$

$$N_i = \frac{W_i - c.l_i . \frac{1}{\eta} Sin\theta_i}{\frac{1}{\eta} Sin\theta_i \tan\phi + Cos\theta_i}$$

Combining previous equations:

$$T_i = \frac{W_i - c.l_i . \frac{1}{\eta} Sin\theta_i}{\frac{1}{\eta} Sin\theta_i \tan\phi + Cos\theta_i} \tan\phi + c.l_i$$

$$T_i = \frac{W_i \tan\phi - c.l_i . \frac{1}{\eta} Sin\theta_i \tan\phi + c.l_i . \frac{1}{\eta} Sin\theta_i \tan\phi + c.l_i \, Cos\theta_i}{\frac{1}{\eta} Sin\theta_i \tan\phi + Cos\theta_i}$$

$$T_i = \frac{W_i \tan\phi + c.l_i \, Cos\theta_i}{\frac{1}{\eta} Sin\theta_i \tan\phi + Cos\theta_i}$$

Knowing that:

$$\eta = \frac{\sum_{i=1}^{n} T_i \, r}{\sum_{i=1}^{n} W_i \, Sin\theta_i \, r}$$

$$\eta = \frac{\sum_{i=1}^{n} \left[\dfrac{(W_i . - u_i\, b_i)\tan\phi + c.b_i}{\frac{1}{\eta}\, Sin\theta_i \tan\phi + Cos\theta_i} \right]}{\sum_{i=1}^{n} W_i\, Sin\theta_i}$$

1.5.2.5 FELLENIUS METHOD OF ANALYSIS

This method is also known as Swedish Circle Method or Fellenius Method. The subdivision of the trial wedge in slices was first proposed by Fellenius in 1936. In this method, it is assumed that the forces acting on the sides of any slice have zero resultant in the direction normal to the failure arc of the slice, as shown in Figure below.

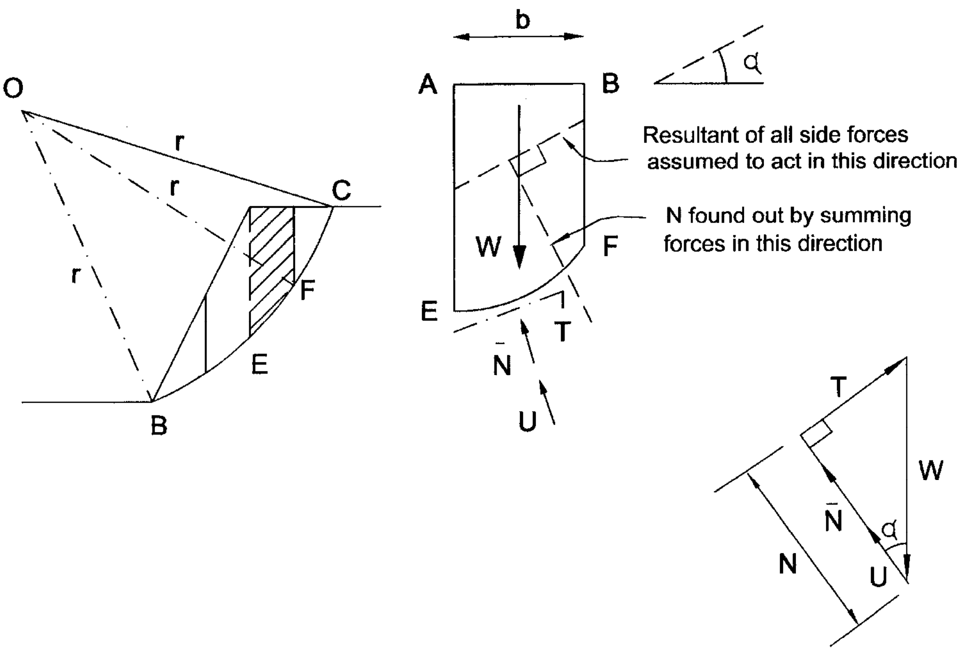


Figure 1-35 Slope Stability

$$\Sigma F_v = 0: \quad W_i\, Cos\theta_i = N_i + U_i$$

Solving for the normal force N_i acting perpendicular to the failure arc, we get:

$$N_i = W_i\, Cos\theta_i - U_i = W_i\, Cos\theta_i - u_i\, l_i$$

The destabilising moments are expressed as:

$$M_D = \sum_{i=1}^{n} W_i\, r\, Sin\theta_i$$

$$M_R = r\left[\sum_{i=1}^{n} N_i \tan\phi + \sum_{i=1}^{n} c.l_i\right]$$

The resisting moments are:

$$\eta = \frac{r\left[\sum_{i=1}^{n} N_i \tan\phi + \sum_{i=1}^{n} c.l_i\right]}{\sum_{i=1}^{n} W_i \ r \ \mathrm{Sin}\theta_i}$$

Combining previous equations, the safety factor can be expressed as:

$$\eta = \frac{\tan\phi \sum_{i=1}^{n}(W_i \ \mathrm{Cos}\theta_i + c.l_i - u_i.l_i)}{\sum_{i=1}^{n} W_i \ \mathrm{Sin}\theta_i}$$

The assumptions in this method always lead to an error because the system is over-determined with *n-1* assumptions and *n-2* unknowns. In general it is not possible to satisfy static stability as a balance of forces involved in not achieved. Normally, the safety factor is underestimated as compared to other methods, which satisfy static stability.

