HYDRAULIC AND STRUCTURAL DESIGN OF THE HYDROMET STATION ON SINZA RIVER

By: E.M. Kasenene* and S.V.K. Sarma**

Synopsis

As a follow up of the hydrologic studies conducted within the Hydrology Department where-in the design flood, the diversion spillway and rennovating a low level dam and measuring weir on Sinza River (which was washed away during the 1975 floods) are planned; efforts are made herein to furnish the design particulars for hydraulic and structural components of the said structures. While the criteria for fixing the inverts of weir and spillway design flood were undertaken, under the guidance of Dr. Eric Schiller, the present design is conducted by the final year students of 1977878, under the supervision of the senior author.

Introduction

The Sinza river which lies north of the MAJI, Ubungo complex is a sinous river and it is a tributory of the Msimbazi River system, draining into the Indian Ocean near Ocean Road, Dar es Salaam. There had been a low lying weir with a spillway, together with provision for hydrometric measurements by way of a cable-way upstream of the weir. Because of faulty construction (rectangular structure with shallow foundation and perhaps of weak concrete mix), the whole structure was washed away during the unprecedented torrential 1975 floods. Since then, a survey of the whole area upstream and downstream of the existing (washed off) weir was made to look into the possibility of better selection of site for constructing the proposed measuring weir. Soil investigations were conducted by MAJI, Ubungo, and particulars of subsoil were furnished to this department. Apart from the weir which serves as a practical instruction tool for hydrometric measurements to serve the needs of training water resources technicians as also the Civil Engineering Students of the University of Dar es Salaam, the necessity of of a spillway upstream of the weir (see figure 1) was conceived, which would take care of the excess flood during the rainy season and which saves the premises of the Water Resources Institute from getting flooded, when the river flows over its banks. This requires river training, to lead the excess discharge through a straight path from sec. 11 which joins the main river at sec.55.

Design Considerations

The meandering reach of the channel was split into 4 parts for convenience of back water computations. The roughness coefficient s were estimated from Chow's Handbook and actual field conditions by reconnoitering the whole reach of about 200 metres.

Subreach	Length m	Manning's n	long slope
1-2	61.83	0.04	0.004
2-3	38.07	0.08	0.009
3-4	45.00	0.150	0.004

With cross-sections as adopted in figure 1, results of back water calculations have yielded the following:-

* Hydrologist, Ministry of Water, Energy and Minerals, Dar es Salaam.

