ENGINEER - Vol. XXXXV, No. 03, pp. [11 - 20], 2012

Treatment of Seepage through Vendarasankulum Twin Reservoir in Eastern Sri Lanka - Cost Comparison of Alternative Techniques

D. A. R. Dolage, S. P. P. Gamage and T.V.K.I.S. Karunasena

Abstract: The purpose of this study is to select a more effective technique from 'upstream cutoff' and 'grouting treatment' for the control of seepage through the Vendarasankulum reservoir. Using engineering judgment, empirical and theoretical knowledge, trapezoidal cross sectional dimensions of layers of different soil materials were determined. For example, the cross section at 490m (from left); SC layer on the U/S side slope has widths, 3m and 12m at bund top and bottom respectively; clay layer in the core has widths 12 m and 3 m at the top and bottom respectively. The effective fetch (1.72 km) and thereby the maximum wave height (0.84m) were computed in order to extract values for thicknesses of riprap (0.45 m) and bedding layer (0.30m). Grouting treatment involves clay-cement grouting in the overburden and 6m deep cement grouting in the rock, requiring a total of 296 grout holes. While the estimated cost of construction of the upstream cutoff is SLR 30.3 million, the same for grouting treatment is SLR 11.3 million, indicating the former option would cost 168 percent more. Therefore, grouting treatment is more economical and sufficient, although upstream cutoff could offer better seepage control.
.

Keywords: Seepage, Earthen embankments, Upstream cutoff, Grouting

1. Introduction

Vendarasankulum reservoir is an ancient earthen reservoir located in Kantale in the Eastern Province. This reservoir is plagued with a severe seepage problem occurring through the bund and the foundation, which has drawn the immediate attention of the Irrigation Department of Sri Lanka (IDSL). The sub surface exploratory studies conducted by the IDSL revealed that the material the bund is composed of is permeable and the underlying rock formation is highly fractured. Although IDSL has tried out several methods at different stages to control seepage ever since it was detected the seepage continues unabated. This situation has necessitated treatment, with IDSL having two alternative treatments at its disposal; upstream cutoff and grouting.

The research reported here is on the economic evaluation of upstream cutoff and grouting. Essentially, this study is aimed at ascertaining the most cost effective technique to mitigate seepage.

The strike direction of the rock is N 30^0 -35^0 W dipping vertical. Rock fractured into two directions, one along the foliated plane and the other in W 15^0-25^0 S dipping vertical. The capacity of the reservoir is 20,240 acre feet at Full supply level (FSL), 39.5 ft above MSL, and the command area 1606 acres. It has no catchment of its own but receives water from Kantale reservoir, a much larger reservoir, and functions more as a balancing reservoir; see Figure 1 for geographic location map. The cross section of the earth embankment of Vendarasankulum reservoir is trapezoidal and has parameters as given in Table 1.

Table 1 - Parameters of earth embankment

Characteristics	Value (m)
Length	700
Average height	18
Crest width	6
Base width	96

Eng. (Dr.) ***D. A. R. Dolage,*** *CEng, FIE(Sri Lanka), BSc Eng. (Moratuwa), MSc (Reading), MA (Colombo), MBA (SJP), DBA (UniSA), Senior Lecturer, Department of Civil Engineering, The Open University of Sri Lanka.*
Eng. S. P. P. Gamage, *CEng, MIE(Sri Lanka), BSc Eng. (Peradeniya), MSc.Eng.Geology (Brussels), Deputy Director (Engineering Geology), Irrigation Department of Sri Lanka.*
Eng.T.V.K.I.S. Karunasena, *BTech (Eng) (Hons) (OUSL), Civil Engineer, National Water Supply and Drainage Board.*