1.5.3 LIMIT ANALYSIS METHOD

This method considers yield criteria and the stress strain relationship. It is based on lower bound and upper bound theorem for bodies of elastic or plastic materials. The following step are involved.

- Compute stress using elastic or plastic theories
- Compare unit stress with unit shear strength

1.5.4 FINITE ELEMENT METHOD

This method accounts for deformation and is useful where significantly different materials are used in slopes along which the probable movement of the soil mass may occur. It is a more rigorous method and is used in more complex problems.

1.5.5 ACCEPTABLE FACTOR OF SAFETY

The following factors of safety are acceptable for slope stability analysis:

- FOS ≤ 1 Stability is questionable
- FOS of 1 to 3 For cuts and fills other than earth dams
- FOS ≥ 3 Earth dam

1.5.6 EXAMPLE FOR STABILITY OF SLOPE

Figure shows a cross-section of a slope which has the following properties

- γ =1900 Kg/m^3
- $\phi = 20^0$
- c = 25 KPa
- ru = 0.3

Determine the FOS using Bishop method of analysis.

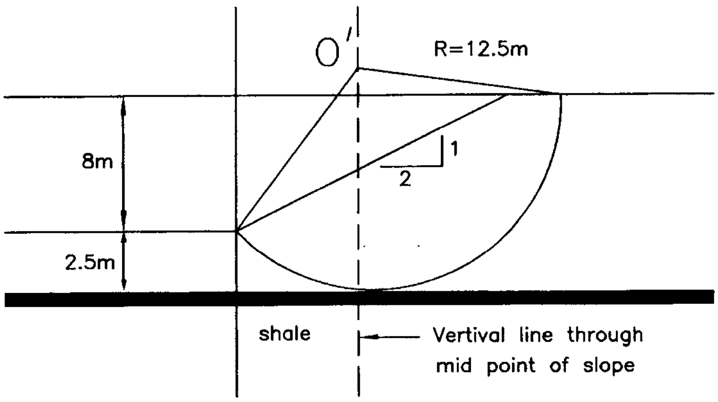


Figure 1-36 Slope Cross-Section

Solution
Divide the slip circle into six slices each of 3.4 m width. Construct the force triangle as shown in Figure below:

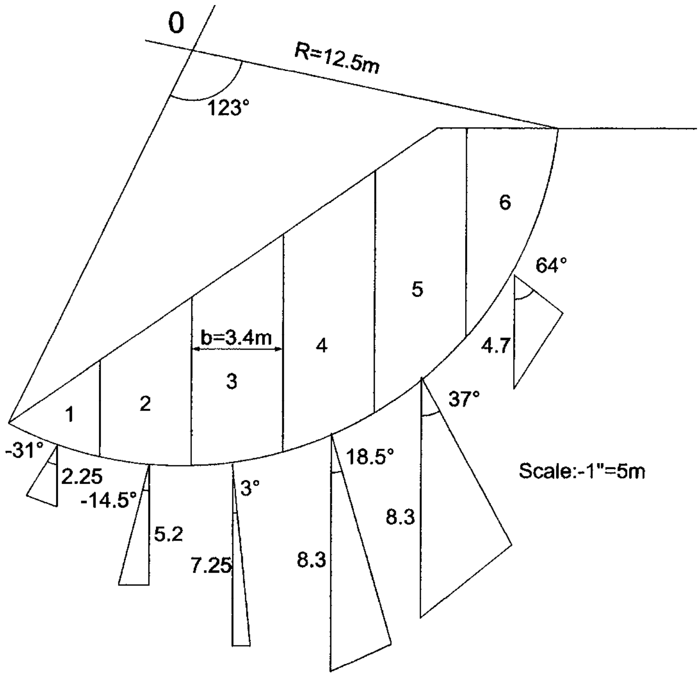


Figure 1-37 Slope Stability Force Triangle

According to Bishop's method

$$F = \frac{\sum \frac{cb + (W - ub)\tan\phi}{M\alpha}}{\sum W \operatorname{Sin}\alpha}$$

$$M\alpha = \operatorname{Cos}\alpha\left(\frac{1 + \tan\alpha\tan\phi}{F_m}\right)$$

F_m = The trial value of FOS

Slice	b	H	W	α	sin α	(4)* (6)	cb	u	ub	tan ϕ (W-ub)	(8)* (11)	Mα	(12)* (13)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	3.4	2.25	142.6	-31	-0.515	-73.44	85	12.58	42.77	36.34	121.34	0.732	165.77
2	3.4	5.2	329.5	-14.5	-0.250	-82.5	85	29.08	98.87	83.94	168.94	0.907	186.26
3	3.4	7.25	459.4	3	0.052	24.4	85	40.54	137.84	117.04	202.04	1.011	199.84
4	3.4	8.3	526.0	18.5	0.317	166.9	85	46.41	157.79	134.02	219.02	1.025	213.68
5	3.4	8.3	526.0	37	0.602	316.5	85	46.41	157.79	134.02	219.02	0.945	231.77
6	3.4	4.7	297.9	64	0.899	267.75	85	26.28	89.35	75.91	160.91	0.656	245.29
Σ						619.3							1242.61

For F_m = 1.5 FOS = 1242.61/619.3 = 2.01 OK.