** Associate Professor, Civil Engineering Department, University of Dar es Salaam

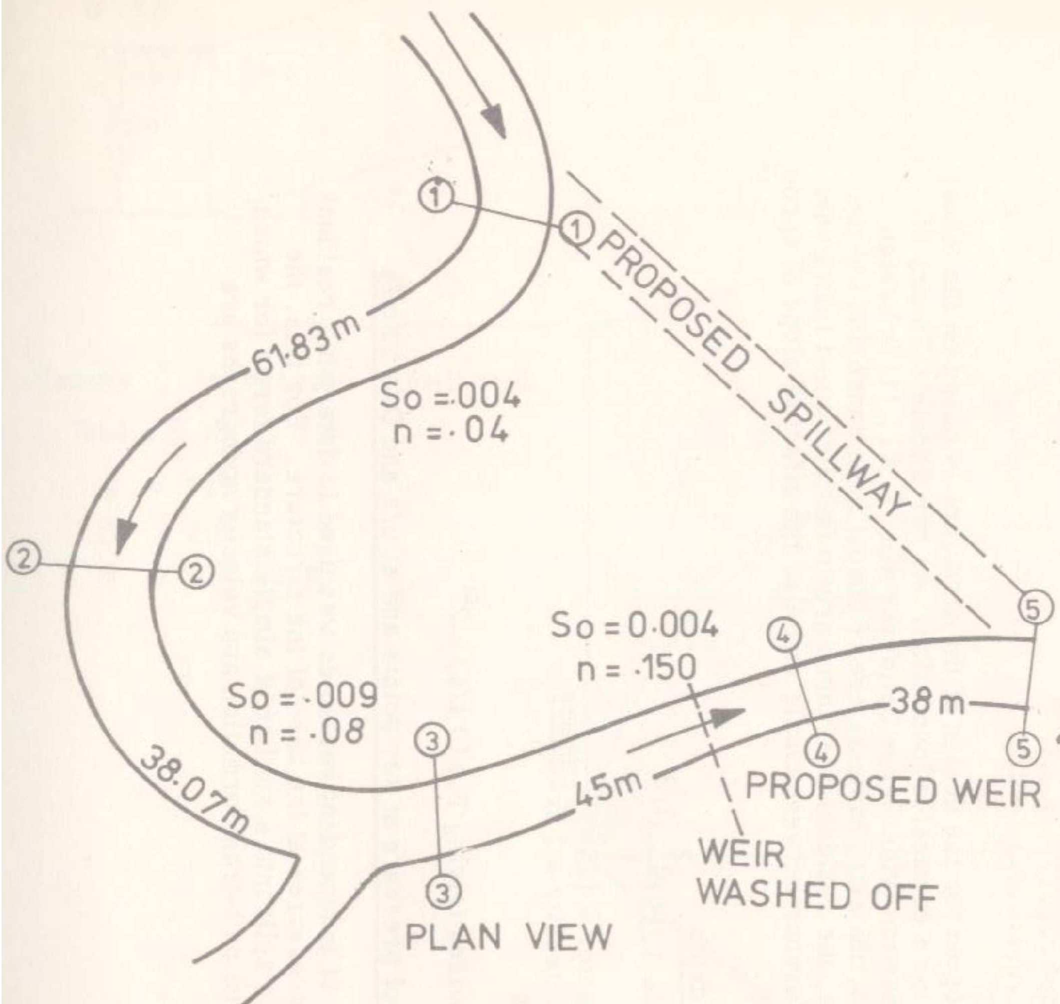


Fig. 1 PROJECT SKETCH

Showing the meandering reach of SINZA RIVER UNDER CONSIDERATION

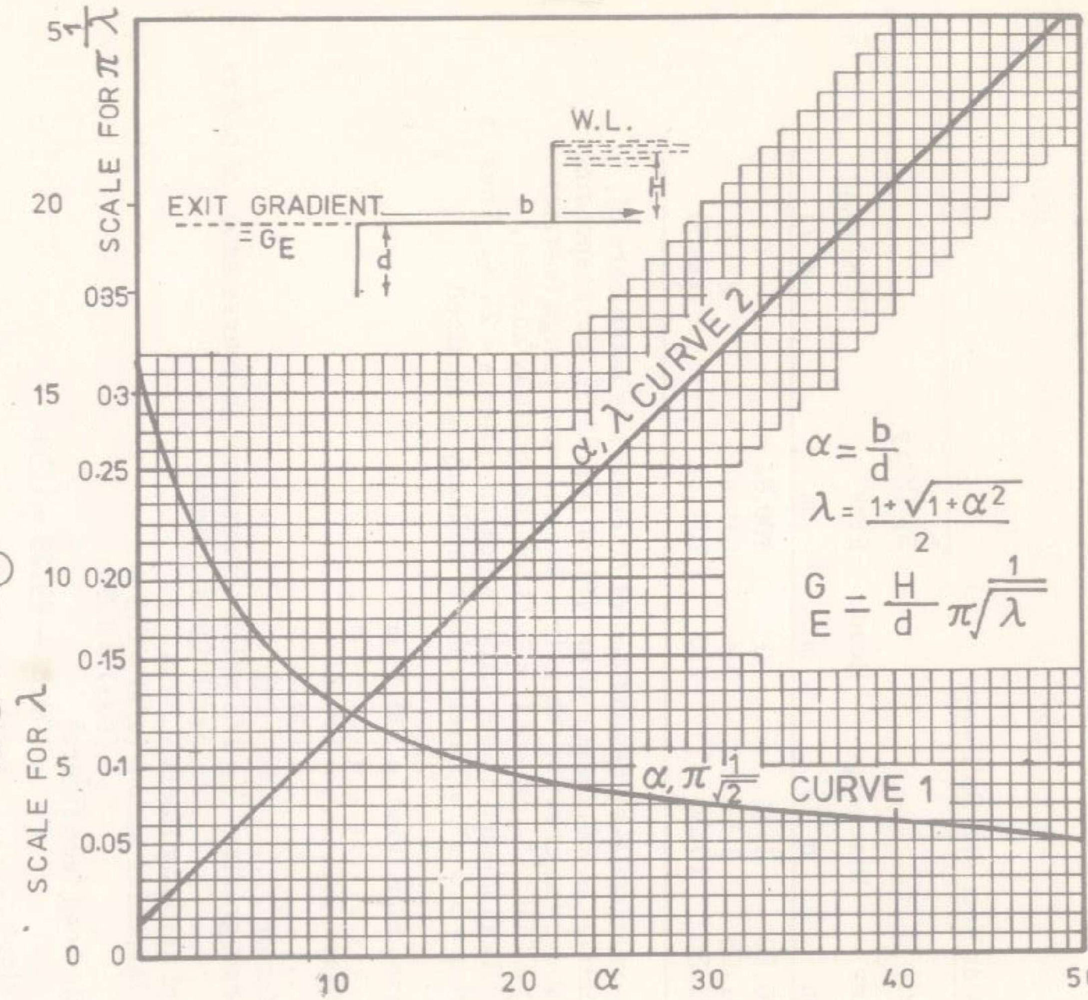


Fig. 2a

Drawn/SFM

Water Levels at Weir and Spillway

Head	Weir Elev^n	Discharge	Head	Spillway Elev^n	Discharge
m	m	m^3/s	m	m	m^3/s
0.5	100.80	0.24	0	100.84	0
0.75	101.05	3.84	0.6	101.44	20.73
1.00	101.30	10.36	1.0	101.84	44.60

Thus, the 50-year spillway designflood of 40 cumecs is taken care of, in view of its accommodating 44.60 cumecs at an MFL of 101.84 El. corresponding to 1 m of flow depth above the crest of the weir. The relevant design (metric) equation used was $Q = CLH^{3/2}$, where C = 2.23 for the spillway adopted. Minimum width of spillway needed as per Lacey's design criteria = 24 m. Thus, a discharge intensity of $q = 1.6\ m^3/s$/meter width of spillway is obtained

D/s W.L. during high flood = 100.75 m
u/s .... = 101.84 m
crest of spillway .... = 100.84
u/s bedlevel .... = 100.60
Lacey's silt factor .... = 1.5
Permissible exit gradient = 1/5 which corresponds to course sand of bed as per Bligh's criteria
Total length of spillway floor= 9.52 m
Thickness of launching apron= (u/s) = 0.9 m
(D/s) = 0.8 m
Maximum static head (see figure)
= 100.84 - 99.25 = 1.59
u/s spillway slope .... = 1:1
D/s .... = 2:1

The criteria adopted for the design of the structure is based on the theory that the weir is resting on a permeable foundation. As per Khosla's Theory of weirs on permiable foundation (vide Serge Leliavsky-vol. I, II, III. Design of Hydraulics structures), the exit gradients are of utmost importance and for the cut offs, at u/s and D/s, the residual pressures are to be determined taking into consideration the interference between cuoffs and also the effect of slope of apron on the pressure gradient.