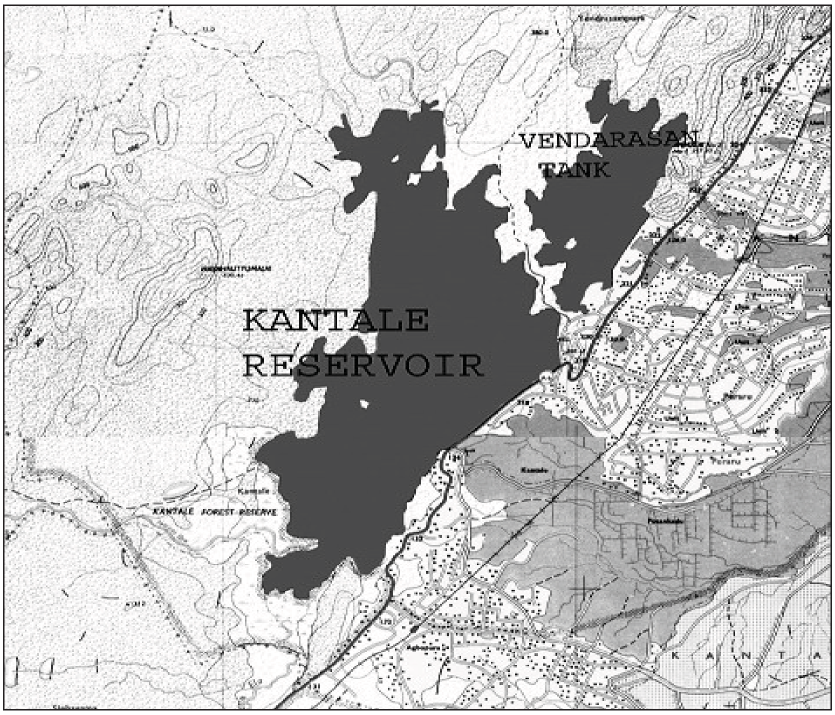

Figure 1 - Location map of Kantale and Vendarasankulum reservoirs

2. Literature Review

2.1 Seepage in earthen bunds

Earthen reservoirs have proved to be a sustainable solution to the requirement of storing water for the purpose of irrigating farms in the dry zone. In the provinces which experience little rainfall, acute shortage of water is a major concern and so reservoirs play a vital role in supplementing the other sources of water. According to Malkawi and Al-Sheriadeh [1], there are many hydrogelogical, geological and geotechnical problems associated with dams and reservoirs. Among them seepage, is a major concern because it reduces dam's storage capacity and may cause unforeseen failures.

The issue of seepage has drawn the attention of many researchers during the past decades. For example, Uromeihy and Barzegari [2] evaluate the treatment of seepage problem with Chapar-Abad Dam in Iran; Turkmen [3] examines the seepage problem with the Kalecik dam in Turkey; Ahmad and Shafiq [4] evaluate the effectiveness of different methods namely chemical, physical and biological, in mitigating seepage.

Uromeihy and Barzegari [2] further say controlling the quantity of seepage that occurs after construction is difficult and quite expensive. The same study reveals the following methods to control seepage; upstream blanket, cut off wall, grout curtain. Basak [5] identifies seepage failure as one of the three major causes of failure of earthen dams. Also, the author presents two common causes for the seepage failure:

Piping or undermining - Due to the continuous seepage flow through the embankment and through the sub soil below the embankment, the downstream side gets eroded or washed out and a hollow pipe like groove is formed which extends gradually towards upstream through the base of the dam.

Sloughing - The crumbling of the toe of the dam is known as sloughing. Due to the force of the seepage water the toe of the dam goes on crumbling gradually.

The processes available to mitigate seepage in earthen dams can be broadly categorized under two: those that keep the water out or reduce the seepage quantities; those that use drainage methods to control the water entering. Mohan Das and Saikia [6] recommend use of soil of low permeability to control seepage. The authors say, although the locally available soil is used for cheap construction, proper selection of material, mixing of different kinds of soil, proper zoning may provide the best combination for seepage control by reducing permeability.

2.2 Reasons for high seepage in Vendarasankulum reservoir

1. The bore hole details show the bund is mainly composed of Silty gravel (GM) instead of Clayey sand (SC). GM is a coarse material containing large particles, which has caused high permeability in the bund.

2. The overburden material in the bund is heterogeneous, having different layers some of which are composed of decomposed rock.

3. The Core recovery (CR) which is an indication of rock quality is very irregular in some bore holes.

4. The Rock Quality Designation (RQD) values are low in all bore holes except for two, which indicate poor quality of rock. Further, the investigation reveals that the rock is fractured and zones of fractures are present.