1.5.7 COMPUTER PROGRAMME FOR SLOPE STABILITY ANALYSIS

1.5.7.1 GENERAL

The program has been developed by GTZ for the stability analysis of embankment slopes using the method of slices, allowing the possibility to consider multi layered soils and retaining walls. The program could also be used for preliminary analysis of embankment dams, provided that a pseudo static analysis is sufficient to account for earthquake loading.

The program has been written in FORTRAN-IV programming language and runs on any IBM-compatible personal computer on DOS (Disk Operating System).

The main objective of the program is to provide a computational efficient and easy to use tool for the computation of stability of embankments using internationally accepted engineering practice. In this connection, DIN 4084 norms have been used as a basis in the determination of factors of safety.

The detail about how to use this programme can be found in "**Guidelines for the Use of Program Slope**" for Slope Stability Analysis Programme. However, small description about the data, the method on which it is based is given below.

1.5.7.2 METHODOLOGY

The program follows standard procedures in the determination of stability of embankments. In every case, a circle of failure is selected and subdivided in various slices, with the following main features:

- A rectangular grid where the centre of the slip circle can be located
- Various soil layers can be considered
- Stability of retaining walls against slope failure may be evaluated
- Free and groundwater can be considered
- Earthquake forces are accounted for through pseudo-static method

1.5.7.3 FORCES DUE TO WATER

Water should be accounted for in determining the stability of slopes in two different manners:

- Uplift force
- Free water

1.5.7.3.1 UPLIFT

Uplift forces are calculated for saturated soils as recommended in DIN 4084, approximating through the position of the seepage line. A further simplification, especially in case of retaining walls, may be the assumption of horizontal flow.

1.5.7.3.2 FREE WATER
Free water is considered as an additional acting weight, which destabilises the slope. Water cannot be considered as part of the stabilising moments because it cannot resist shear stresses.

1.5.7.4 EARTHQUAKE LOADING
A pseudo-static analysis is carried out to take into consideration earthquake loading. The horizontal acceleration forces are therefore calculated as a function of the earthquake acceleration factor and the weight of the soil and/or retaining wall in each slice within the slip circle.

1.5.7.5 STABILITY TESTS
Stability tests are carried in accordance to 2 methods, such as:

- **FELLENIUS**

The simplified method corresponds to Fellenius assumptions. Normally this method tends to underestimate the safety factor as compared to the simplified Bishop method.

- **BISHOP**

This method is also known as simplified Bishop method and is recommended in DIN 4084 and should be used as a basis to determine stability of slopes.

1.5.7.6 INFORMATION REQUIREMENTS
The information required to apply the program is exactly the same which will be needed to manually carry out stability analysis of slopes and retaining structures. The only special consideration is with respect to the sequence in which the data input is done and the basic rules adopted to describe the geometry of each soil layer.

It is strongly recommended to prepare a detailed drawing (cross section) of the slope and/or retaining wall to facilitate the input of all geometric data.

1.5.7.6.1 GEOMETRIC DATA
The required geometric data comprises:

- Co-ordinates of soil layers and/or retaining wall. Both retaining wall and soil layer are described by giving co-ordinates of upper part. At least 2 points are required to define a layer.
- Co-ordinates of free and surface water. At least 2 points are required to define the position of the water.
- Co-ordinates of forced point to pass the slip circle. Following two cases are common:

  - For soil slopes, at inflection point where slope begins
  - For retaining walls, at lowest point of contact between soil and wall in the back of wall

- Co-ordinates of centre of trial slip circle with incremental values in x and y directions
- Co-ordinates of uniform load. Load is assumed to be horizontally applied.

1.5.7.6.2 SOIL CHARACTERISTICS
The basic parameters required to define soil characteristics comprise:

- Specific weights of soil layers and retaining structure
- Angle of internal friction of each soil layer
- Cohesion of each soil layer

1.5.7.6.3 LOADS
Only uniformly distributed loads are considered.

1.6 EMERGENCY RELIEVING STRUCTURE
1.6.1 GENERAL
Every hydropower plant must be provided with means to pass flood water or the flows during closure of the powerhouse due to tripping unit, faults on transmission or distribution network. The structure constructed for this purpose is named as relieving structure or generally spillway. These structures have many types such as:

- Free over fall spillway
- Side spillway
- Labyrinth weir
- Gated spillway
- Orifice spillway
- Siphon spillway
- Bottom outlet
- Shaft spillway, etc.