Determination of scour depth

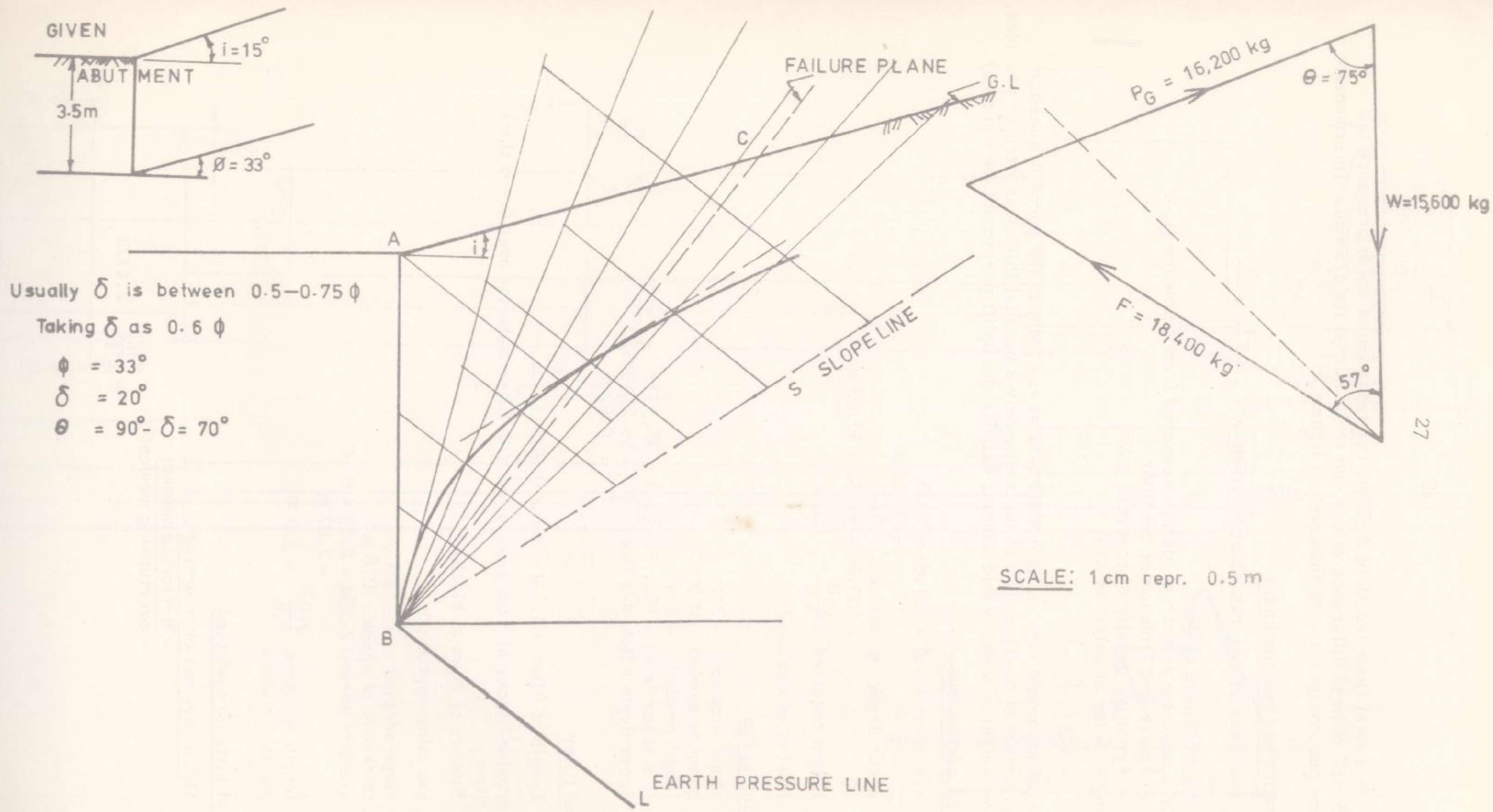
Scour depth $D = 1.35 \left(\frac{q^2}{f}\right)^{1/3}$

where f = Silt factor = 1.5
q = flow intensity = 1.6 $\frac{\text{cumecs}}{\text{m}}$
= 1.61^m

scour depth provided (giving F.s (1 1/4) = 2^m

Computation of percent of pressure at key points and at u/s and D/s cutoffs

Khosla's theory of independent variables was used to determine residual pressures i.e. uplift pressures at the base of the structure. For this, the section of the spillway is split into a number of simple standard forms for which analytical solutions based on t-transformation and velocity hodographs are available (Figure 2a).



COLUMAN'S GRAPHICAL SOLUTION TO COULOMB'S WEDGE THEORY

In a total floor length of 8.52 m, the variation of uplift pressure is from 28.62% of head difference at u/s end to 78.80% at its d/s end. Intermediate pressure percentages are as indicated in figure 2.

Computation of floor thickness

thickness of concrete floor is given by $t = \frac{h' - t}{s-1} = \frac{h}{s-1}$

where t = thickness of floor
h′ = ordinate of hydr-gradeline measured from below the floor
h = measured from top of the floor
s = relative density of concrete = 25
providing a factor of safety of 4/3,
we have $t = 4/3\ (\frac{h}{s-1})$
which equation would furnish floor thickness at any point of the apron in question.
For the portion of the floor u/s of the weir, only a nominal thickness need be provided, since the weight of water would counter balance the uplift pressure (see figure 2).