5. The Lugeon value, a measure of rock permeability is quite high, which indicates that the rock is highly pervious.

2.3 Remedial measures to mitigate seepage in Vendarasankulum reservoir bund

The seepage through the length of the Vendarasankulum reservoir bund had been there for nearly two decades and got worse in 1987. As a remedial measure, in 1989, the IDSL constructed a loading berm which had components; a stabilization fill, a sand blanket and a network of sand drains (finger drains) and a toe drain. In order to control seepage through the bund foundation a strip of sand blanket, merged with finger drains, was placed along the downstream toe of the dam. Finger drains had been constructed underlying the sand blanket to channel the water collected in the sand blanket to the toe drain. About 3 m thick stabilization fill was placed on top of the sand blanket to serve as a surcharge. However, despite the stabilization fill on the Vendarasankulum reservoir bund, the seepage continued and the following observations were subsequently made:

1. Sand washed away from sand blanket found collected in the rubble lined toe drain. At present, it is feared that the existing pot holes would increase in number and in size, which could lead to failure of the bund.
2. Water can be stored only up to 10m level which is far below the FSL.
3. Boggy areas were observed at eight locations on the downstream side of the embanment.
4. Seepage is occurring along the entire length of the embankment (700m).
5. Downstream toe drain had been totally ineffective for a long time. This was hardly visible since the reservation had been encroached by farmers.
6. Apparently due to the failure of the toe filter, sand from the finger drains are being washed away causing sink holes (sink holes are created when bund material escape with seepage water) on the downstream side of the embankment.

This shows the solution of loading berm has failed to remedy the problem and seepage continued almost at the same rate and as a result the reservoir could not be filled with water beyond 60 percent of the capacity. In year 2002, an investigation revealed that there were pot holes in the boggy area, along the D/S stabilization fill, caused by piping; the worst affected segment being the segment between chainages 525m and 575m.

The seepage problem in the Vendarasankulum reservoir continued to be a major concern of the IDSL calling for solutions. In order to identify the cause for high seepage problem, drilling investigation was carried out towards the end of 2003. Drilling had been carried out between chainages 470 m and 590 m. Eight bore holes have been drilled in the bund, namely BH1 to BH8, and details are depicted in Table 2. The IDSL carried out mitigative grouting work in two stages (as per Figure 2); Stage 1, treatment of overburden; Stage 2, treatment of rocky foundation.

Table 2 - Geological investigations along bund

Bore hole Location (in meters)	Distance from centre line of bund	Level on bore hole top (from MSL)	Depth in meters	Material of bottom
BH1-485	6.2 (U/S)	55.17	27.6	Rock
BH2-520	3.0 (BTL)	56.67	35.43	Rock
BH3-551	18.0 (U/S)	52.84	25.55	Rock
BH4-560	2.3 (BTL)	56.94	24.2	Rock
BH5-563	3.0 (BTL)	56.82	10.0	Over burden
BH6-564	23.0 (D/S)	48.94	14.7	Rock
BH7-563	43.0 (D/S)	43.37	12.1	Rock
BH8-513	28.0 (D/S)	46.67	15.0	Rock

LEGEND: BH - Bore hole, U/S - Upstream, D/S - Downstream, BTL - Bund top level
Source: Sol Investigation Report- IDSL