In low-head power plant the spillway of siphon, gated and bottom outlet are commonly used. Bottom outlets equipped with hydraulically operated gates are very common and should be employed in plants on irrigation canals. The gated spillway is the most common type being used in hydropower plants. In Pakistan almost all the low-head power plants are equipped with this type of spillway. Pakistan has a very good experience in the operation and maintenance of such type of spillway because all the barrages meant for irrigation are equipped with this type of spillway. The type of gates used is radial and vertical with counter weight. The spillway gates are very important and have been decided at first before the design of the spillway.

The design of spillway means to know about the overall length and the means of energy dissipation i.e., size of stilling basin and apron. The static and hydraulic stability has to be checked very carefully as many structures have failed in the past due to these phenomena. The hydraulic design of different type of spillways will be discussed in the proceeding paragraphs.

1.6.1.1 GATED SPILLWAY
In this type of spillway, weir crest may be sharp or broad. The discharge capacity of a sharp crested weir (Figure : 1 - 36) is given by the formula

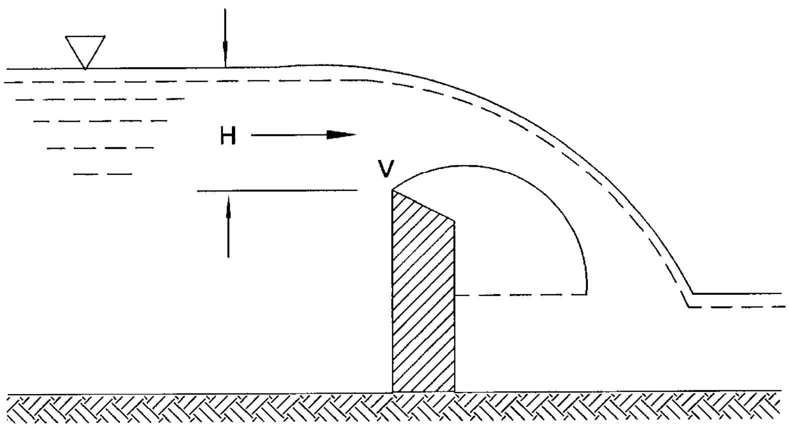


Figure 1-38 Sharp Crested Weir

$$Q = 3.29 \ (b - H/10)\ H^{3/2}$$

Where

Q = Discharge, ft^3/Sec
H = Length of Weir, ft
b = Head over the crest, ft
a = Contraction at weir site in canal section, ft
P = Height of crest, ft

and

$$V = Q / b\ (P + H)$$

Where

V = Approach velocity, ft/Sec.

But when a = 0 which means that the full width of the canal is being used for weir. Then

$$Q = K_d \ b\ H^{3/2}$$

Where

K_d = Coefficient of discharge. Its value depend upon type of crest and approach velocity.

But practically sharp crested weirs are seldom used in gated spillway. The common type of crest being used is flat, broad and rounded. The stream flowing over the broad crested weir depend upon the width of the crest such as if width of crest is more than 2/3H, then stream become 2/3H before leaving the crest as shown below

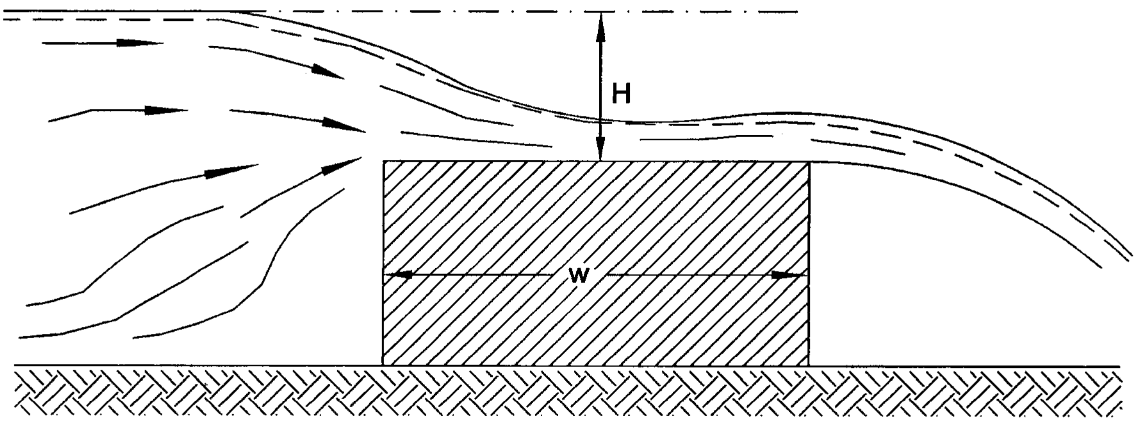


Figure 1-39 Broad Crested Weir.

The effect of the downstream water rising above the crest of the weir is surprisingly small as far as discharge for a given upstream head is concerned. The reduction in discharge does not exceed 2 to 3 % for increase in downstream head of about 20 %.

The suggested values of discharge co-efficient K_d is given in table below for different type of weir crest.

