Sri Lanka National Committee on Large Dams of International Commission on Large Dams

SLNCOLD

Technical Conference & Annual General Meeting

Organized by

Young Engineers' Forum – SLNCOLD

24th January 2024

Technical Conference &
Annual General Meeting - 2024

Organized by
Young Engineers' Forum –
Sri Lanka National Committee on Large Dams

Date	: 24th January 2024
Time	: 8:45 a.m. onwards
Venue	: Main Auditorium, Irrigation Department
Keynote Speaker	: Dr. Panduka Neluwala Department of Civil Engineering, Faculty of Engineering, University of Peradeniya

Sri Lanka National Committee on Large Dams (SLNCOLD)

(C/O Irrigation Department)

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- Mahaweli Authority of Sri Lanka (MASL)
- Ceylon Electricity Board (CEB)
- Central Engineering Consultancy Bureau (CECB)
- Northern Provincial Irrigation Department (NPID)
- National Water Supply and Drainage Board (NWS&DB)

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Message from the Director General of Irrigation President (SLNCOLD)

Updating technical skills and building knowledge base is more important to career growth of an engineer. Water resources engineering is one most developing and expanding field of Engineering in the world. Researching new knowledge and technologies, sharing experience among the engineering community are way forward to the development.

Sri Lanka National Committee on Large Dams (SLNCOLD) always willing to facilitate its members to share their research findings, ideas, views experiences and knowledge in water resources management sector. Organizing of this technical conference under the theme of "Dam Design & Monitoring" is one successful milestone of the annual programme of the SLNCOLD in 2024.

There are several papers submitted by the engineers from different organizations who are currently engage with designs and dam management. Sharing of knowledge and experience is the main purpose of the symposium. 1 invite all the participants to get the maximum use of this fruitful event, by accruing the valuable knowledge from the engineers who are present today.

I would like to convey my sincere gratitude for the committee members of the SLNCOLD for their dedication in organizing this event and specially thanking to the the Young Engineers forum (YEF) – SLNCOLD who is the energy behind the success of this marvelous job. Also, I wish to express my gratitude for all the member organizations and universities for being a part of this great event by participating and supporting. I thank you and appreciate the support of all sponsors who strengthen our hands to make this event success.

Eng. A. Gunasekara,
Director General of Irrigation,
President,
SLNCOLD.

Message from the Secretary (SLNCOLD)

I have great pleasure in writing this massage as the Secretary/ Treasurer of Sri Lanka National committee on Large Dams (SLNCOLD) of International Commission on Large Dams to the bulletin issued conjunction with technical conference on "Dam Design & Monitoring".

Sri Lanka is one of the world's oldest civilizations that developed water resources. There are many large dams still functioning well with minimum renovations and modifications which were built in Kings era. Designing and building of a dam is not an easy task. It involves a lot of calculations and assessment of risk associated with huge energy planning to store behind the dam. Engineers' duties will not be completed by building a dam but remains for the entire lifetime of the dam.

Updating knowledge will lead engineers to act smart and face the challenges in their career. SLNCOLD always has been a platform to share and enhance the knowledge and this technical conference is one of the successful events of the SLNCOLD's calendar for the year 2024.

I sincerely hope this conference would be useful to all dam professionals in the water resources development sector and boost the young professionals to develop their interest on new areas of knowledge.

I take this opportunity to thank all the members from the collaborating organizations for their unwavering support in organizing and participating this conference. Further, I extend my gratitude towards the keynote speaker and all the presenters in this conference for sharing their knowledge and experience. I would like to extend my sincere thanks to all the sponsors for their financial support in making this event a great success. 1 congratulate all the committee members, YEF Committee Members, participants and who were directly or indirectly involved in this event for making it a grand success.

Eng. Y.A.C.R. Kumara,
Secretary,
SLNCOLD.

Message from the President (YEF-SLNCOLD)

It is a great pleasure as the President of the Young Engineers' Forum (YEF) of Sri Lanka National Committee of Large Dams (SLNCOLD) of International Committee of Large Dams (ICOLD) to send this message for the e-bulletin issued for the first technical conference organized by YEF-SLNCOLD.

Over the history of human civilizations, water has been one of the key factors in day-to-day lives of every living being. Dams built in either concrete, earth or any construction material are the barriers for water flow and temporary reserves water to transfer during a dry period or for a dry zone. Hence, Dams can be considered as an important feature in water related lifestyle of mankind. The topic "Dam design & monitoring" has been a timely related topic for today's world of dams, because the history of the Sri Lankan civilization extends back to the years in BCs spanning over 2500 years in the history. It is the time to monitor our ancient dams and discuss the ways and means on how we can protect them for our next generations.

SLNCOLD has always been a platform to share the various views, experiences, and ideas of the dam engineers through their working environment with the other engineers who are working in the water sector. I am very much grateful for the establishment of YEF-SLNCOLD under SLNCOLD for sharing the dam related knowledge.

I would like to extend my best wishes to all the young Engineers who submitted the abstracts and hope that the proceedings of technical session will enhance the awareness of "Dam Design and Monitoring" among all the engineers in the country. Further, I take this opportunity to forward my immense thanks to my colleagues who supported to in making this event a success.

Eng. M. D. Janaka Priyantha Wickramasooriya,
President,
YEF-SLNCOLD.

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The International Commission on Large Dams (ICOLD)

The International Commission on Large Dams (ICOLD) was founded in 1928 as a non-governmental International Organization which provides a forum for the exchange of knowledge and experience in dam engineering. The Organization leads the profession in ensuring that dams are built safely, efficiently, economically, and without detrimental effects on the environment. Its original aim was to encourage advances in the planning, design, construction, operation, and maintenance of large dams and their associated civil works, by collecting and disseminating relevant information and by studying related technical questions. ICOLD members are essentially practicing engineers, geologists and scientists form governmental or private organizations, consulting firms, universities, laboratories and construction companies.

Since the late sixties, focus was put on subjects of current concern such as dam safety, monitoring of performance, reanalysis of older dams and spillways, effects of ageing and environmental impact. More recently, new subjects include cost studies at the planning and construction stages, harnessing international rivers, information for the public at large, and financing.

Mission of ICOLD

ICOLD leads the profession in setting standards and guidelines to ensure that dams are built and operated safely, efficiently, economically, and are environmentally sustainable and socially equitable.

ICOLD wishes to be the world's leading professional organization, dedicated to advancing the art and science of dam engineering and promoting the wise and sustainable development and management of world's water and hydropower resources.

ICOLD is assisting nations to prepare to meet the challenges of the 21st century in the development and management of the world's water and hydropower resources.

ICOLD Technical Committees

Presently, ICOLD has 29 Technical Committees that address current technical issues related to the development and management of water resources. Each Technical Committee is given a mandate by the General Assembly and works for 3 or 4 years and its works arc published as a "Technical Bulletin"

These publications can be purchased directly on the web site. But one copy is available at Irrigation Department Library which were received to the National Committee of Sri Lanka. Several Sri Lankan Engineers are serving to these Technical Committees by giving their inputs.

Sri Lanka National Committee on Large Dams of ICOLD (SLNCOLD)

Sri Lanka National Committee on Large Dams (SLNCOLD) was founded in 1953 to contribute to national development by the advancement of knowledge in and encouragement for improvements in the investigations, design, construction, maintenance, operation and management of large dams and their reservoir storages and related facilities.

SLNCOLD coordinates with the International Commission on Large Dams (ICOLD) and other member countries (presently there are 83 member countries that have been enrolled as member of ICOLD) to enhance the capability of SLNCOLD members.

In addition, following key functions are carried out by SLNCOLD;

- Organize seminars, symposiums and field visits for the members to share the knowledge and experience of dam engineers engage in planning, designs, construction and management of dam projects.
- Coordination with the ICOLD Central Office and other member Countries of ICOLD (presently there are of 106 member countries that have been enrolled as members of ICOLD). Disseminate the news and publications of ICOLD to the members.
- Enhance the capability of SLNCOLD members by coordination with the overseas experts who have expertise and experience on dam constructions.
- Organize presentations and discussions on social, environmental and economic aspects of water resources management by relevant experts.
- Represent Sri Lanka at ICOLD Executive annual Meetings Submission of papers, articles and communications received by members for the Congress (organized at every 3 years).
- Disseminate the information and knowledge obtained from other Member Countries throughout the world to the Sri Lanka National Committee members and the corporate members to enhance their capability of application of state of art technologies.
- Submission of information related to Sri Lanka for the questionnaires on technical surveys carry out worldwide.
- Publishing of newsletters and bulletins to disseminate recent developments, research information and current news on dam sector.

Young Engineers' Forum - SLNCOLD

Functions of YEF-SLNCOLD

YEF-SLNCOLD inspires young engineers to become active participants of the work carried out by SLNCOLD. Key functions of YEF-SLNCOLD are:

- Creating a network to encourage the attendance and involvement of younger engineers at the SLNCOLD meetings.
- Providing an opportunity for knowledge transfer to the next generation and ensuring the long-term functioning of SLNCOLD.
- Providing an opportunity for the young engineers to connect with each other to enable sharing of experiences.
- Organizing capacity development and knowledge sharing programmes.

YEF-SLNCOLD Council 2023/2024

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Eng. S. Gowrithasan (Provincial Irrigation Department, Eastern Province)

Synopsis of the Keynote Speech

Harnessing Technology for Dam Safety: A New Era of Preparedness and Efficiency

Dr. Panduka Neluwala
Department of Civil Engineering, Faculty of Engineering, University of Peradeniya.

The world is home to an estimated 62,000 dams according to the ICOLD database, a testament to the significant role these structures play in our societies. These dams, scattered across the globe, serve multiple purposes that are crucial for human development and environmental sustainability. Dams are integral for flood control and mitigation, providing a buffer against the destructive power of water. They also offer recreational and environmental benefits, creating reservoirs and lakes that become habitats for diverse flora and fauna. Moreover, dams are a key component in hydropower generation, contributing to the global renewable energy mix. They also play a pivotal role in water storage and supply management, ensuring a steady supply of water for domestic, agricultural, and industrial use.

Sri Lanka is home to nearly 100 large dams and 300 medium-sized dams. The major hydropower stations in the country contribute to a significant 34% of the country's power generation capacity, producing 5,364 GWh in 2022. This underscores the importance of these structures in the country's energy infrastructure. However, the benefits of dams come with inherent risks. The catastrophic failure of two dams in Derna, Libya, which resulted in more than 10,000 fatalities and displaced over 400,000 individuals, serves as a stark reminder of the potential hazards associated with dam infrastructure.

As the baton of responsibility for maintaining and improving these critical infrastructures is passed from Generation X to Generation Y, a unique opportunity arises. Generation Y engineers, who are on the cusp of stepping into leadership roles, bring with them a deep understanding and proficiency in digital technology. This generation, having grown up in the digital age, is well-equipped to leverage tools like satellite data for remote sensing, real-time monitoring systems for instant feedback, numerical simulations for predictive modeling, and machine learning for pattern recognition and prediction. These tools, combined with their innovative thinking, position them to usher in a new era of dam safety and efficiency. As they navigate the challenges and uncertainties presented by climate change, their role in water resource management becomes even more crucial. Their task is not just to sustain but to enhance the legacy left by Generation X, harnessing the power of technology to ensure a safer and more sustainable future.