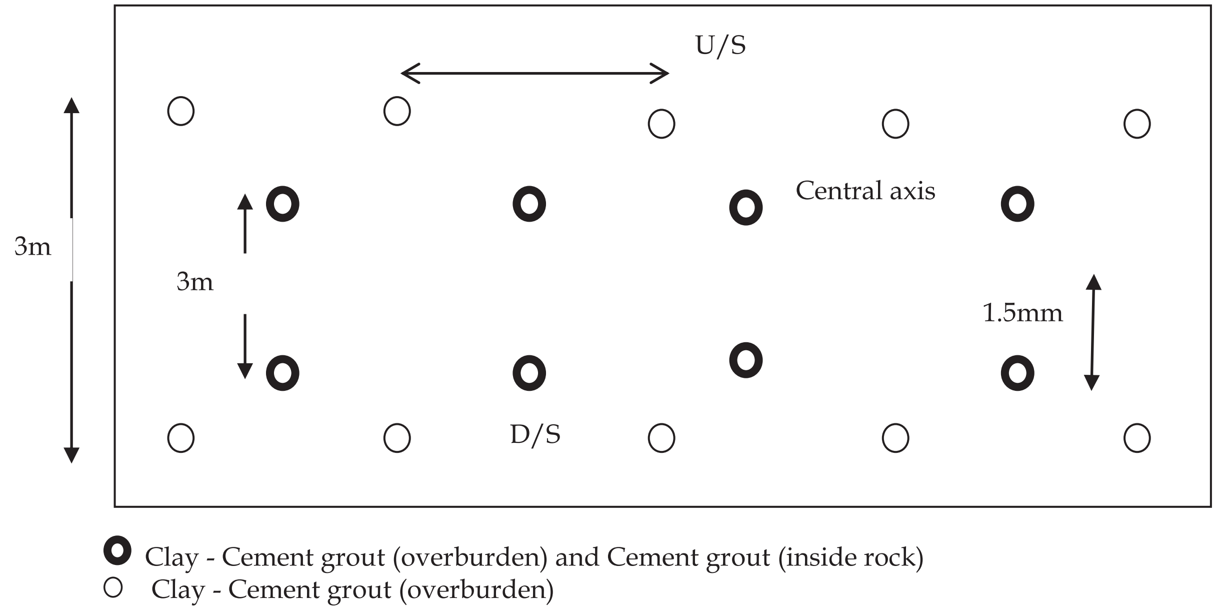

Figure 2 - Plan view of grout hole arrangement at bund top level

Stage 1 - Treatment of overburden using clay-cement grouting

Thirty seven holes (N=91.82 mm) were drilled in the overburden in two rows parallel to each other, each row being placed 3 m away from the centre line of the bund top. The grout used had the water: clay: cement ratio of 20:10:1, by volume.

Stage 2 - Treatment of rocky foundation using cement grouting

Grouting in the overburden as well as the rocky foundation resumed after a lapse of two months from the completion of Stage 1, in 2007 in the same dam stretch. Thirty six holes were drilled in two parallel lines on the bund top but this time each row being 1.5m away from the dam axis. The same mix ratio adopted in Stage 1 was used for grouting in the overburden but cement grout was used for the rocky foundation. At the beginning, a thinner grout of water/cement ratio of 7:1 was used but gradually the grout was made thicker by reducing the water content. Accordingly, two types of mixtures were used; 5:1 and 3:1.

2.4 Present Situation and objectives of the study

With the mitigative grouting work completed in the 50m stretch of the dam bund, in two stages, the IDSL has been able to reduce seepage considerably, particularly in this section. As literature indicated earlier, upstream cutoff which could contain seepage to a higher degree than grouting, seemed a viable alternative, despite ensuing higher cost of construction. The policy makers are interested in knowing how much more the upstream cut off would cost than grouting treatment. This would enable them to ascertain whether the additional cost incurred could outweigh the superior seepage control afforded.

Therefore, the broad objectives of the study are:

1. To evaluate the cost of construction of a upstream cutoff for the Vendarasankulum reservoir.
2. To investigate the cost effectiveness of grouting as a remedial measure compared to upstream cut off to mitigate seepage in the Vendarasankulum reservoir.

3. Methodology

3.1 Evaluation of cost of construction of an upstream cutoff

The rubble riprap is the most common slope protection method adopted in embankments of earthen dams in Sri Lanka [7]. This is placed on a bedding layer and when an upstream cutoff is constructed both riprap and bedding layers have to be removed along with the materials that become loose (loose material). The bulk of cost incurred in construction included the costs of removing these three layers and placement of the three layers; SC, bedding and riprap.

In order to compute their costs, volumes of material to be removed and placed should be known. Hence, designing the upstream cutoff is a vital task which involves determination of dimensions of the cross sections and parameters of a suitable soil material. Also it is essential to design the rubble riprap, which calls for the determination of wave height. The wave height is determined according to the Savilie formula (shown in section 4.2)[7]. Once the wave height is known, from Table 1 of Technical Guideline for Irrigation published by the IDSL, the layer thicknesses of riprap and bedding as well as the average rock size of riprap can be extracted [7].

The main elements of cost of construction of upstream cut off are given below:

1. Removal of existing riprap, bedding layer and loose material
2. Coffer dam construction
3. Excavation of upstream cutoff trench
4. Supplying of clay material, placement and compaction
5. Supplying of SC material, placement and compaction
6. Supplying of gravel, placement and compaction
7. Placement of riprap
8. Placement of extra rubble required for the new riprap

3.2 Evaluation of cost of grouting treatment

The IDSL in a separate study has identified a suitable clay type and a source for the continuation of the grouting in a dam stretch of 400m. It is assumed that grouting will be executed in the same way it was done for the initial 50m. Therefore, cost of supplying of clay material could be worked out objectively. Since the reservoir is located in the Eastern province the rates applicable was obtained from the IDSL provincial office for both cost estimations.