Table 1-5 Values of K_d

H	w (ft)						a (inch)					cot θ			
(ft)	0.48	0.93	1.85	3.17	8.98	16.3	9	18	36						
0.5	3.01	2.76	2.73	2.66	2.61	2.61	3.22	3.23	3.23	3.23	3.22	3.22	3.64	3.31	3.14
1.0	3.24	3.01	2.93	2.70	2.66	2.64	3.35	3.46	3.46	3.27	3.57	3.44	3.82	3.33	3.42
1.5	3.33	3.19	3.03	2.73	2.67	2.65	3.44	3.61	3.64	3.40	3.59	3.59	3.83	3.34	3.52
2.0	3.33	3.29	3.08	2.73	2.67	2.65	3.47	3.68	3.75	3.46	3.60	3.66	3.69	3.35	3.61
3.0	3.33	3.33	3.12	2.71	2.64	2.61	3.48	3.75	3.87	3.67	3.58	3.68	3.55	3.38	3.66
4.0	3.33	3.33	3.15	2.69	2.61	2.59	3.48	3.81	3.96	3.65	3.55	3.70	3.55	3.39	3.66
	a						b			c	d	e			f

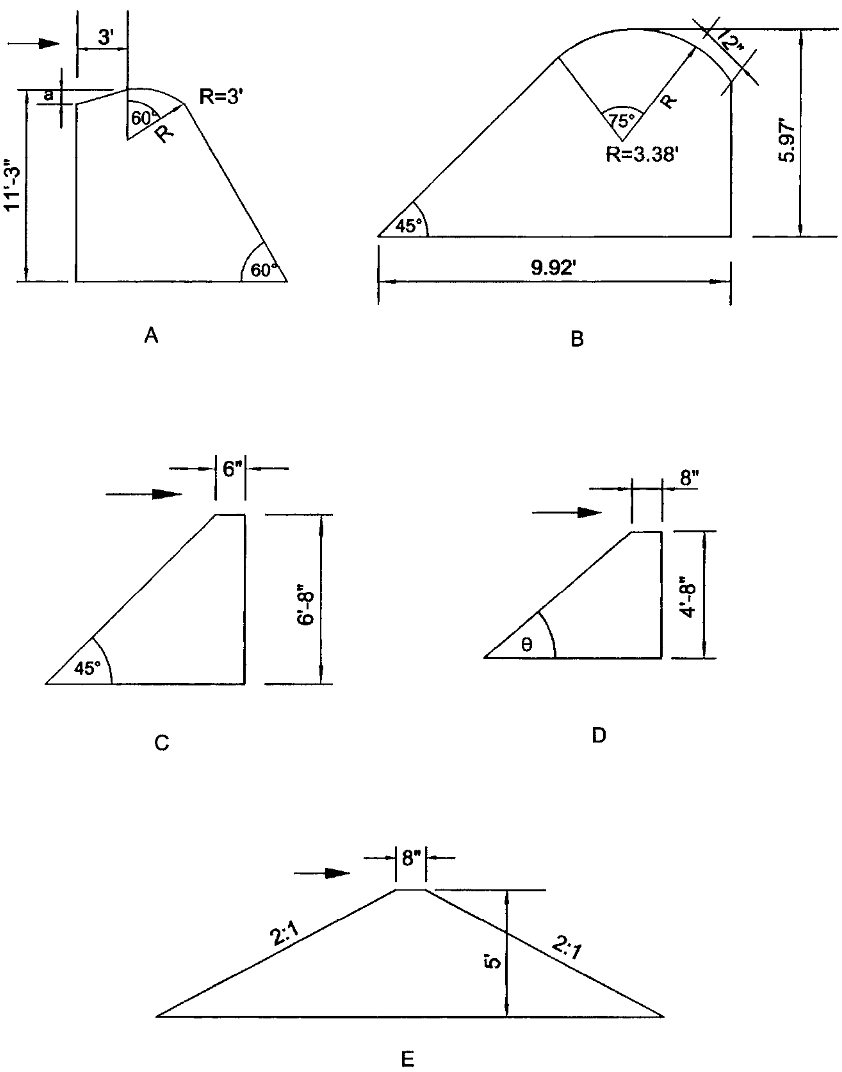


Figure 1-40 Weirs having Various Types of Crest

1.6.1.2 SIPHON SPILLWAY

Siphon spillway occupies far less space than the free flow spillway. It is suitable for limiting the rise of water level in canal forebay. In normal type of siphon spillway, the crest is at normal level in case of reservoir or full supply level in case of irrigation canal and water will flow through the siphon as soon as the level rise above the crest. The flow of water through the siphon passage tends to evacuate the air and siphon primes and flow increase up to maximum capacity. It will continue flowing in this manner until the level falls to the crest level. As soon as the siphon is primed, its discharge increases abruptly to its maximum rate. A well designed siphon should prime when upstream water level rises 1/3 of the throat height. To prime a siphon at early stage few design changes given below have to be made:

- Introduction of offset
- Introduction of water seal along with offset
- Introduction of hanging siphon

The upstream lip of the siphon is generally kept below the crest level to avoid air entrance by vortex or by wave action. Then introduction of air vent at crest level in the lip is very much needed, to avoid loss of extra water.