Upstream protection

u/s scour level = 101.84 - 2.00
= 99.84 m
scour depth 'd' below u/s floor
= 100.60 - 99.84 = 30.80 m
length required $= \frac{0.8}{0.9} = 1.0$ m
number of rows = 2

Launching apron

slope adopted = 2:1
quantity needed = 0.9 m^3
length needed = 2 m
D/s scour level = 100.75 - 2.42 = 98.33 m
scour depth below d/s floor = 99.25-98.33
= 0.92 m

Inverted filter

Length of filter = 1.5 d = 1.5 x 0.92 = 1.3 m

Thus, provide blocks at 5 cm gaps filled with filter material over 0.4 m thick gravel filter.

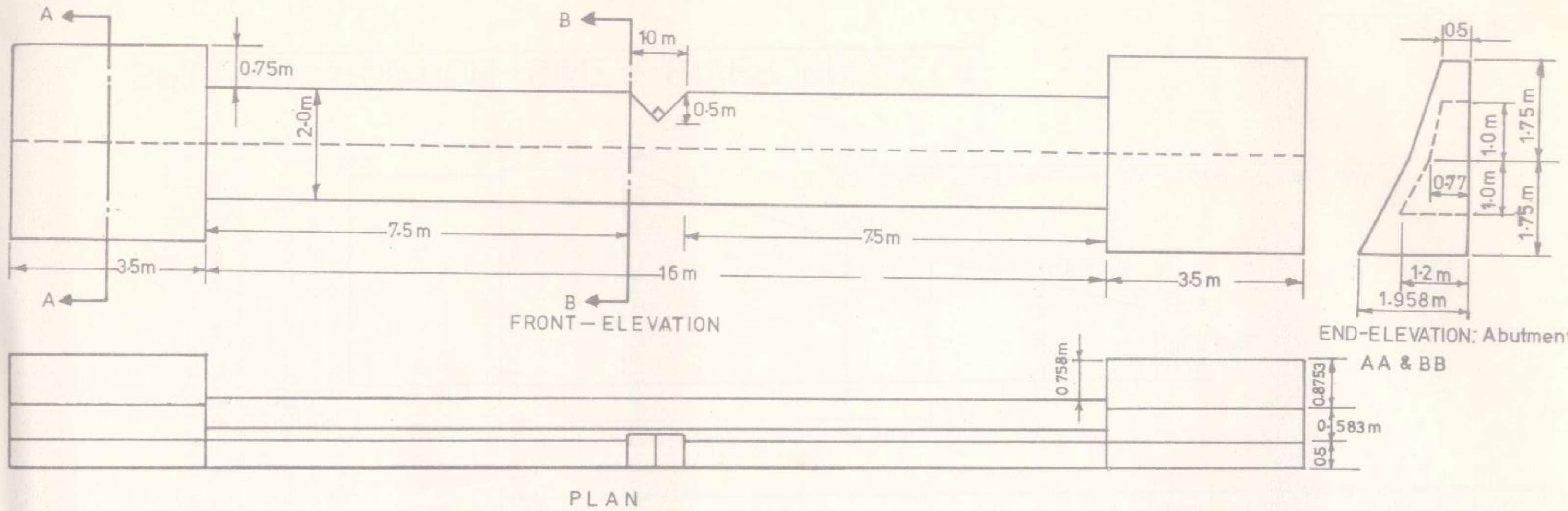
Number of rows of blocks = 2

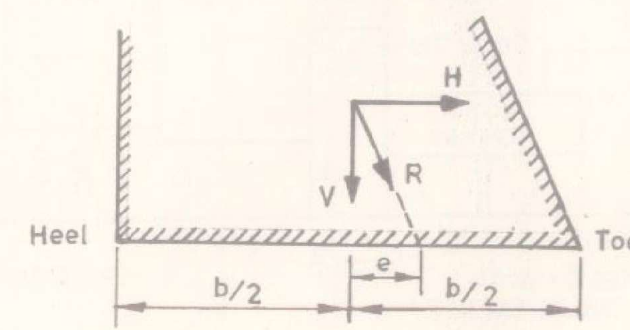
Lanching appron (D/s)

slope adopted 2:1
thickness of apron 0.8 m
quantity needed 2.25d = 2.25 x 0.92
= 2.07 m
length of apron per meter width $\frac{2.07}{0.8} = 3.0$ m