4. Results and Discussion

4.1 Evaluation of cost of construction of upstream cutoff

4.1.1 Design of SC Layer and core trench

The height of the dam varies in the longitudinal direction as follows: at 0m, height =15m; at 490m, height =18m and at 700m, height =15m. A specimen calculation for dam height of = 18m is shown below. According to Ponrajah[8], when homogenous material (semi-impervious SC type soil) is used for bund height over 6.1m, U/S slope should be at least 1 on 2.5. Figure 3 depicts a typical cross section of an upstream cut off.

Slope of the bund = 1 on 2.5
Height of dam += D_1
Depth of core trench (up to rock level) = D_2
Minimum width of SC layer placed on U/S slope of bund = 3m
Width of SC layer at ground level = L_1
Slope of the SC layer placed on U/S slope of bund = 1 on 3
Depth of core trench (up to rock level) = 7.73m
Minimum bed width of core trench = 3m
Slope length of SC layer = L_2

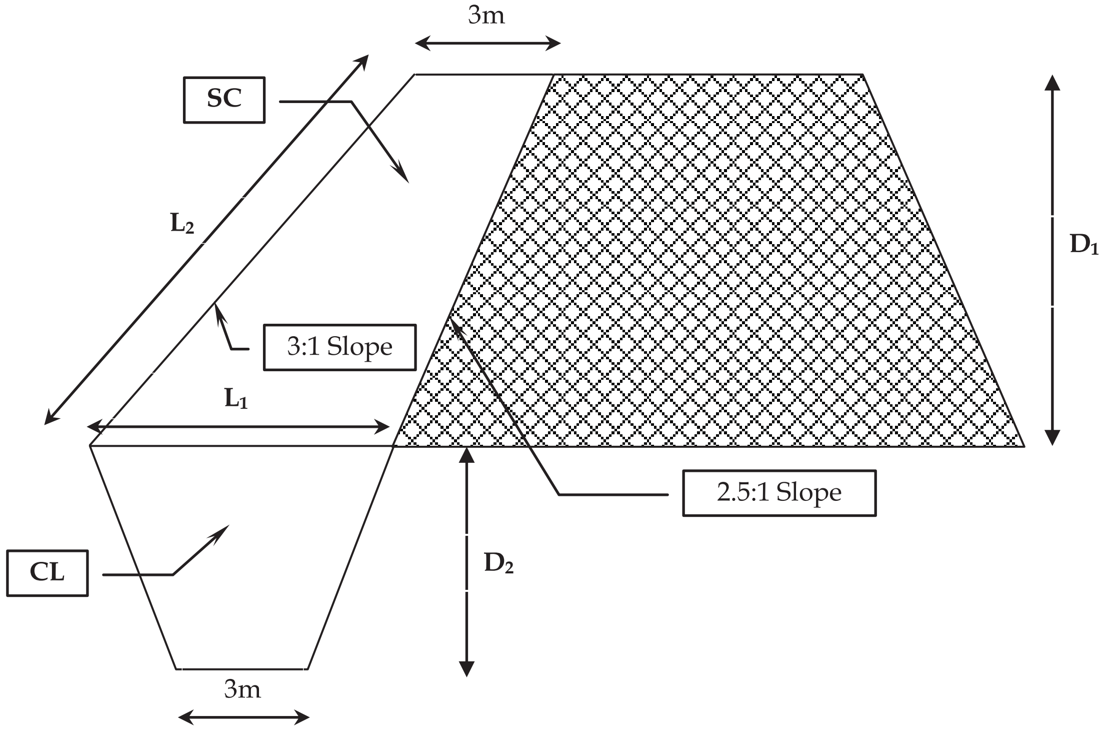

Figure 3 - Typical cross section of upstream cutoff

Since dam height is varying at three places along the axis, the cross sectional area would also vary. Therefore, the volume computation which is the average cross section area multiplied by the distance over which height of the dam, is a variant. Table 3 displays the total volume of the SC layer.

Table 3 - Cross Sectional area and volume of SC layer

Cross Sectional area (m^2)			Volume (m^3)		
at 0	at 490	at 700	between 0-490	between 490-700	Total
101.25	135	101.25	57,883.70	24,807.3	82,691.0

The dimensions of the core trench at: C/S 490, depth 7.73m and top width 12m; C/S 0 and 700, depth 3m and top width 10.5m. The total volume of core trench is depicted in Table 4.