The crest is usually made circular. A large radius gives good hydraulic performance, but a sharp radius improves the priming capabilities of the siphon. Therefore, a good design should make a compromise between them.

The width of the throat is governed by structural design because the vacuum created in the throat result in considerable external pressure during siphon running. Therefore, span of the opening is restricted to avoid excessive bending stresses.

The hydraulic design of siphon will be done and notation given in Figure below will be used.

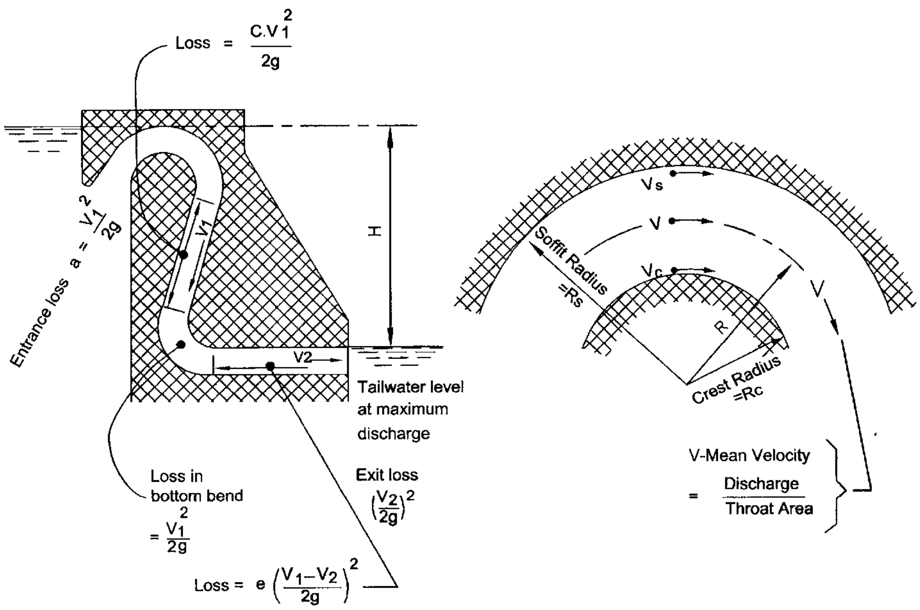


Figure 1-41 Siphon Spillway.

The gross head may be calculated by using Bernoulli equation

$$H = (V1^2/2g)\ (a + b + c + d + e + f)$$

or

$$H = K\ (V_1^2/2g)$$

Where

a, b, c, d, e are the coefficient for bend and friction as indicated in figure.
e and f depend on the ratio of V_1/V_2

The determination of these is not possible, however for preliminary design there value such as a = c = 0.1, b = d = 0.3 and e = 0.2 can be used. The value of K ranges between 0.7o to 0.95 and it depends upon the throat area and may be as low as 0.40 and may go upto 1 if exit area is more than throat area.

The velocity near crest can be found if maximum permissible vacuum is known. Let it be h. Then

$$V_c = \sqrt{2gh}$$

The velocity near soffit then may be

$$V_s = \left(\frac{R_c}{R_s}\right)V_c = \sqrt{2gh}\left(\frac{R_c}{R_s}\right)$$

The pressure head at soffit may be

$$\frac{V_s}{2g} = \frac{1}{2g}\left(\frac{R_c}{R_s}\right)V_c = \left(\sqrt{2gh}\right)^2\left(\frac{R_c}{R_s}\right)^2 = h\left(\frac{R_c}{R_s}\right)^2$$

The discharge per foot width of the siphon may be

$$Q = \sqrt{2gh} * R_c * \log\left(\frac{R_c}{R_s}\right)$$

The absolute pressure in the siphon should be less than 4 to 6 lb/in^2 in order to avoid unsatisfactory flow condition and the release of dissolved air. The maximum vacuum that may be caused is about 30 ft. The degree of vacuum at the siphon throat depends upon the vertical height of the outlet leg which is normally equal to the head. In medium head to avoid this undesirable vacuum, the siphon exit may be designed convergent throughout its length.

In low-head plant siphon is usually made with divergent exit to achieve the best efficiency, because unless exit velocity is reduced too much of the available head is absorbed in the kinetic energy at the outlet/exit. In order to induce better priming, however, it is usual for the outlet leg to have a vertical or over-hanging upper portion, followed by a right-angle bend and a divergent exist whose optimum angle of divergence is about 8^0 30’.

1.6.1.3 BOTTOM OUTLET

The bottom outlet are provided as low level sediment flushing conduits in medium and high-head power plants. But in low-head power plants bottom outlets are used as emergency spilling structures. To use them as emergency-relieving structure they are equipped with hydraulically operated gates which will be controlled from main powerhouse control room along with the control of turbine unit.