Our team has been successful in measuring reservoir water level elevations using ICESat-2 data. The results have been highly accurate when compared to observations. This research can be further expanded by incorporating data from other satellites such as Sentinel-3, providing a valuable tool for monitoring reservoirs without gauges. We have developed weather forecasting models for multiple basins and created hydrological models to calculate the inflow to the reservoir and simulate reservoir operation. The integration of these two models enables reservoir operation forecasting, which is instrumental in informing the public and facilitating decision-making processes. Furthermore, we can model the impacts of climate change to estimate future water availability and the frequency of extreme events.

Recent events have further underscored the need for comprehensive dam breach studies. For instance, an earthquake event in Turkey, measuring 7.5 on the Richter scale, occurred within a 115 km radius from the Ataturk Dam, the world's third-largest rockfill dam. Although the dam remained undamaged, the incident highlights the importance of considering seismic activity in dam safety assessments. These incidents emphasize the need for robust design, regular maintenance, and comprehensive safety assessments in dam management.

Recently, our group published a review paper titled "Review on Model Development Techniques for Dam Break Flood Wave Propagation" in the highly-ranked WIREs Water journal (Peramuna, et al., 2023). We have conducted extensive research to clarify the important aspects to consider in dam breach studies. Defining the breach size is a challenging aspect of dam breach configuration. The parameters that need to be defined include the Average Breach Width (Bave), Height of Breach (Hb), Formation Time (tf), and Slope of the Breach. There are four main methods to obtain the breach parameters and estimate the dam breach flow: comparative analysis, guidelines from different authorities, regression-based methods, and physically-based simulation models. However, the guidelines for Concrete Faced Rockfill Dams (CFRDs) indicate that estimations can be uncertain or have indefinite values within the ranges. Moreover, research has found that the acceleration response of CFRDs is dependent on dam geometry and the magnitude of ground motion in the case of an earthquake, factors not considered in the guidelines.

A recent publication by Uduweriya et al. (2020) provides key insights into seismic risks in Sri Lanka. Dams are typically designed for a 475-year return period Design Basin Earthquake (DBE), which is the reference design earthquake for the appurtenant structures in many countries. The Mahaweli river basin, located in the central highlands, is likely to experience a 0.1 g Peak Ground Acceleration (PGA) for a return period of 475-year earthquake, and a 0.3 g PGA for a return period of 2500 years.

Our team has conducted an in-depth study on potential dam breach scenarios for the cascade of dams in the Mahaweli River basin. We considered multiple scenarios, including earthquakes and extreme rainfall events. The dam breach parameters were defined carefully based on Finite Element Method (FEM) results and literature reports. This comprehensive study provides insightful results that contribute significantly to preparedness strategies.

In conclusion, while it is impossible to predict when disasters may occur, it is always better to stay prepared. This presentation underscores the importance of leveraging technology and rigorous scientific methods in enhancing dam safety and preparedness, highlighting the crucial role of the new generation of engineers in navigating these challenges and ensuring a safer and more sustainable future.

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Application of the Revised Universal Soil Loss Equation (RUSLE) for the proposed Thalpitigala Reservoir Catchment

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Abstract

Uma Oya catchment is a sub basin of Mahaweli river basin. The proposed Thalpitigala Reservoir is located upstream of Bathmedilla Anicut which receives a considerably higher rainfall intensity. Located in the southern part of the central hills, this proposed reservoir is expected to receive a higher sedimentation throughout the year due to high soil erosion in the catchment. With a proposed 46 m high dam, the sediment level of the reservoir is very critical on the designs, and therefore a sedimentation study was carried out. First, the Revised Universal Soil Loss Equation (RUSLE) was applied for the proposed Thalpitigala Reservoir catchment and the soil erosion rate calculated from that was used to find the sediment yield of the Thalpitigala Reservoir. The study concluded that the average soil erosion rate of the catchment is 7.77 t/ha/year and sediment yield at the reservoir is 1.8 t/ha/year.

Keywords: Soil Erosion, Sedimentation, Sediment yield, Reservoir Sediment Calculation

1. Introduction

1.1. Requirement of Sediment study

All reservoirs formed by dams on natural water courses are subject to some degree of sediment inflow and deposition. The problem confronting the project planner is to estimate the rate of deposition and the period before the sediment will interfere with the useful function of the reservoir. Provisions should be made for sufficient sediment storage in the reservoir at the time of design so as not to impair the reservoir functions during the useful life of the project or during the period of economic analysis.

The continued deposition develops distribution patterns within the reservoir which are greatly influenced by both operations of the reservoir and timing of large flood inflows. Deposition of the coarser sediments occurs in the upper or delta reaches while finer sediments may reach the dam and influence the design of the outlet works. A major secondary effect is the downstream degradation of the river channel caused by the releases of clearer water.

Among the catchments in Sri Lanka, the highest sediment yield has been reported from the Upper Uma Oya catchment. According to a study done in 2019 [1], it has been found that that the soil erosion rates of Uma Oya catchment are 3 to 130 times faster than the soil loss tolerance in Sri Lanka.

Therefore, a higher sedimentation is expected in the proposed Thalpitiagala Reservoir which is in Uma Oya catchment. By this study, the sediment yield expected in the reservoir was estimated after finding the soil erosion in the reservoir catchment.

Considering the upstream reservoirs of the catchment, Puhulpola reservoir

(180km^2) and Dyraaba reservoir (158km^2), the suspended solids passing to the Thalpitigala reservoir, may assumed to be reduced. However, since those reservoirs have bottom outlets to flush the sediments, it is a worthy assumption that the whole catchment sediments are to be collected in the proposed reservoir.

1.2. Soil Erosion

Soil erosion is among the greatest harmful worldwide environmental concerns since it causes detrimental effects on both on-site and off-site. Soil erosion is a process of soil loosening, transport, and deposition. [1]. Runoff and splash are the vehicles by which soil erosion is intensified during rainfall events. The impact of raindrops on the bare soil surface disrupts soil surface structure and generates seal formation. [6].

Other than those natural causes, overgrazing, deforestation, farming practices and construction activities have intensified the soil erosion. Therefore, assessing the extent and seriousness of erosion remains a difficult task since soil erosion has increased up to unacceptable levels in recent years. [4] However, during the last few decades, various empirical, physically based, and conceptual computer models, such as the European Soil Erosion Model, the Universal Soil Loss Equation (USLE), and Machine Learning, Deep Learning have been developed to assess soil erosion. Revised Universal Soil Loss Equation (RUSLE) model is also a widely used method to find the soil erosion of an area.

Main factors affecting soil erosion are the slope, land cover, climate i.e. the rainfall and soil parameters. In RUSLE model, all these factors are assessed and it was used to assess the soil erosion of the Thalpitigala reservoir catchment.

1.3. Details and layout of the proposed Thalpitigala Reservoir project

Uma Oya catchment, a sub basin of Mahaweli river basin, is in the southern part of the central hills in Sri Lanka, covering an area of 720 km^2 in the Nuwara Eliya & Badulla districts. Although the water in Uma Oya is diverted to Kirindi Oya through two reservoirs constructed across Dulgolla Oya (Puhulpola) and Mahathotilla Oya (Dyraaba), there is a considerable catchment in between these two reservoirs and Bathmedilla Anicut. Yield from that catchment is proposed to be used for the Thalpitigala reservoir. The location and the catchment of the project is shown in Fig. 1.

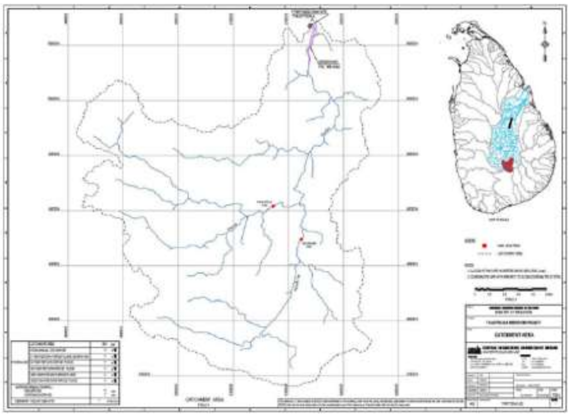

Figure 1: Catchment area and the location of the proposed reservoir

A 46 m high dam is to be built through the Uma Oya near Wasanagama and the irrigation release is sent through a tunnel to the powerhouse located downstream. The proposed gravity dam consists of 5 nos of radial gated spillway, and a stilling basin which will dissipate the energy after the spillway. There are two bottom outlets designed to fulfil the requirements including flushing sediments, emptying the reservoir and cater sudden downstream release.

The intake structure near the dam, underground tunnel, surge tank and the penstock altogether create the 1450 m long waterway. The tunnel will augment the Bathmedilla scheme, and the Powerhouse will generate power thorough 2 numbers of 7.5 MW turbines. See Fig.2.
The amount of sediment in the water can be clearly seen in the rainy seasons in the Figure 3. Therefore, it is evident that the sediment management in the proposed reservoir is a key design factor to maintain the safety and the efficiency of the reservoir in the future.

Figure 2: Layout of the proposed project

Figure 3: Proposed dam location in rainy season

2. Methodology

First the Revised Universal Soil Loss Equation (RUSLE) was used to find the soil erosion of the catchment.
To cover the spatial extent of the catchment, relevant grids of 30mx30m resolution Digital Elevation Model (DEM) was used. In combination with an ArcMap interface to develop soil erosion, each parameter was estimated.
Then the sediment delivery ratio was estimated for the proposed reservoir, from which it was found how much sediments will pass to the reservoir, from the eroded amount of the catchment. Then the sediment yield was estimated for the proposed reservoir.

2.1. Revised Universal Soil Loss Equation (RUSLE)

$$A = R \times K \times LS \times C \times P$$

A= Average annual soil loss per unit area
R= Rainfall-runoff erosivity factor
K= Soil erodibility factor
LS= Slope length and steepness factor
C= Cover and management factor
P= Support and conservation practices factor

Above mentioned is the Revised Universal Soil Loss Equation (RUSLE). The relevant parameters are briefly described following.

Rainfall-runoff erosivity factor

The erosive power of rainfall can be estimated by calculating the erosivity factor for a particular location. It depends on the amount and the intensity of rainfall. Daily rainfall data of 14 rainfall gauging stations for 20 years (1997–2017) duration were used in calculating the R factor along with the rainfall erosivity factor equation developed for Sri Lanka conditions. [5]

$$R = \frac{(972.75 + 9.95F)}{100}$$

R=rainfall-runoff erosivity factor (MJ mm $ha^{-1}h^{-1}yr^{-1}$ and F= Average annual rainfall (mm).

Soil erodibility factor

Soil erodibility factor (K) is one of the main factors governing soil erosion. It expresses the susceptibility of soil towards erosion and measures the contribution of soil types. K factor is defined as the rate of soil loss per unit of erosive energy created by the rainfall calculated under a standard condition by a plot of land consisting of clean bare soil with slope of 9% and 22.6m long.