Table 4 - Cross Sectional areas and volumes of core trench

Cross Sectional area (m^2)			Volume (m^3)		
at 0	at 490	at 700	between 0-490	between 490-700	Total
13.50	57.98	20.25	15,871	8,215	24,086

4.1.2 Design of Slope Protection Layer

The primary purpose of U/S slope protection layer of an embankment is to prevent the US slope from erosion and damage from wave action [7]. Apart from the High Flood Level contour of the Vendarasankulum reservoir the data appearing in Table 5 are required for the design of slope protection layer.

Table 5 - Required data for design of slope protection layer

Required data	Value
High Flood Level	185 MSL
Wind velocity	60 mph (88 fps)
Specific gravity of rubble	2.2
Unit weight of rubble	1602 kg/cu.m
Side slope of bund	1 on 2.5

Effective Fetch Calculation for Trial 1and 2

In the Savilie formula (given below) 'f' is the fetch which is the horizontal distance in the direction of wind over which the wind blows. In Trial 1, on the HFL contour, the longest central line was drawn and for the given α angles $f_1$values were measured and thereby $f_1 \text{Cos}\alpha$ were computed and displayed in Table 6. Similarly, in Trial 2, another central radial line was drawn, the process repeated, and the respective values displayed in the same table. The scale of the map is 1 inch to 1,056 feet.

Table 6 - Effective fetch calculation for trial 1 and 2

α	Cos α	Trial 1		Trial 2	
		f_1(cm)	f_1 Cos α	f_2(cm)	f_2× Cos α
45	0.7071	7.3	5.1618	8.7	6.1518
37 ½	0.7934	7.7	6.1092	9.6	7.6166
30	0.8660	10.7	9.2662	10.4	9.0064
22 ½	0.9239	10.6	9.7933	13.5	12.4727
15	0.9659	10.5	10.1419	12.5	12.0738
7 ½	0.9914	11.3	11.2028	13.3	13.1856
0	1.0000	15.2	15.2000	20.8	20.80
7 ½	0.9914	13.4	13.2848	20.2	20.0263
15	0.9659	12.0	11.5908	15.8	15.2612
22 ½	0.9239	11.3	10.4401	14.5	13.3966
30	0.8660	10.3	8.9198	12.4	10.7384
37 ½	0.7934	11.8	9.3621	10.9	8.6481
45	0.7071	9.5	6.7174	9.4	6.6467
∑	11.4954		127.1902		156.0242

Trial 1

$$EffectiveFetch = \frac{\sum f_1 Cos\alpha\, x \frac{1}{2.54} x 1{,}056 x \frac{1}{5280}}{\sum Cos\alpha} = 0.871 miles$$

Trial 2

$$EffectiveFetch = \frac{\sum f_2 Cos\alpha\, x \frac{1}{2.54} x 1{,}056 x \frac{1}{5280}}{\sum Cos\alpha} = 1.069 miles$$

Since Trial 2 results produces a larger value for effective fetch, the effective fetch for the reservoir is 1.069 miles (1.72km).

Determination of Wave Height

The wave height H is determined by the Savilie formula $H = \dfrac{U^2 x 0.0026 x \left(\dfrac{gf}{U^2}\right)^{0.47}}{g}$

Where U=wind velocity (88 rps), f = effective fetch, g = acceleration due to gravity (32.2 f/s²). After substituting the above values in Savilie formula,

$$H = \frac{88^2 x 0.0026 x \left(\dfrac{32.2 x 1.069 x 5280}{88^2}\right)^{0.47}}{32.2}$$

$$H = 2.7 feet (in meters\ 0.840)$$

When the maximum wave height is 0.840m, the following parameters required for the riprap design can be extracted from the Technical Guideline for Irrigation published by IDSL [7].

Average rock size [D50] = 0.30m

Riprap layer thickness = 0.45m

Bedding layer thickness =0.30m

Loose material layer thickness = 1m

Similar to the way volumes were computed in Table 3 for SC layer and core trench, volumes of material for riprap, bedding layer and loose material were computed and given in Table 7.

Table 7- Cross Sectional areas and volumes of embankment protection layers

Layer	Cross Sectional area (m²)		Volume (m³)		
	at 0 and 700	at 490	between 0-490	between 490-700	Total
Rip rap	6.75	8.1	3,638	1,559	5,197
Bedding	4.5	5.4	2,426	1,040	3,466
Loose material	15	18	8,085	3,465	11,550

4.1.3 Estimate of construction cost of cofferdam

The excavation of core trench is done in two stages; half length of the bund is first excavated. In order to prevent the entry of water to the trench area, a coffer dam has to be constructed and for this kind of work the cheapest form is the stacking of sand bags. The number of sand bags required and the volume of excavation required can be computed in the following manner:

Perimeter of the coffer dam required = $(2\pi r)/2 = \pi\times(350/2) = 549.78$ m
At least 4 sand bags needs to be stacked vertically to get a sufficiently high coffer dam. The dimensions of a sand bag are; depth 0.76, width 0.46 and depth0.2 m.