Safety of structure adopted

a) f.s. against overturning
= Restoring moment / overturning moment
$= \frac{8943}{3710}$ 2.4172

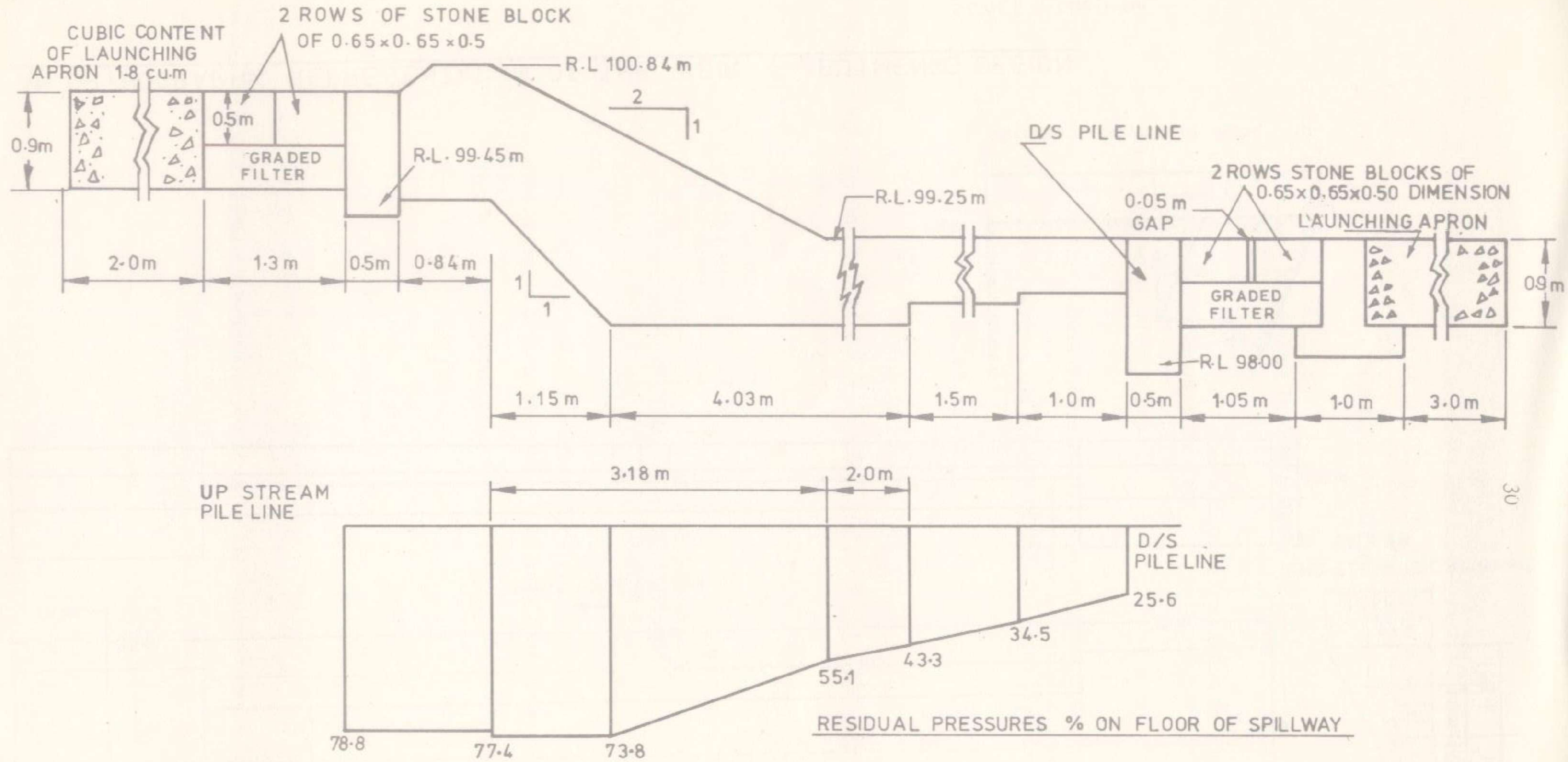




Stress Analysis in dams

AN ORTHOGRAPHIC REPRESENTATION OF THE WEIR & ABUTMENTS DESIGN

SCALE = 1cm = 1m



SPILLWAY PORTION–SINZA RIVER PROJECT

Scale 1cm : 50cm

Drawn MAPUNDA, S.F.

b) f.s. against sliding

$$= \frac{(w-u)}{H} = 0.7\frac{(9710 - 1713)}{1245} = 4.49 > 1.0$$

c) Shear friction factor

$$= \frac{f(W-u) + r S_a A}{H}$$

$$= \frac{0.7\ (9710-1713) + 0.5 \times 4 \times 10 \times 1.958}{1244}$$

$$= 22. > 4\text{-}5$$

thus safe against shear

d) Normal stress at heel $= \frac{v}{b}(1 - \frac{6e}{b}) = 4959\ (\frac{1-6 \times 0.058}{1.958}) = 4078\ kg/m^2$

Normal stress at toe $= 5840\ kg/m^2$

As the eccentricity is 0.058 m, (figure 3) the structure is free from tension. In a similar way, the weir also is checked for safety against disturbing forces and moments and is found to be generally safe.

Bearing capacity of foundation

Using Terzaghi's equation for arriving at B.C. of foundation

$$B.C. = \bar{B}(CN'_c + \frac{B}{2}\gamma_c\ N'_s + N'_q)$$

where

$\bar{B} = B - 2e$; B = base width, m

e = Eccentricity, m

Nc, Nj, Nq = B.C. factors being.16,22 abd 22 respectively for $\theta = 7.3^o$ (from standard graphs)

$$\bar{B} = 1.958 - 2 \times 0.058 = 1.842\ m$$

$$q = \frac{R}{B} = \frac{9789}{1.98} = 5000\ kg/m^2$$

B.C. = 41200 kg/m^2, while the limiting stress of soil is 20,000 kg/m^2 thus showing that the soil can stand to the conditions imposed.

Check against shear failure by Wedge Theory

Analysis of the abutments, on either side of the structure by Culmann's Wedge theory revealed that the weight of wedges was 15600 kg, earth pressure was 16200 kg and resultant force was 18400 kg, as shown in sketch (figure 4).