Slope length and steepness factor

Slope length and steepness has the greatest influence on soil loss and describes the effect of topography on soil erosion. Factor measures the effect of slope steepness and was calculated by following equations, using raster calculator based on the relationship given by,

$$S = 10.8 \sin\theta + 0.03\ ; for\ slope < 9\%$$
$$S = 16.8 \sin\theta - 0.5\ ; for\ slope \geq 9\%$$

Where S = slope steepness factor and θ=Slope angle in degree. L factor was calculated by following equation using raster calculator on ArcMap, based on equation given by;

$$L = \left(\frac{\lambda}{22.1}\right)^{m}$$

where L = slope length factor, $\lambda =$ horizontal projected slope length, (m) ($\lambda =$ flow accumulation x cell size), and m = slope length exponent. In this equation, "m" is the slope length exponent that varies based on slope steepness. Slope length exponent equals 0.5 if the slope is 4.5% or more, 0.4 on slopes between 3% and 4.5%, 0.3 on slopes between 1% and 3%, and 0.2 for flatter terrains with gradient less than 1%.

Cover and management factor and Support and conservation practices factor

The effect of the cropping system and crop management practices on the rate of erosion is expressed using the C factor. It also represents the contribution of soil disturbing activities, plant sequence and productivity level, soil cover and sub surface bio-mass on soil erosion. Cover and management factors are important in developing conservation plans.[1]

2.2. Sediment Delivery Ratio (SDR)

Sediment delivery ratio (SDR) is the fraction of eroded sediment that is transported from a watershed to a specific point of interest, such as a reservoir, river mouth, or other water body. It is expressed as a percentage and is calculated as the ratio of sediment yield at the point of interest to gross erosion within the watershed. The average slope of the stream channel is more significant than other parameters in estimating sediment delivery ratio, and it is a function of slope of mainstream channel. [3]

$$SDR = 0.627 \times (SCS)^{0.403}$$

This equation is used to estimate the sediment delivery ratios in different river basins where SCS= main stream channel slope measured in percent unit. This value was calculated from google earth as 8.4%.

From that the sediment yield can be calculated since

$$SDR = \frac{Sediment\ yield}{Gross\ erosion}$$

3. Results

Soil Erosion

Rainfall-runoff erosivity factor, R value was calculated as 228.6, Since the average annual rainfall of the catchments given as 2200mm.

Erodibility values used for the Uma Oya watershed are based on two major soil types. [1]

Table 1: Soil erodibility values (K factor)

Soil Type	K
Reddish brown earth	0.27
Red-yellow podzolic soils	0.22

0.22 was used in the study as soil erodibility factor from the given table.

ArcMap was used to find the LS factor and, Since the Slope map shows an average of 13 (Fig.4), m was taken as 0.5. L and S factors were calculated by using flow accumulation and slope in degree as inputs and finally LS factor map was generated by the multiplication of both L and S factors in raster calculator in ArcMap. The mean LS value was read as 2.5. (Figure 5).
The following factors were taken for the calculation of factors c and p based on the land use of the catchment.

Table 2: Factors C, P

LULC Description	C factor	P factor	Sediment retention
Other cultivation	**0.1**	**0.25**	**0.25**

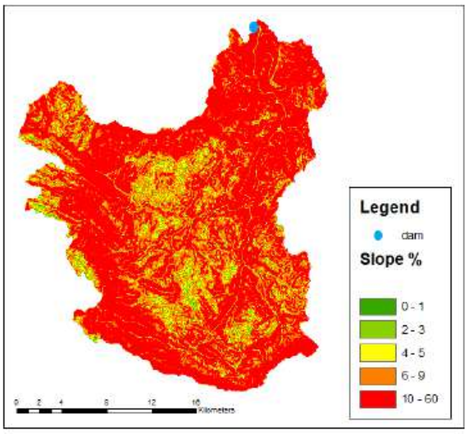

Figure 4: Slope of the catchment

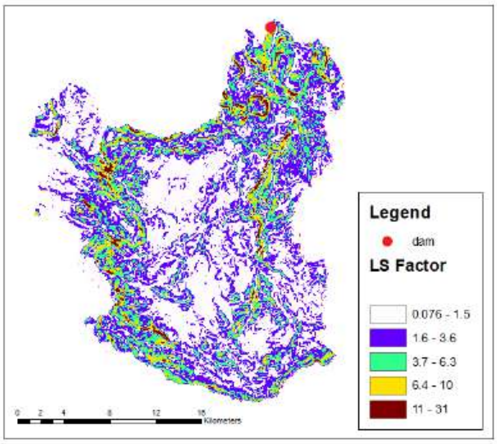

Figure 5: LS Factor

Table 3: Summary of RUSLE inputs

Parameter of RUSLE	**Value**
Rainfall-runoff erosivity factor, R	**228.6**
Soil erodibility factor, K	**0.22**
Slope length and steepness factor, LS	**2.5**
Cover and management factor, C	**0.1**
Support and conservation practices factor, P	**0.25**

Therefore,
A = 3.14 t/a/year = **7.77 t/ha/year**

Sediment Yield

SCS was calculated from google earth as 8.4%. Therefore;

Sediment Delivery Ratio; SDR=0.627 x $(0.084)^{0.403}$=**0.23**
Then, Sediment yield = 7.77 x 0.23 =**1.8t/ha/year**

4. Conclusion

From this study, the sediment yield for the proposed Thalpitigala Reservoir was estimated as 1.8 t/ha/year. For that, annual soil erosion rate was used as 7.77 t/ha/year, which was obtained from the RUSLE. The following table shows another four annual soil erosion rates in Sri Lanka, which were qouted from other studies for comparison.

Table 4: Mean annual soil erosion rates

Study	Mean annual Soil erosion rate (t/ha/year)
Feasibility Study for Thalpitigala Reservoir Project-by Irrigation Department	**7.2**
Assessment of Soil Erosion in Uma Oya Catchment (B.A.R.H. Dias, 2019)	**14.3**
Assess Soil Erosion in "Kalu Ganga" River Basin (D. L. D. Panditharathne, 2019)	**0.63**
Soil loss estimation in Kelani river basin (C.M. Fayaz, 2019)[2]	**10.88**

The soil erosion rate concluded by this study is almost same as the value given in the Feasibility Study for Thalpitigala Reservoir Project. Also, the Mean annual Soil erosion rate for the total catchment, is 14.3 t/ha/year, and therefore this result from the study, 7.77 t/ha/year can be concluded as acceptable.

5. Significance

This study was carried out as a part of the project analysis of the proposed Thalpitigala Reservoir Design. Therefore, some of the parameters were not analyzed thoroughly, with the time frame and data availability. However, with more accurate data such as the land usage and soil types in the catchment, and the Arc Map model of those details, the result can be more validated as a future study. Also, with the help of the studies on the sediment characteristics, the sediment level of the proposed reservoir could be obtained, which will be a very valuable and important parameter on designing the dam and appurtenant structures.

Since the soil loss tolerance in Sri Lanka is 5 t/ha/year [1], the value is higher than the tolerance. Therefore, soil erosion management techniques and sedimentation management techniques should be applied to the catchment and reservoir.

Some of the proposed techniques are as following.

- Reduce sediment yield from upstream – Introduce check dams to capture sediments.
- Route Sediments/ Remove sediments – Introduce two bottom outlets to the dam.

The most feasibility option could be identified from a future study.

Acknowledgements

The author is thankful to the General Manager of CECB and staff members of WRP section for helping with the data and study.

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Potential of Enhancing the Capacity of Wemedilla Reservoir, adopting a Piano-Key Weir Type Spillway

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Abstract

The study showcases how incorporating a Piano Key Weir (PKW) spillway can enhance Wemedilla Reservoir's capacity without altering bund levels or acquiring upstream lands. Wemedilla Reservoir has been identified as a key location in diverting Mahaweli water to North Western Province by the Ministry of Irrigation, Sri Lanka. Increasing the reservoir's storage will benefit many areas and Wemedilla Reservoir was therefore selected for the present case study. Full Supply Level (FSL) of Wemedilla Reservoir is 221.34 m above mean sea level (AMSL) and its High Flood Level (HFL) has been taken as 222.56m AMSL, by the Irrigation Department of Sri Lanka. The difference between HFL and FSL is, therefore, 1.22m. The elevation of the proposed PKW crest was taken as 222.2m AMSL and hence the reservoir's capacity can potentially be increased by 16%, with the selected PKW configuration. 72% reduction of afflux, in comparison with the Existing Spillway, was observed with PKW. However, before implementing this proposal, further steps, including physical model testing, cost-benefit analysis, and a comprehensive study of environmental and social impacts are essential.

Keywords: Climate Change, PKW Type A, PKW Type B, Spillway Discharge, Water Security

1. Introduction

It is a widely known fact that Sri Lanka has been listed among the countries most affected by extreme weather events. Due to climate change impacts, high-intensity rainfalls, floods, and droughts are already more often and severe in many places of Sri Lanka. One of the adaptation options identified in the National Adaptation Plan for Climate Change Impacts in Sri Lanka: 2016 – 2025 [1], is the development of storage facilities, over the island. The incorporation of Piano Key Weir (PKW) spillway at Giritale Wewa (first of its kind in Sri Lanka) has led to a capacity increase of the reservoir without increasing the dam height and marginal increase in High Flood Level (HFL) [2]. This intervention for Giritale Reservoir has demonstrated PKW's twin capability of increasing water storage, while minimising upstream inundation due to its increased low head flood discharge efficiency [3]. According to [4] increasing the storages of existing reservoirs by adopting PKW at Giritale is the best option for development of storage capacities in comparison with desilting and new reservoir construction.

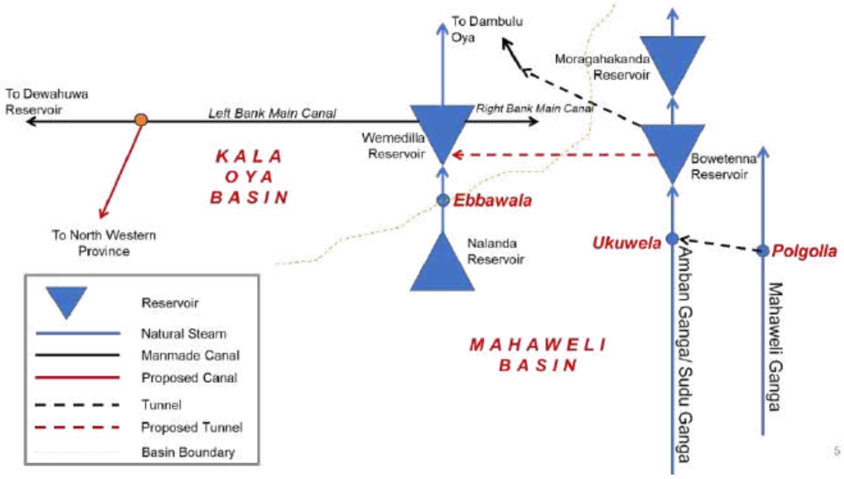


Figure 6: Wemedilla Reservoir, associated water conveyance system and their proposed linkage to the Mahaweli System

The objective of the present work was to carry out a case study for Wemedilla Reservoir for demonstrating the potential of adopting PKW type spillways to enhance the reservoir's capacity, while maintaining Bund Top Level (BTL) and High Flood Level (HFL) at their original levels.