Hence, number of sand bags required = (total perimeter/sand bag length) × 4 = (549.78/0.76m) × 4 = 2,886 sand bags. Excavation volume for sand bag placing = perimeter of coffer dam × (total height of sand bags + working space) × (width of sand bag + working space)
Hence, excavation volume for sand bag placing

= 549.78× 1.3× 1 = 714.71 m3

The construction of coffer dam of was worked out to be SLR 360,000. According to engineering judgment, dewatering would take 2 weeks with the use of two 15 cm diameter pumps.

4.1.4 Estimate of construction cost of upstream cutoff

Using the basic rates, specific rates were prepared for the main cost elements of the upstream cutoff, which are displayed in Table 8. The total project cost of the dam length of 700 m is SLR 30,309,326; approximately SLR 30.3 million.

4.2 Evaluation of cost of grouting treatment

4.2.1 Mix proportions of grout

Since the grouting treatment adopted in 2007 for the critical section seemed satisfactory, it is envisaged to continue the same treatment in the rest of the dam bund. The clay-cement grouting is to be done up to the rock through the overburden and cement grouting to a depth of 6m inside the rock. According to the engineering judgment, grouting done within a span of 400m where seepage is occurring (inclusive of the critical section) can accomplish considerable seepage control. The experience gained and the information obtained in the grouting work carried out in the 50m critical section is used to estimate the grouting cost for the 400m length to be done in four stages. Since, 37 grout holes were completed in the 50m critical stretch, the maximum number of grout holes required for 100m critical stretch is 74.

The mix ratio of grout used by the IDSL, in 2007, was based on engineering judgment and theoretical knowledge of clay cement and cement grouting. The mix proportions adopted, by volume, are given below:

Clay cement grouting- water: clay: cement is 20:10:1
Cement grouting-cement: water is, to start with, 1: 7 and thereafter thickens up to 1:1, until it reaches refusal.

4.2.2 Requirement of grouting material

The requirement for the 100 m critical stretch (based on the information available from 50 m critical stretch already treated) is given below:
Clay - 12 cubes
Cement required for the clay: cement grouting - 16 bags
Cement required for the cement grouting - 40 bags

Since rest of the bund stretch (300m) is not as bad as the critical section already treated, material requirement for 100m can be assumed to be half that for the critical section:

Clay - 6 cubes
Cement required for the clay: cement grouting - 8 bags
Cement required for the cement grouting - 20 bags

4.2.3 Evaluation of cost of grouting treatment

Using the basic rates, specific rates were prepared for the main cost elements of the grouting treatment, which are displayed in Table9. The total project cost of grouting 400 m is SLR 11,262,195; approximately SLR 11.3 million

Table 8-Estimate of construction cost of upstream cutoff

Item	Description	QTY	Unit	Rate (SLR)	Total (SLR)
1	Mobilization	3	Stage	250,000	750,000
2	Removal of existing riprap	5,197	m^3	124	644,428
3	Removal of bedding (gravel) layer	3,466	Nos	143	495,638
4	Removal of loose materials in upstream	11,550	Nos	123	1,420,650
5	Construction of coffer dam	Lump sum			360,000
6	Dewatering using two pumps	Lump sum			170,000
7	Excavation of cutoff trench	24,086	m^3	217	5,226,662
8	Transportation, Placement and compaction of CL in cutoff trench	24,086	m^3	82	1,975,052
9	Transportation, Placement and compaction of SC Layer	82,691	m^3	111	9,178,701
10	Placement of bedding layer	2,000	m^3	245	490,200
11	Placement of riprap layer	5,000	m^3	1,129	5,644,500
12	Total cost of civil works				26,355,831
13	Contingencies (10% of civil works, critical section)				2,635,583
13	Engineering and administration (5% of total cost of civil works)				1,317,912
Total project cost for the dam length of 700 m					30,309,326