The hydraulic design of the bottom outlet may be carried out such as, suppose an orifice of circular area A is discharging in to atmosphere. According to Bernoulli Theorem the velocity of water through the orifice is as

$$V = \sqrt{2gh}$$

More exactly

$$V = c_v \left(\sqrt{2gh}\right)$$

Where
c_v = Coefficient of velocity

But if contraction is also counted then

$$V = c_c\, c_v \left(\sqrt{2gh}\right)$$

Then discharge is

$$Q = AV = A * c_c\, c_v \left(\sqrt{2gh}\right)$$

Or

$$Q = A * c_d \left(\sqrt{2gh}\right)$$

Where
c_d = Coefficient of discharge

The coefficient of discharge for well-rounded orifice is equal to 0.98. A rectangular sharp-edge orifice has rather smaller value than square orifice. The difference may be 2 % for a 4:1 ratio of sides and 4 % for ratio of 12:1. If the orifice is submerged then the discharge through the orifice remain the same as in case of orifice discharging into atmosphere.

As it is clear that bottom outlet is not the true representation of the orifice, then the loss in head due to other factors such as bend, friction should also be counted. The discharge through bottom outlet becomes:

$$Q = A * c_d \left(\sqrt{2gh_n}\right)$$

Where
h_n = Net head

1.7 DEWATERING OF POWERHOUSE PIT
1.7.1 GENERAL
For construction of foundation slab of the powerhouse, the ground water level of the construction area, which is normally close to ground surface, will be lowered down to 0.5 m below foundation level. The coefficient of permeability will be established during field investigation. Dewatering can be done by different methods, which totally depend upon the sub-soil conditions. The following methods are more important:

- Deep Tubewell with Cut-off Wall
- Deep Tubewell without Cut-off Wall

- Well points
- Method of Ossmoss.

1.7.2 DEWATERING BY DEEP TUBEWELLS

This method is normally employed when granular soil is available in the foundation.

For inflow computation, the Dupuit Thiem formula is given below

$$Q = \frac{\pi * K * (H^2 - h^2)}{2.3 \log\left(\frac{R}{RE}\right)}$$

Where

R	= Radius of influence	
	= (1-2)e	
e	= Distance between dewatering surface centre to open water surface.	
R_E	= Radius of equivalent well	
H	= Total dewatering head	
	= S+h	
h	= hf + h	
hf	= filter length	
K	= Permeability coefficient	

1.7.3 EXAMPLE - DEWATERING OF GUDDU HYDROPOWER PROJECT PIT.

For construction of foundation slab of the powerhouse, the ground water level of the construction area, which is normally close to ground surface, will be lowered down to 20 m about 0.5 m below foundation level. The coefficient of permeability will be established during field investigation. Dupuit-Thiem formula for computation of pumping discharge is adopted by keeping in view the existing boundary conditions. Because impermeable strata are not available up to greater depth, the dewatering of powerhouse pit will be carried out by deep tube wells.

1.7.3.1 DATA

Ground water level	= 76.0 m.a.s.l
Foundation level	= 54.0 m.a.s.l.
Permeability coefficient	= 1.0 * 10^{-3} m/s
a_1 = larger dimension of Pit Stage I	= 116 + 2 * 40 = 196 m
b_1 = Shorter dimension of Pit Stage I	= 61+50 = 111 m.
a_2 = larger dimension of Pit Stage II	= 116.m
b_2 = Shorter dimension of Pit Stage II	= 61 m.

1.7.3.2 COMPUTATION OF INFLOW

For inflow computation the Dupuit Thiem formula is

$$Q = \frac{\pi * K * (H^2 - h^2)}{2.3 \log\left(\frac{R}{RE}\right)}$$

Where

R	=	Radius of influence
	=	(1-2)e
e	=	Distance between dewatering surface centre to embankment of canal

R_E = Radius of equivalent well
R_E = η * Shorter dimension

$$m = \frac{\text{Longer Dimension}}{\text{Shorter Dimension}} = \frac{116.0}{61.0} = 1.90$$

m = 1.90, for which η = 0.80
R_E = 0.8 X 61.0 = 48.80 m
H = Total dewatering head
= S + h
h = hf + h
hf = filter length
h = 10.50+2.50 = 13.0 m
S = 78.0 -54.0 = 24.0 m
H = 24.0+13.0 = 37.0 m

$$Q = \frac{\pi * 0.001 * \left[(37)^2 - (13)^2\right]}{2.3 \log\left(\frac{1.5e}{48.80}\right)}$$

Qmax = 1.10 Q
Qmax = Increased pumping discharge at first period of dewatering
Qtotal = 1.30 Qmax
Qtotal = 30% increase in Qmax because of partial penetration of wells

em	**Rm**	**Q(m^3/sec)**	**Qt (m^3/sec)**
50	75	8.78	12.56
60	90	6.17	8.82
75	113	4.52	6.46
100	150	3.36	4.81
125	188	2.81	4.01
150	225	2.47	3.53
175	263	2.24	3.21
200	300	2.08	2.97
225	338	1.95	2.79
250	375	1.85	2.65
275	413	1.77	2.53
300	450	1.70	2.43
325	488	1.64	2.35
350	525	1.59	2.27
375	563	1.54	2.21
400	600	1.50	2.15
425	638	1.47	2.10
450	675	1.44	2.06
475	713	1.41	2.01

$$Q = \frac{1.64}{\log\left(\frac{e}{32.53}\right)}$$

The curve drawn between R (radius of influence) and Qt (total pumping discharge) indicates that the safe distance for the diversion of Ghotki Feeder Canal from the powerhouse pit can be taken as 200 m.