2. Wemedilla Reservoir and the Existing Spillway

Wemedilla Reservoir has been constructed in 2008 and it is currently operated under Dambulla Division in Kandy Range of Irrigation Department (ID), Sri Lanka. Wemedilla Reservoir is augmented with the diversions from Nalanda Reservoir through Ebbawala Regulator. It has also been proposed to divert water from Bowatenna Reservoir through a tunnel to Wemedilla Reservoir in diverting Mahaweli water to North Western Province, under Mahaweli Water Security Investment Programme (MWSIP) of Ministry of Irrigation. Wemedilla Reservoir, associated water conveyance system and their proposed linkage to the Mahaweli System are shown in the schematic diagram given in ***Figure 6***. Wemedilla Reservoir therefore, in future, will act as an intermediate storage reservoir within the said system. Storage capacity increase in Wemedilla Reservoir hence increase the operational flexibility of the system and it will consequently add benefits to many areas. Wemedilla Reservoir was therefore selected for the present case study.

Figure 7: Existing Spillway of Wemedilla Reservoir

As per the Reservoir Data available with ID, BTL of the Wemedilla Reservoir is 223.93 m above mean sea level (AMSL). Full Supply Level (FSL) of the reservoir is 221.34 m AMSL and its HFL has been taken as 222.56m AMSL. The reservoir capacity at FSL is 4,594 acre feet (5.67 MCM), as per the records of ID.

The Existing spillway of Wemedilla Reservoir is an ungated spillway of 50.3m in length that consists of a 1.2m high ogee section followed by a chute. Photographs of the Existing Spillway are shown in *Figure 7*.

In the present case study it is proposed to replace the existing ogee section of the spillway with a PKW to enhance its discharge capacity.

3. Selection of PKW Geometric Parameters

3.1 Nomenclature

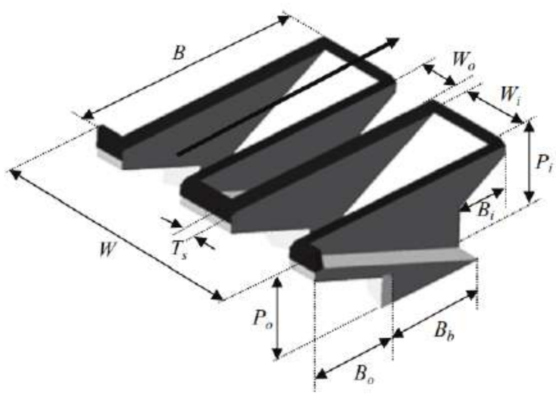


Figure 8: Fundamental PKW parameters (3D view)

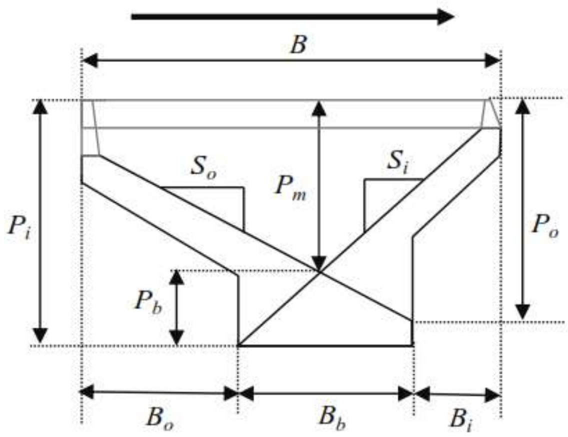


Figure 9: Fundamental PKW parameters (Cross-section)

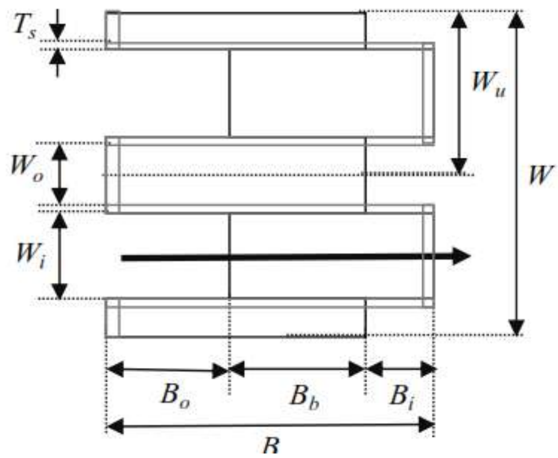


Figure 10: Fundamental PKW parameters (Plan view)

Table 5: Nomenclature for the fundamental PKW parameters [5]

Symbol	Definition
B_b	Base length
B_o	Upstream (outlet key) overhang length
B_i	Downstream (inlet key) overhang length
B	Upstream-downstream length of the PKW ($B = B_b = B_i + B_o$)
B_h	Sidewall overflowing crest length (between inlet and outlet key crest axes, length less than B)
P_i	Height of the inlet entrance measured from the PKW crest
P_o	Height of the outlet entrance measured from the PKW crest
P_b	Height of the apron level at inlet key and outlet key intersection
S_i	Slope of the inlet key apron (length over height)
S_o	Slope of the outlet key apron (length over height)
W_i	Inlet key width
W_o	Outlet key width
W	Total width of the PKW
W_u	Width of a PKW unit
T_s	Sidewall thickness
L_u	Developed crest length of the PKW unit along the overflowing crest axis ($L_u = W_i + W_o + 2B_h + 2T_s$)
L	Total developed length over the overflowing crest axis
N_u	Number of PKW units constituting the structure

In the present work, PKW naming convention proposed by [5] and the relevant symbols used therein were adopted. The key geometrical parameters of a PKW are defined in ***Error! Reference source not found.***, while the fundamental parameters are also depicted graphically in ***Figure 8***, ***Figure 9*** and ***Figure 10***.

3.2. Types of PKW

The following four types of PKW, have been identified by the researchers according to the shape of the weir.

- *Type A:* Having overhangs on both upstream and downstream
- *Type B:* Having overhangs only on upstream side
- *Type C:* Having overhangs only on downstream side
- *Type D:* No overhangs

Figure 11 provides illustrations of the said four PKW Types.

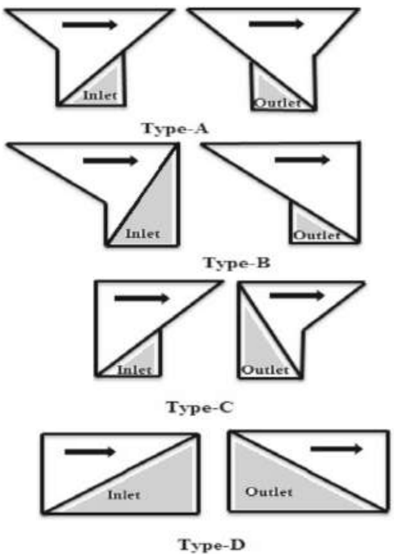


Figure 11: Illustrations of the four PKW Types

parameters on discharge capacity

[6] mentions that inlet and outlet overhangs enhance the discharge capacity of PKW, as they implicitly increase *L*, and PKW types with overhangs were therefore preferred for the present case study. On the other hand, PKW Type D with no overhangs would not be an economical option for relatively higher weir heights. [7] states that Type C is ineffective compared to Type A and Type B. As per [8], discharge capacity of Type B is higher at lower discharges. The larger overhangs on upstream of PKW Type B, increases its efficiency, as the inlet flow area is increased, while reducing the energy loss [9]. Due to the symmetrical shape, structural design of PKW Type A could be simpler, compared to Type B. Both Type A and Type B were therefore studied in the present work.

3.4 PKW configurations

The existing free board of the reservoir is 1.37m (the difference between BTL and HFL) and free board requirement of the reservoir was reassessed during the present study based on the guidelines given in [10], before moving into PKW studies. Required free board was hence found to be 4.138 feet (1.26m) and the existing free board is therefore adequate.

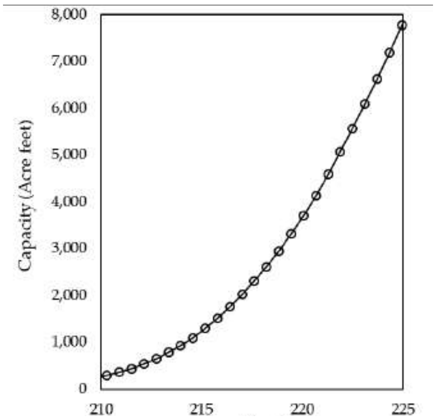


Figure 12: Elevation – Capacity relationship for Wemedilla Reservoir

Capacity Relationship of the reservoir (see ***Figure 12***), the estimated capacity at the proposed FSL is 6.56 MCM. Toe level of proposed PKW was taken same as that of the Existing Spillway. A parapet wall of 0.15m high, was incorporated, at the top of PKW for all the configurations considered in the present work.

Proposed general PKW configuration for Wemedilla Reservoir is hence shown in ***Figure 13***.

Table 6: PKW Type A configurations taken for the study

Configuration	A-4.2-1.25	A-4.2-1.33	A-4.2-1.5	A-4.8-1.25	A-4.8-1.33	A-4.8-1.5
B_b	4.2	4.2	4.2	4.8	4.8	4.8
B_o	2.1	2.1	2.1	2.4	2.4	2.4
B_i	2.1	2.1	2.1	2.4	2.4	2.4
B	8.4	8.4	8.4	9.6	9.6	9.6
P	1.5	1.5	1.5	1.5	1.5	1.5
W_i	2.25	2.40	2.70	2.25	2.40	2.70
W_o	1.8	1.8	1.8	1.8	1.8	1.8
W_i/W_o	1.25	1.33	1.50	1.25	1.33	1.50

All lengths are in metres

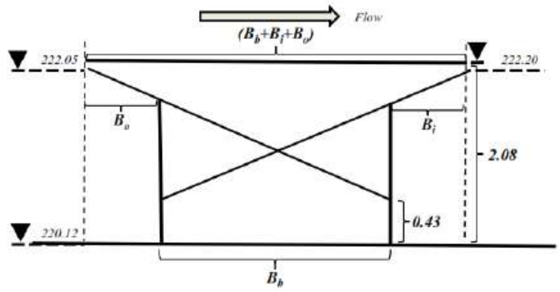
Table 7: PKW Type B configurations taken for the study

Configuration	B-4.2-1.25	B-4.2-1.33	B-4.2-1.5	B-4.8-1.25	B-4.8-1.33	B-4.8-1.5
B_b	4.2	4.2	4.2	4.8	4.8	4.8
B_o	4.2	4.2	4.2	4.8	4.8	4.8
B_i	0	0	0	0	0	0
B	8.4	8.4	8.4	9.6	9.6	9.6
W_i	2.25	2.40	2.70	2.25	2.40	2.70
W_o	1.8	1.8	1.8	1.8	1.8	1.8
W_i/W_o	1.25	1.33	1.50	1.25	1.33	1.50

All lengths are in metres

Two values for B_b were selected as 4.2m and 4.8m, since there is no limitation of space. B_i and B_o for Type A were taken as $B_b/2$, while B_o for Type B was taken to be equal to B_b.