With a permissible pressure of 20,000 kg, it can be said that the structure is safe from failure due to passive earth pressure from the overburden.

Conclusion

1. An alternative design is suggested for replacing the weir near Maji Ubungo, which facilitates low flows to be taken care of by the weir at site, while during floods, major part of flows is diverted over a spillway located at an upstream site.

2. The design consideration for the weir and spillway are firmly based as per standard practices for weirs on permeable foundations and the proposed structure is safe against exit gradients, as also from structural stability against sliding, overturning and shear friction.

References

1. Muscat, M. - Flow of homogeneous fluids through porous media, McGraw Hill, N.Y.; 1937.

2. Poluborinova Kochina, P.Y. - Theory of groundwater movement-translated by Roger de Weist from Russian to English, Princeton University Press, 1962.

3. Schiller, E.J. - Hydrologic design for Sinza River Project, paper accepted for publication in process of East African Academy 13th Annual Conference held at Nairobi, Kenya September 13th to 17th , 1977.

4. Khosla, A.N.; Bose, N.K. & Taylor, E.M. - Design of weirs on permeable foundations, New Delhi, 1934, June, Central Board of Irrigation and Power.

5. Aravin, V.I. & Numerov, S.N. - Theory of fluid flow in underformable porous media, translated from Russian, by Israelli Program for Scientific Translations, Jerusalem. Israel , 1965 .

6. Pavlorsky, N.N. - The theory of movement of ground water under hydraulic structures, Gosstriidet, U.S.S.R., 1948.

7. Milton E. Harr, Ground water & Seepage, McGraw Hill, New York, 1962.