R = 1.5x200 = 300.00 m.

$$Q = \frac{1.64}{\log\left(\frac{e}{32.53}\right)} = \frac{1.64}{\log\left(\frac{200}{32.53}\right)} = 2.07\,m^3/s$$

Qmax = 1.10 x Q
= 1.10 x 2.07 = 2.28 m^3/s
QTotal = 1.30 x Qmax
= 1.30 x 2.28 = 2.96 = 3.0 m^3/s.

1.7.3.3 DEWATERING STAGES

Considering the available space for installation of pumps and economic viability of the project. The dewatering will be completed in two stages.

1.7.3.3.1 FIRST STAGE

a_1 = 196.0 m
b_1 = 111.0 m

$$m = \frac{196}{111} = 1.76 \text{ for which } n = 0.75$$

R_E = 0.75x111
= 83.25 m
H1 = S1 + hf + h
= (78-65.0) + 10.5 + 3.0
= 26.50 m

$$Q_1 = \frac{\pi * 0.001 * \left\lfloor (26.50)^2 - (13.50)^2 \right\rfloor}{2.3 \log\left(\frac{300}{83.25}\right)}$$

Q1 = 1.28 m^3/s
Q_{max} = 1.10 x1.28 = 1.41 m^3/s
Q_{Total} = 1.41x1.30 = 1.83 m^3/s

1.7.3.3.1.1 CAPACITY OF ONE WELL

Dia = 800 mm

$$q_1 = \frac{2\pi * h_f * K}{15}$$

$$= \frac{2\pi * 10.50 * 0.001}{15}$$

= 0.055 m^3/s

1.7.3.3.1.2 NUMBER OF WELLS

$$n_1 = \frac{Q_T}{Q_I} = \frac{1.83}{0.055} = 34 \text{ wells}$$

1.7.3.3.1.3 DISTANCE BETWEEN WELLS

$$DWI = \frac{2(a_1 + b_1)}{34}$$

$$= \frac{2(196.0 + 111.0)}{34} = 18 \text{ m}$$

1.7.3.3.1.4 VERIFICATION OF FILTER LENGTH

One lateral flow is considered.

$$q_1 = \frac{Q_{I\,max}}{n_I} = \frac{1.41}{34} = 0.041 \text{ m}^3/\text{s}$$

$$h_f = \frac{h^2 - \frac{1.5 q_I (I_n D_{W1} - \ln r)}{2}}{3.14 * K}$$

$$h_f = \frac{(13.50)^2 - \frac{1.5 * 0.041(I_n 18 * 0.40)}{2}}{3.14 * 0.001}$$

$$= 11.0 > 10.50 \text{ O.K.}$$

1.7.3.3.2 SECOND STAGE

R_E = 48.80 m

Q_{II} = Q_{total} - Q_I total/2 = 3.0 - 1.83/2 = 2.1 m^3/s.

- 50% of the wells from Ist Stage will stop working

1.7.3.3.2.1 CAPACITY OF ONE WELL

Dia = 1000 mm
h_f = 11.0 m
q_{II} = 3.14 x 1.0 x 11.0 0.001/ 15

= 0.073 m^3/s

1.7.3.3.2.2 NUMBER OF WELLS

$$n_1 = \frac{2.10}{0.073} = 29 \text{ wells}$$

1.7.3.3.2.3 DISTANCE BETWEEN WELLS

$$D_{WII} = \frac{2(a_2 + b_2)}{n_{II}}$$

$$D_{WII} = \frac{2(116.0 + 61.0)}{29}$$

$= 12.20$ m

Lateral flow for a well

$$q_{II} = \frac{2.10}{1.30 * 29}$$

$= 0.055\ m^3/s$

1.7.3.3.2.4 VERIFICATION OF FILTER LENGTH

$H = h_f + h$
$= 11.0+3.0 = 14.0$ m
$h_f = ((14.0)^2 - 1.50 \times 0.055 \ln (12.20)/2 \times 0.5) / 3.14 \times 0.001$
$= 11.41 > 11.0$ O.K.

1.7.3.3.3 POWER REQUIRED

1ST STAGE

Hp = Pumping head
= Water Disposal level - Level of central Point of effective filter length+losses
H_{PI} = 80.0 - 56.75 +5.0
= 28.25 m
Q_I = 1.83 m^3/s
np = Pump efficiency = 0.65
P_I = Power required for first Stage Dewatering

2ND STAGE

H_{PII} = 80.0 - 45.50 +5.0
= 39.50 m.

$$P_{II} = \frac{Q_{II} * H_P}{n_p} * 9.81$$

$= 1252.0$ kW

TOTAL POWER REQUIRED

$$P_{Total} = \frac{P_I}{2} + P_{II}$$

$$P_{Total} = \frac{780.23}{2} + 1252.0$$

$= 1642.11$ kW

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