Many past studies on inlet and outlet key widths agree that W_i/W_o >1 has higher discharge efficiency compared to W_i/W_o <1. [11], having carried out physical model test for W_i/W_o = 1, 1.25 and 1.5, has found that the optimal range of W_i/W_o is between 1.25 and 1.5. Three values of W_i/W_o (1.25, 1.33 and 1.5) were consequently selected for the present work.



All lengths are in metres

Figure 13: Proposed general configuration of PKW for Wemedilla Reservoir

T_s was chosen as 0.3m, considering the practicability of construction with reinforced concrete while maintaining sufficient cover. Type A and Type B PKW configurations considered for the present study are shown in ***Table 6*** and ***Table 7***, respectively.

3.5. PKW discharge relationships

Several empirical equations on PKW discharge relationships, based on the results of physical model tests, have been proposed by the researchers, but some of them are only be applicable to PKW Type A. [12] developed Equation (1) to estimate the discharge coefficient, C_d of a PKW, based on model results carried out for PKW Type A, B and C.

$$C_d = 0.212\left(\frac{H}{P}\right)^{-0.675}\left(\frac{L}{W}\right)^{0.377}\left(\frac{W_i}{W_o}\right)^{0.426}\left(\frac{B}{P}\right)^{0.306} \times \exp\left(1.504\frac{B_o}{B}+0.093\frac{B_i}{B}\right)+0.606 \qquad (1)$$

Equation (2) can subsequently be used to estimate the discharge, Q from the PKW in consideration.

$$Q=\frac{2}{3}C_d\sqrt{2g}WH^{3/2} \qquad (2)$$

C_d and Q were accordingly, estimated for each PKW configurations shown in ***Table 6*** and ***Table 7***.

4. Flood Routing

Behaviour of the Existing Spillway and the proposed PKW configurations were modelled for 1,000-year return period flood, by performing flood routing. Accordingly, adequacy of the discharge capacity of the proposed PKW configurations were checked. The same weirs were also simulated for 10,000-year return period flood, for making sure that the bund is not overtopped by flood water. Inflow flood hydrographs were determined using the Snyder's method, based on the methodology given in [10], while the design rainfall computations were based on [13].

5. Results and Discussion

5.1 PKW discharges

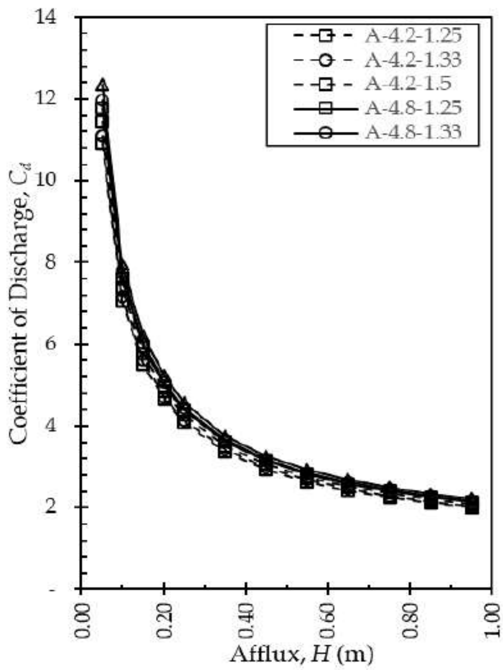


Figure 14: Variation of C_d values with H for PKW Type A configurations

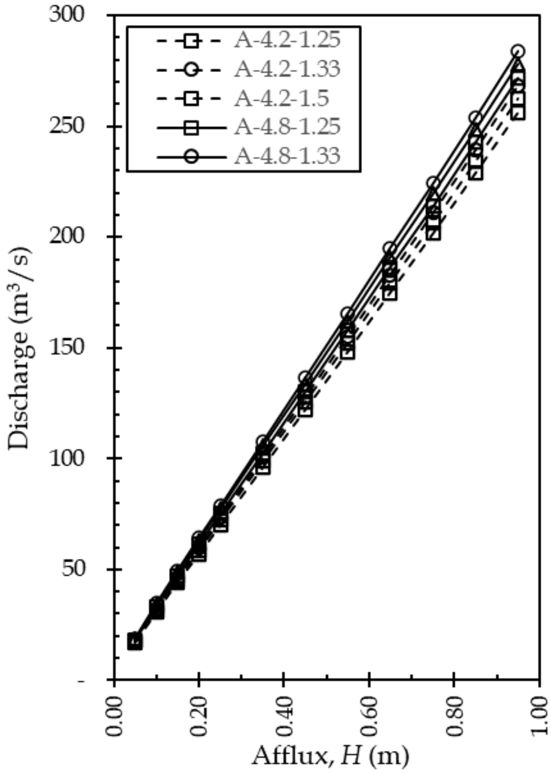


Figure 15: Variation of PKW discharge with H for Type A configurations

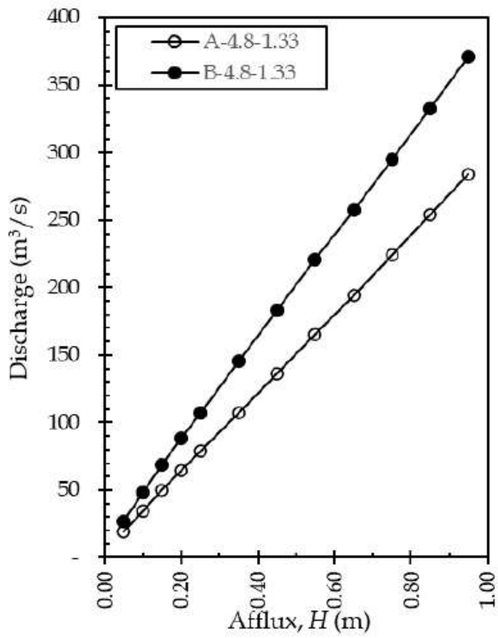


Figure 16: Variation of discharges of A-4.8-1.33 and B-4.8-1.33 with H

Variation of the C_d values with H is shown in ***Figure 14*** for PKW Type A configurations and the discharge values for different H values are consequently depicted in ***Figure 15***, for the same PKW configurations. Highest discharges were resulted for PKW configurations A-4.8-1.33 and B-4.8-1.33 and their discharge variations with H are shown in ***Figure 16***. Hence, it is also evident, from the results derived from Equation (1), that the highest discharge capacity is resulted for W_i/W_o = 1.33.

5.2 Flood routing

Table 8: Summary of results of flood routing

Return Period	Spillway	Max. Inflow (m^3/s)	Max. Outflow (m^3/s)	Max. Water Surface Elevation (m)	Max. Afflux (m)
1,000-year	**Existing**	183.5	115.9	222.56	1.22
	A-4.8-1.33	183.5	172.8	222.61	0.41
	B-4.8-1.33	183.5	176.6	222.54	0.34
10,000-year	**Existing**	222.7	145.8	222.75	1.41
	A-4.8-1.33	222.7	210.4	222.68	0.48
	B-4.8-1.33	222.7	214.8	222.59	0.39

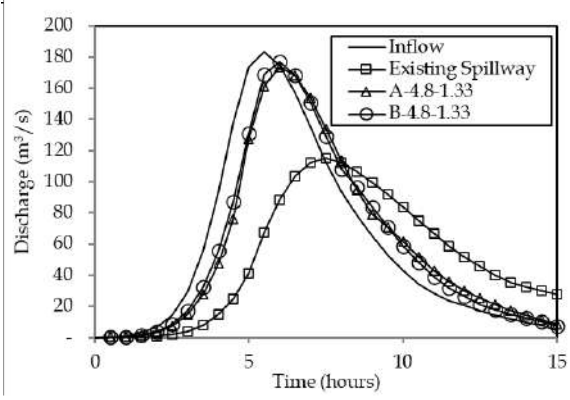


Figure 17: Response of A-4.8-1.33 and B-4.8-1.33 for 1,000-year return period flood, in comparison with the Existing Spillway

Flood routing, for 1,000-year return period was performed for the Existing Spillway and PKW configurations A-4.8-1.33 and B-4.8-1.33. They were further simulated for 10,000-year return period flood and the two selected PKW configurations were capable enough to withstand 10,000-year flood without overtopping the bund. During the flood routing the capability of the Existing Spillway for passing the 10,000-year return period flood was also confirmed. The results of flood routing are summarised in ***Table 8*** and inflow and outflow flood hydrographs for the said PKW configuration are depicted, in comparison with the same for the Existing Spillway, in ***Figure 17***. The lowest estimated afflux values were found to be for PKW configuration B-4.8-1.33, as depicted in ***Table 8***.

It is hence evident that FSL of Wemedilla Reservoir can successfully be raised up to 222.2m AMSL, keeping the BTL of the reservoir unchanged. According to the Elevation - Capacity Relationship of Wemedilla Reservoir, reservoir capacity can therefore be increased up to 6.56 MCM (5,321 acre feet), by 0.89 MCM (727 acre feet) which is a 16% increase compared to its original capacity. Afflux is reduced by 72% for both 1,000-year and 10,000-year floods, with the incorporation of B-4.8-1.33, when compared with the Existing Spillway. As the HFL for B-4.8-1.33 is below the present HFL of the reservoir, it is anticipated that no additional land acquisition is required for reservoir capacity enhancement, with the present proposal.
From the results of the computations done in the present case study, Type B configurations were shown to be better compared to Type A, which is also evident from literature.

5.3. Limitations of the present study and proposed future work

The present work is solely a hydraulic engineering study carried out using empirical equations, for demonstrating the possibility of enhancing Wemedilla Reservoir's capacity, with the incorporation of PKW spillway, which does not include the studies on environmental impacts, cost benefit analysis and any impact on the encroachers of reservoir bed (if any). Detailed studies on such impacts are essential before implementing this proposal.

[12] mentions the limitations of applying Equation (1) and due to such limitations and the approximations made in the same equation, it is proposed to verify the results through physical model tests. In addition, the performance of the selected PKW configurations for the Probable Maximum Flood must also be tested during such model studies. It is also essential to evaluate and analyse the selected geometries in detail, considering structural, geotechnical and construction engineering aspects.

6. Conclusions

Capacity enhancement potential of Wemedilla Reservoir was demonstrated with the incorporation of PKW, without raising of reservoir bund level or land acquisition in the upstream area. The PKW Type B configuration with a base length of 4.8m and the ratios between inlet and outlet key widths of 1.33, can be identified as the best out of the proposed configurations. The capacity of Wemedilla Reservoir can be increased by 16% with the selected best PKW configuration. 72% reduction of afflux was observed for both 1,000-year and 10,000-year return periods with PKW, in comparison with the Existing Spillway of the reservoir. Since the present work limited to a hydraulic engineering study based on empirical equations, physical model testing and subsequent alterations to the proposed configuration may require to match with the site-specific conditions. Detailed structural and geotechnical designs, cost-benefit analysis and a comprehensive study on environmental and social impacts are further required before implementing this proposal.