Table 9-Estimate of grouting treatment

Item	Description Mobilization	QTY	Unit	Rate	Total
1	Mobilization	4	Stage	250,000	1,000,000
2	Installation of water pumps (all inclusive)	2	Nos	4,100	8,200
3	Preparation of ordinary platform for drilling over DH locations	26	Nos	3,064	79,664
4	Rotary drilling through overburden	1000	m	1,045	1,045,000
5	Rotary drilling through rock	180	m	1,306	235,080
6	Mud grouting including cost of labour and material (critical section) Mud grouting including cost of labour and material (non critical section)	17 17	m^3 m^3	3,957 3,604	67,269 61,268
7	Cement grouting in borehole (critical section) Cement grouting in borehole (non critical section)	30 30	m m	5,139 3,264	154,170 97,920
8	Core boxes	10	Nos	2,500	25,000
9	Total cost of civil works (critical section) Total cost of civil works(non critical section)				2,614,383 2,392,944
10	Contingencies (10% of civil works, critical section) Contingencies (10% of civil works, (non critical section)				261,438 239,294
11	Engineering and administration (5% of total cost of civil works) (critical section) Engineering and administration (5% of total cost of civil works) (non critical section)				130,719 119,647
Project cost for a dam length of 100m -Critical section					3,006,540
Project cost for a dam length of 100m –Non critical section					2,751,885
Total project cost for the dam length of 400 m (100m of critical section + 300m of non critical section)					11,262,195

5. Conclusion

The upstream cutoff is in two different cross sections due to the varying dam height along the dam axis. It has a SC layer laid over the upstream slope and a clay filled core trench, and a dam protection layer. Both layers, having a trapezoidal section, will be laid over the entire dam length of 700 m. The SC layer (at the cross section on 490 m) had on the U/S side slope, 3 m and 12 m widths at bund top and bed respectively. The clay layer in the core had 12m and 3 m widths at the bed level and the trench bottom respectively. While the effective fetch is 1.069 miles, the maximum wave height is 0.84 m. The riprap layer has a thickness of 0.45 m, and that of the bedding layer is 0.30m. Grouting treatment involves clay-cement grouting in the overburden and 6m deep cement grouting in the rock; requiring a total of 296 (4x74) grout holes.

The estimated cost of construction of the upstream cutoff is SLR 30.3 million. The estimated cost of grouting treatment, assuming it is done only on a 400m stretch of dam length, would be SLR 11.3 million, indicating the former would cost about 2.7 times the latter. Therefore, grouting treatment is a more economical option although upstream cutoff offers better seepage control. Further, in order to make way for the construction of upstream cutoff, the reservoir has to be emptied, which action will deprive farmers of having to use the reservoir for cultivation. The resulting economic and social cost is not considered when evaluating the cost of upstream cutoff.

The popularity of earth dams, compared to concrete dams, is increasing steadily. As indicated by Lambe and Whitman [9], the cost of earth construction per unit volume has remained approximately constant for last 50 years (the increased cost of labour has been offset by the improvements in earth handling equipment), whereas the cost of concrete per unit volume has steadily increased. Seepage being a major problem plaguing the use of earth dams, would call for more research to improve the grouting treatment to control seepage. Hence, further research is required to evaluate the use of different clay material (including bentonite) and finer cement to increase the effectiveness of grouting treatment.

References

1. Malkawi A.I.H., Al-Sheriadeh M., Evaluation and Rehabilitation of Dam Seepage Problems. A Case Study: Kafrein Dam, Engineering Geology, 56(2000), pp 335-345.
2. Uromeihy A., Barzegari G., Evaluation and Rehabilitation of Dam Seepage Problems at Chapar-Abad Dam, Iran, Engineering Geology, 91(2007), pp 219-228.
3. Turkmen S., Treatment of the Seepage Problems at the Kalecik Dam (Turkey), Engineering Geology, 68(2003), pp 159-169.
4. Ahmad S., Asalm M., Shafiq M., Reducing Water Seepage from Earthen Ponds, Agricultural Water Management 30(1996), pp 69-76.
5. Basak N.N., Irrigation Engineering, Tata McGraw-Hill Publishing Company Limited, 1999.
6. Mohan Das M., Saikia M., Irrigation and Water Power Engineering, PHI Learning Private Limited, 2011.
7. Ponrajah A.J.P., Technical Guidelines for Irrigation works, 1984.
8. Ponrajah A.J.P., Design of Irrigation Headworks, 2nd Edition, Irrigation Department of Sri Lanka, 1984.
9. Lambe T. W., and Whitman R.V., Soil Mechanics, John Wiley India, 2008.