Acknowledgements

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Spillway Gates Operation Optimization using Fuzzy Logic: A Case Study of Victoria Dam in Sri Lanka

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Abstract

The spillway gate operation is crucial in real-time reservoir operation, particularly during high floods and unexpected abnormal inflows. Heavy rainfalls cause massive inflows to the reservoirs that consequently result in dam overtopping, downstream flooding, and severe damage to the dams. At times, abnormal inflows can occur, which the dam managers are unable to handle to operate spillway gates properly. The main reason is that the gradual increase in inflow over a relatively short period. The existing spillway gates operating procedure followed by the dam is less reliable and risky to follow during high floods and abnormal flows through the dam because that procedure cannot handle these situations effectively and efficiently. To overcome this crucial problem, an intelligent fuzzy logic model is proposed in this research. The methodology is illustrated through the case study of Victoria dam in Sri Lanka as the current spillway gate operation procedure of the Victoria dam is unable to manage this kind of high unexpected inflow effectively. To simulate the performance of the fuzzy model, a comparison is conducted with a past high flood event. The results demonstrate that the proposed fuzzy model gives better reliable results than operating from the existing spillway gate operating procedure at the Victoria dam. The overall result showed that during high inflows and unexpected abnormal flows, the proposed fuzzy logic model is a more reliable and efficient solution that delivers superior outcomes.

Keywords: Abnormal inflows, Dam overtopping, Fuzzy logic model, Spillway gate operating procedure.

1. Introduction

Annually, Sri Lanka receives more than 2000 mm of rainfall, equivalent to nearly 130 billion cubic meters of water. [1] The longest river in Sri Lanka, the Mahaweli River, receives 28000 million cubic meters (MCM) of precipitation annually, of which 9000 MCM is discharged to the sea as a result. [2] Therefore, reservoirs were built to store this massive amount of water flowing in the Mahaweli river for a variety of purposes, including generating hydroelectric power, flood control, and for agricultural irrigation. Each year, nearly 2000 MCM of water pass through the Victoria reservoir in Sri Lanka. [3] This massive inflow has the potential to result in serious disasters, causing dam overtopping and risking the dam safety.

To resolve those issues, the Victoria dam constructed a spillway gate structure, that operates according to the spillway gate operating procedure. [4] When an abnormal flow or high inflow occurs, the opening of gates does not operate efficiently according to the existing procedure, because it cannot provide appropriate gate openings according to the coming inflow. Because procedure should wait until a reservoir rises to a predefined water level. Because of the wide intervals between the reservoir levels of the existing procedure, it is a slow and unreliable mechanism. Therefore, dam managers have to carry out a manual decision-making process which may risk the safety of the dam.

The main problem is that the existing gate operating sequence procedure does not consider the rate of change of the reservoir level parameter. This parameter is very important in real-time scenarios like this because, in a very short period, the rate of rising water levels is high. Therefore, to overcome this problem, one of the soft computing techniques, fuzzy logic, can be used to develop an intelligent fuzzy logic model to determine the gate opening effectively, especially in high and abnormal inflow situations. This can help to optimize the spillway gate operation for effective real-time reservoir management and improve efficiency.

2. Literature Review

The optimization of spillway gates has become a central concern in dam management. A particularly promising approach has emerged through the application of fuzzy logic, which offers the means to handle complex, imprecise, and dynamic hydrological data.

Karaboga et al. proposed a Fuzzy Logic Control (FLC) for the Catalan dam with the purpose of operating spillway gates during any flood of any magnitude that is not predicted beforehand. [5] The simulation results were produced by three different methods, which are the manual method, Proportional Derivative (PD) type control, and the proposed FLC. The results of FLC gave more smooth outflow hydrographs than other methods.

A Fuzzy Logic System (FLS) is to adjust the dam elevation as per rule level, was suggested by Nigam and Yadav. [6] They have considered three strategies according to the monsoon rain periods and constructed three fuzzy models. Their FLS produces smoother outflow hydrographs than those obtained by actual observed hydrographs. M. Imran et al. present the design of a flow controller for dam gates using a FLS. The basic concept of their study is that this system will show an extreme response in extreme conditions otherwise, it will only show normal results. [7] The system was checked for practical results using a microcontroller circuit and showed promising results. A. Bagis et al. present a FLS for reservoir control using integrated it with the Tabu Search Algorithm (TSA). [8] The Membership Functions (MFs) are refined using TSA to improve the performance of the FLS. The research presented by A. Heidari explained the principles for improving flood mitigation operations in multipurpose dams and maximizing reservoir performance during flood occurrences, with a focus on the real-time operation of gated spillways. [9]

Some of the research in this field presents how to optimize the outflows. E. A. Soentoro et al. proposed a FLS to determine the maximum total release volume based on the water availability. [10] They considered a monthly release to be equal to or more than monthly demand.

It is evident that there is a compelling need for innovative and adaptive approaches, especially during high unexpected inflow situations. Here, the inflow rate is considered, which is less observed in related studies, as it can give more accurate results. The use of fuzzy logic for spillway gate operation is a study that is less observed in the Sri Lankan context. Therefore, this research aims to develop an intelligent fuzzy model to optimize spillway gate operation to help with proper and efficient reservoir management in dams.

3. Study Area

The Victoria dam, which is the highest dam in Sri Lanka, taken as the case study. The dam construction consists of a high concrete arch dam, a reservoir, a four-mile-long tunnel, and a power station. Its main purposes are hydroelectric power production and irrigation.

Figure. 1: Victoria dam in Sri Lanka.

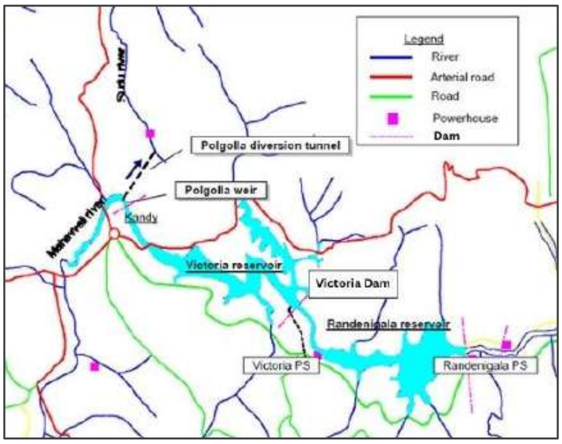

Figure. 2: Location of Victoria reservoir.

The dam consists of eight radial spillway gates that are designed to open automatically in a sequential manner when pre-determined water levels are reached. [4]

3.1. Dam and Reservoir Characteristics

The reservoir levels indicated in the Figure 3 as follows:

- Highest Flood Level (HFL) = 441 m
- Operational Maximum Flood Level (OMFL) = 438.64 m
- Full Supply Level (FSL) = 438.04 m
- Overspill Crest Level (OCL) = 430 m
- Minimum Supply Level (MSL) = 370 m

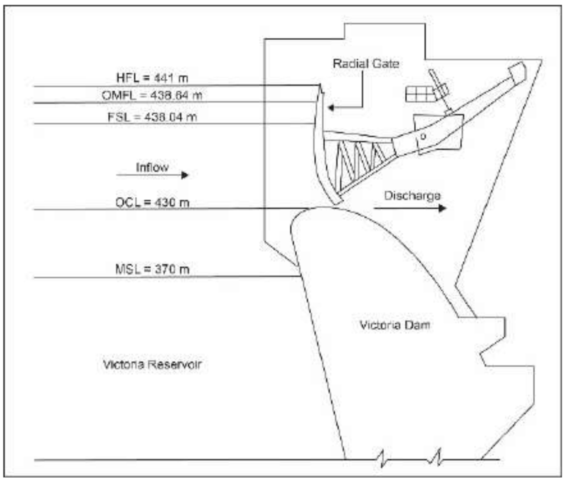

Figure. 3: Schematic sketch of the Victoria dam gated spillway.

For this research study, the spillway operating range taken as from FSL to OMFL. When the reservoir water fills up to the FSL the gates should start to discharge water to keep reservoir level not to exceed the OMFL.

4. Methodology

Fuzzy logic is one of the soft computing techniques because it deals with uncertainty and imprecision in decision-making. [11] It can incorporate intermediate values like partially true and partially false. FLS consists of four main components.

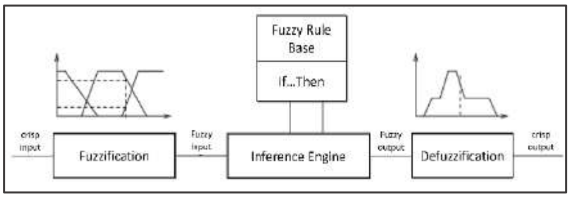

Figure. 4: Fuzzy logic system architecture.

Fuzzification converts the crisp number into fuzzy values. It is usually created with fuzzy sets that are defined by a MF. [12] Rule base used for storing the set of rules, MFs and the If-Then conditions given by the experts are used for controlling the decision-making systems in general form:

IF antecedent proposition
THEN consequent proposition

The number of rules is determined by the number of input parameters along with the MFs. Fuzzy Inference Engine (FIE) determines the degree of match between fuzzy input and the rules. [13] Defuzzification, takes the fuzzy set inputs generated by FIE, and then transforms them into a crisp value. The universe of discourse refers to the complete set of values that are relevant to a particular problem within a FLS.

4.1. Data Collection and Analysis

Figure 5 represents the discharge at one-meter steps of gate opening vs reservoir level. [4] The specimen calculation is used to calculate discharge according to the discharge curve.

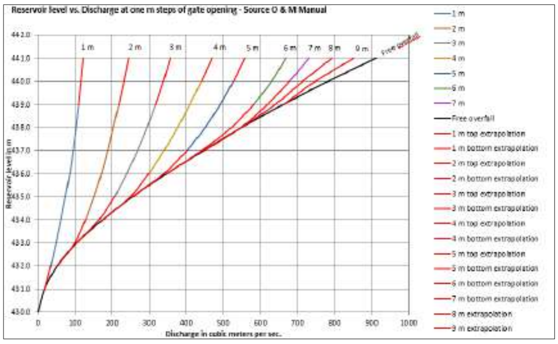

Figure. 5: Discharge curve of Victoria dam.

In high or abnormal inflow situations the coming inflow can varies and according to that the water level varies. Considering the fact, the gate openings can also need to be adjust per the real time requirement. The storage capacity of the Victoria dam according to the reservoir level data used to calculate the amount of inflow rate.

4.2. Model Parameters

The input parameters for the fuzzy logic model are the inflow rate (I) and rate of change in reservoir level (dH).
Inflow rate represents the volume of water coming into the reservoir in a time period (t). Universe of discourse for the parameter I is selected as [0,4866]. For simulation purposes, the time duration for this research was selected as 30 minutes based on past data analysis of the Victoria dam.

$$\mathrm{I} = \frac{(S_2 - S_1)}{t} \qquad (1)$$

Where I Inflow Rate
S_2 Reservoir capacity after t time (MCM)
S_1 Reservoir capacity before t time (MCM)
t Time duration to rise or fall of reservoir capacity (s)

The rate of reservoir level rising or falling represented by the parameter dH. Universe of discourse for the dH selected as [-1, +1].

$$dH = \frac{(CRL - PRL)}{(HFL - OCL)} \tag{2}$$

Where *dH* Rate of change in reservoir level
CRL Current Reservoir Level
PRL Previous reservoir level
HFL Highest flood level (441m)
OCL Overspill Crest Level (430m)

The output parameter for the fuzzy model is Gate Opening (GO) which predicts the appropriate gate operating positions. Universe of discourse for the parameter GO selected as [0,9].

4.3. Fuzzy Sets

Each fuzzy variable is formed into a fuzzy set, which is a group that represents the conditions of each variable. Linguistic variables are used to simplify the expression of rules. The prediction accuracy is improved by defining more fuzzy sets for each input variable. [14] The Table 1 includes the fuzzy terms and fuzzy sets for each parameter.

4.4. Membership Functions

MFs for each parameter are selected based on human-expert experience and on observed datasets. Triangular MFs used for all the parameters because it simplifies the computation in the Fuzzy Inference Engine.

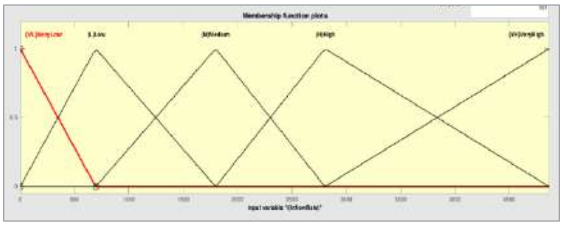

Figure. 6: Membership function of inflow rate.

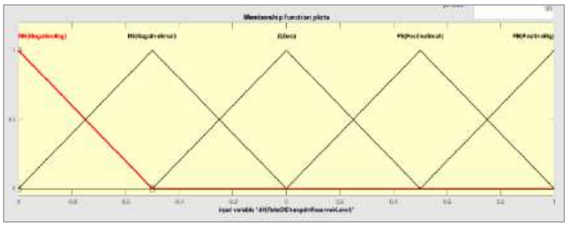

Figure. 7: Membership function of rate of change in reservoir level.

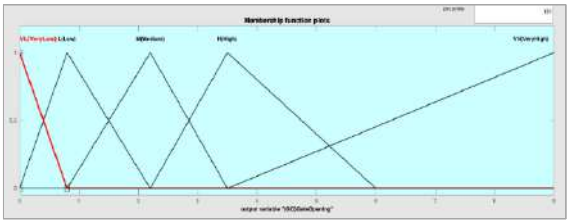

Figure. 8: Membership function of gate opening.

4.5. Rule Base Structure

Fuzzy rules for any reservoir system are formulated based on human-expert knowledge. [15] Specially the experience of the dam experts with the operation of spillways in high inflows.

Total of 25 rules were constructed for the study and Table 1 shows the fuzzy rule base with MFs ranges. Some examples of rules in the form IF-THEN as follows:

- **IF** Inflow Rate (I) is Very Low and Rate of change in reservoir level (dH) is Negative Big **THEN** Gate Opening (GO) is Very Low.

- **IF** Inflow Rate (I) is High and Rate of change in reservoir level (dH) is Positive Small **THEN** Gate Opening (GO) is High.

4.6. Fuzzy Inference Mechanism

For the input and output operations, the logical implication operators selected as AND and OR are Min and Max, respectively. The output of each rule is determined by the Mamdani max-min inference method. It is because it has more intuitive and easier-to-understand rule bases and well-suited to expert system applications.

4.7. Defuzzification Method

After several simulation trials, the Center of Gravity (COG) method was selected for this study because it performed better than the other defuzzification techniques.

Table 1: Fuzzy Rule Base Structure

Fuzzy Set for I	Domain of I			Fuzzy Set for dH	Domain of dH			Fuzzy Set for I	Domain for GO		
Very Low (VL)	0	0	695	Negative Big (NB)	-1	-1	-0.5	Very Low (VL)	0	0	0.8
Low (L)	0	695	1798	Negative Small (NS)	-1	-0.5	0	Low (L)	0	0.8	2.2
Medium (M)	695	1798	2803	Zero (Z)	-0.5	0	0.5	Medium (M)	0.8	2.2	3.5
High (H)	1798	2803	4866	Positive Small (PS)	0	0.5	1	High (H)	2.2	3.5	6.0
Very High (VH)	3463	4866	4866	Positive Big (PB)	0.5	1	1	Very High (VH)	3.5	9.0	9.0

The model implemented using fuzzy logic toolbox in Matrix Laboratory (MATLAB) version R2018a software created by MathWorks.

Figure 9 represents the surface viewer model, which provides a three-dimensional curve that maps the input parameters I and dH to the output parameter GO.

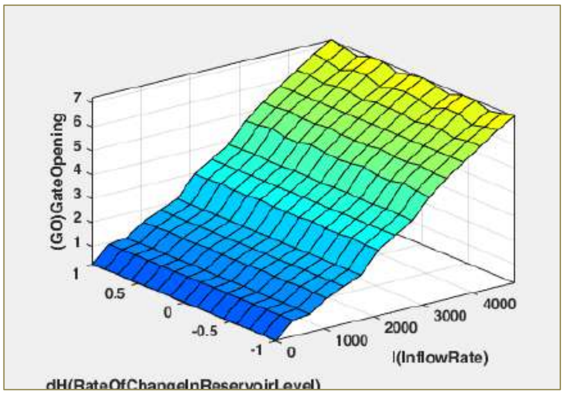

Figure. 9: Surface graph of the fuzzy model.

4.8 Operating System User Interface

The spillway gate operating system Graphical User Interface (GUI) was implemented using MATLAB app designer and functionalities were programmed.

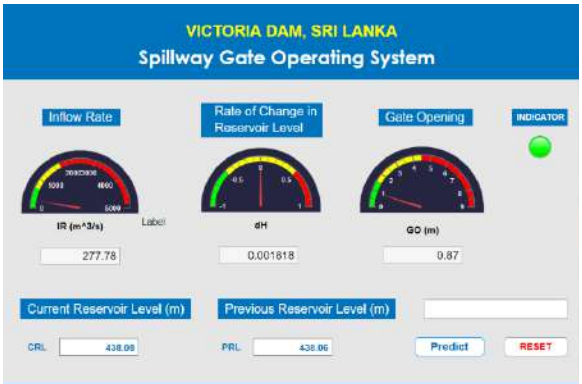

Figure. 10: Membership function of gate opening.

The GUI helps to get predicted gate openings easily by giving inputs CRL and PRL.

5. Results and Discussion

The high inflow event that occurred in December 2014 in Victoria is taken for the case study. The high inflow lasts from 20th December to 5th January 2015 and the maximum inflow between 25th and continued up to 27th December. [16]

Figure 11 shows that, the fuzzy model operates spillways effectively according to the rising and falling of the high inflow, while the existing procedure cannot operate according to parameter I.

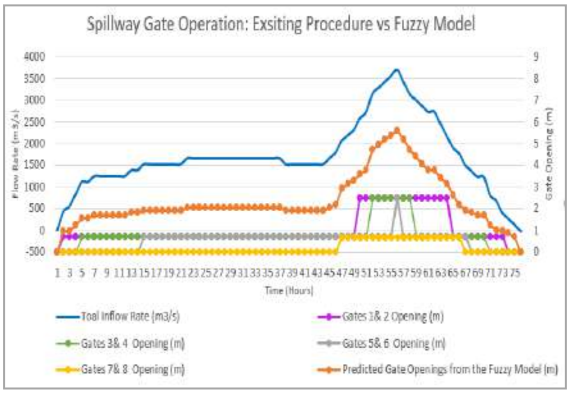

Figure. 11: Spillway gate operation comparison with existing procedure vs fuzzy logic model predictions.

A comparison between total observed discharge and total discharge according to the predicted gate openings is showed in the Figure 12. The data that shown as observed discharge are, how dam operators operated and discharged the inflow, at the past inflow event in, 2014. The results indicate that discharging a higher quantity according to the high inflow ensures dam safety by preventing dam overtopping and it optimized the spillway operation.

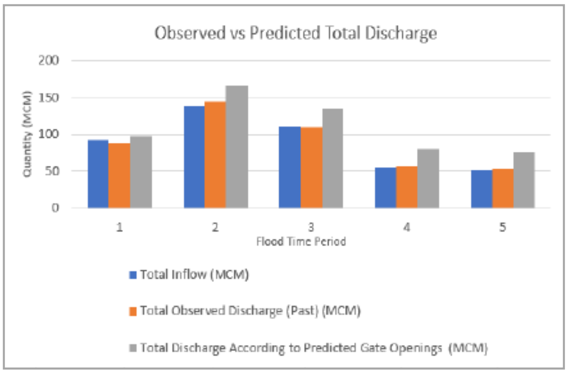

Figure. 12: Observed vs Predicted total discharge.

The performance of the fuzzy model is evaluated by the coefficient of determination (R^2). The R^2 value for the fuzzy model is 0.98 is within limits, which suggests better model performance, and the model's predictions are closely aligned with the observed values, demonstrating the reliability and accuracy of the FLS in simulating spillway gate operations.

6. Conclusion

In Sri Lanka, all the dams followed a fixed or manual spillway gates operation. In high or abnormal inflows, it is difficult to handle the spillway gates operation using the existing methods in the dams because the reservoir level is rising in a relatively short period of time. As a result, it can cause dam overtopping problem. This research study presents a reliable and efficient method using fuzzy logic to optimize the operation of spillway gates effectively in high-inflow situations by considering a case study of the Victoria dam. The fuzzy logic model is compared with a past high-inflow situation occurred at Victoria dam. Compared to existing operation, our fuzzy logic model responds dynamically to changing inflow conditions, ensuring a more precise and adaptive control strategy. Therefore, using a fuzzy logic model has the potential to enhance safety by providing a more responsive and optimized approach to spillway gate operations during high inflows to prevent the risk of dam overtopping and ensure dam safety.

The study intends to provide insight into how the fuzzy model can be effectively utilized by dam operators. For future developments, the model can be refined by incorporating additional parameters, and it can also be expanded by using other artificial intelligence techniques. Furthermore, when considering practical applicability, the model can be integrated with the existing systems in the dam to enhance dam safety in real-time dam operations.

Acknowledgements
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Operation of Kotmale Reservoir for Flood Control

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Abstract

Kotmale reservoir is considered a flood-controlling reservoir in addition to its primary functions because of the reduced risk of flooding in the previous flood-prone lands along the Mahaweli River from Kotmale to Peradeniya. However, those areas are still vulnerable to flooding because of the enormous Mahaweli River discharge, which comes from the upper Mahaweli Catchment, which begins in the Hatton area and the Kotmale reservoir, which discharges into the Mahaweli River via Kotmale Oya. The combination of the releases from the Kotmale reservoir and the flood peaks from the Upper Mahaweli catchment amplifies the resulting flood peaks, which increase the risk of inundation in populated downstream areas. Hence, it is important to operate the Kotmale reservoir to avoid flood peaks, which can only be done by forecasting the catchment's flood hydrograph. Thus, a rainfall-runoff model was developed for the Upper Mahaweli catchment using HEC-HMS software. Using historical data records as a case study, the model was used to show how the timing of Kotmale reservoir's spillway discharge release affected the resulting flood hydrograph at the confluence. As a result, in comparison to a base scenario, the flood peak at the confluence increases by 6.7 m^3/s when reservoir releases are advanced by 24 hrs, whereas it decreases by 21.6 m^3/s and 34.6 m^3/s when spillway discharges are advanced by 36 hrs and 48 hrs, respectively. Accordingly, when the releases were advanced by 36 hrs and 48 hrs, respectively, the peak discharge was reduced by 9.5 % and 15.2 % in comparison to the base scenario. The study serves as an example of how to use a rainfall run-off model in decision making regarding the operation of the Kotmale reservoir.

Keywords: Flood Control, HEC-HMS, Kotmale Reservoir, Mahaweli River, Reservoir Operation

1. Introduction

Kotmale Reservoir is the second uppermost reservoir of the Mahaweli Reservoir cascade system, which has been built damming the Kotmale Oya; one of the major tributaries of the Mahaweli River in Sri Lanka. Kotmale reservoir stores water for irrigation and hydropower generation. Other than its primary functions, the Kotmale reservoir plays the role of flood control. The direct catchment of Kotmale reservoir is 561 km^2 and the Probable Maximum Flood (PMF) was estimated to be 5,598 m^3/s with a time base of 45 hrs [1]. The area previously subjected to floods, from Kotmale to Peradeniya was developed rapidly due to the reduced risk of flooding and now vulnerable to significant damage due to an extreme flood event that could occur [2]. The flood-prone area receives flood from the upper Mahaweli catchment which begins in Hatton area as well as through the Kotmale Oya which carries releases from the Kotmale reservoir and confluence with the Mahaweli River at Ulapane. Coinciding of releases from the Kotmale reservoir and the flood peaks from the Upper Mahaweli catchment amplifies the resulting flood peaks at the confluence, which increases the risk of inundation in populated downstream areas. Hence, it is important to operate the Kotmale reservoir to avoid flood peaks, which can only be done by forecasting the catchment's flood hydrograph. Figure 1 demarcates the project area within the Mahaweli Catchment and Figure 2 indicates important locations within project area.

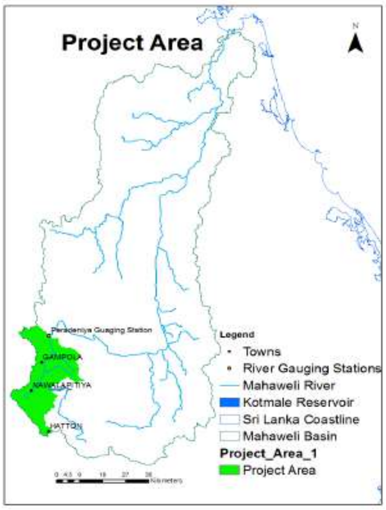

Figure 18 Project Area in Mahaweli-basin

Real-time flood forecasting models are used worldwide for flood prevention [3, 4, 5]. But, the limited weather data and river flow data available in real-time for the catchment, constrain the development of a real-time flood forecasting model. The Dam and Reservoir Control Centre, Polgolla operated by the Mahaweli Authority of Sri Lanka has initiated the real-time monitoring of reservoir status and weather at the major dam sites in the Mahaweli reservoir cascade system. Further, weather forecast data from different sources are reordered and analysed with actual data for accuracy and suitability for the dam sites. The centre plans to implement real-time reservoir control in future projects.

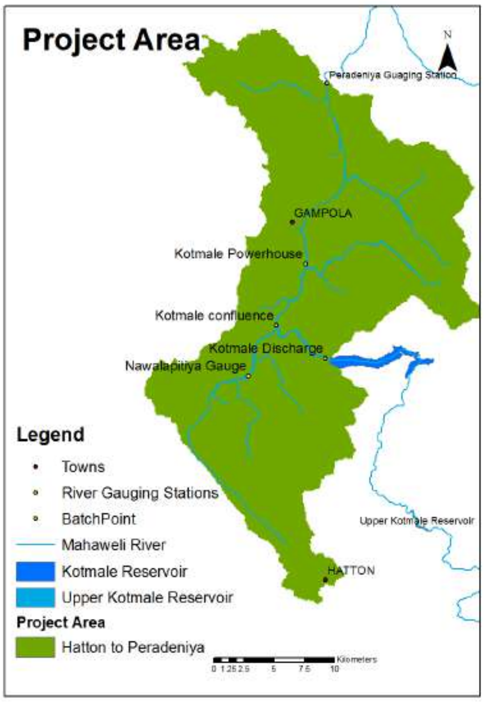

Figure 2 Project Area with important locations

Therefore, this study focuses on developing a rainfall-runoff model using the historical rainfall and river flow data, to demonstrate the ways by which flood peaks at the confluence could be reduced by varying the timing of the Kotmale reservoir's spillway discharge release. Peradeniya river flow gauging station operated by the Irrigation Department of Sri Lanka was set as the lower boundary of the rainfall-runoff model. Downstream of the Kotmale dam where the discharge from Kotmale reservoir is joined the Kotmale Oya and then directed to the Mahaweli River was set as an upper boundary of the model. The Nawalapitiya river gauging station operated by the Irrigation Department of Sri Lanka was introduced to the model as a junction with recorded data to fine-tune the calibration and validation process of the model. Freely available HEC-HMS software package was used to develop the rainfall-runoff model. Sets of historical data on hourly basis which are relevant for different rainfall events were used in the model for calibration and validation process as well as for the case study by which influence of the timing of spillway discharge release of Kotmale reservoir on variation of the flood peak at the confluence was demonstrated.

2. Methodology

The purpose of the study is to understand the catchment response to rainfall events, and spillway discharges which will support decision making in reservoir operation. Therefore, the model to be developed shall be capable of generating runoff discharge at a certain location of the river reach, which will result from real-time or forecasted rainfall events. Hence, it was decided to develop an event-based rainfall-runoff model for the catchment.

The methodology of the study is shown in Figure 3.

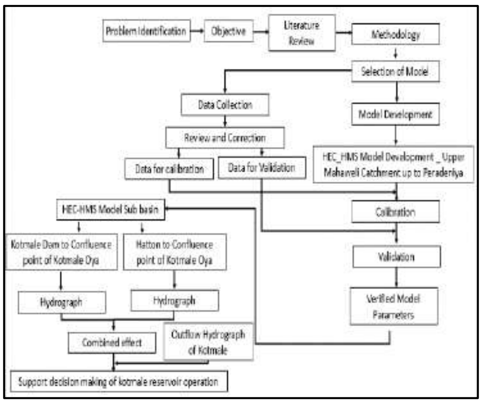

Figure. 3 Methodology of the Research

2.1 Data Collection

To develop the HEC-HMS rainfall-runoff model, the basin characteristics are to be obtained. For this purpose, the HEC-GeoHMS tool extension in Arc GIS software was used. As input to the HEC-GeoHMS software, a digital elevation model which was created from a 5m contour survey map of the study area was used. To calculate the curve number values, soil map and land cover map of Sri Lanka were used. Sub basins, important junctions, upper boundaries, and outlets were defined in the HEC-GeoHMS process and hence the basin model was created. The basin characteristics were calculated accordingly to use as the initial values for the parameters of the HEC-HMS rainfall-runoff model processes. Figure 4 shows the graphical view of sub-basin properties of catchment up to Peradeniya.

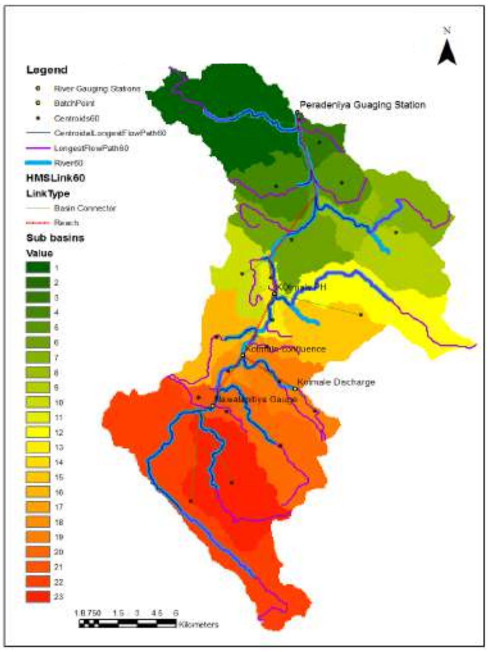

Figure 4 Graphical View of Sub-basin Properties of Catchment up to Peradeniya

The meteorological Model, one of the crucial components of the HEC-HMS Model, was also generated using the HEC-GeoHMS tools with given inputs of rainfall gauging station locations. The

Department of Meteorology of Sri Lanka confirmed that no rainfall gaging stations are available within the selected study area itself with rainfall data in lesser than 24-hour intervals. However, in the development of an event-based rainfall-runoff model to obtain the project objective, rainfall data with 24-hour intervals are not sufficient. Therefore, four stations with hourly rainfall data that are close to the study area or within the Mahaweli catchment were selected for the analysis. They are Katugastota, Kitulgala, Nuwara Eliya, and Norwood which are located within the Mahaweli catchment or close to the Mahaweli catchment within a 7.1 km buffer from the boundary of the catchment. Another set of stations within the study area with daily rainfall data was used to check the pertinence of selected gauging stations. Accordingly, the Pearson coefficient was calculated with the daily rainfall data and the correlation of Nuwara Eliya, Norwood, and Kithulgala daily rainfall was checked with Nawalapitiya, Watawala, and Kotmale rainfall data.

Table 1 Pearson Coefficient among rainfall gauging stations

Stations	Norwood	Kithulgala	Nuwara Eliya
Nawala-pitiya	0.58	0.39	0.19
Watawala	0.66	0.67	0.19
Kotmale	0.63	0.65	0.24

In different studies Pearson's correlation coefficient has been used to justify the correlation between two data sets and the significance of the correlation has been classified from very weak to very strong [6]. Accordingly, the Nuwara Eliya rainfall gauging station which shows a weak correlation with Nawalapitiya, Watawala, and Kotmale rainfall gauging stations was omitted from the analysis. Kithulgala and Norwood rainfall gauging stations show a moderate to strong correlation with rainfall gauging stations within the study area. Also, the Katugastota being the nearest rainfall gauging station to the river discharge gauge at Peradeniya and located within the Mahaweli catchment was selected to use in the analysis.

The rainfall data available were to be distributed to each sub-basin of the study area. The Theissen polygon weights were calculated for the catchment using the three rainfall gauging stations selected and the calculated weights for each sub-basin within the catchment were used as input to the HEC-HMS model.

2.2 Outlet and Sources

Runoff generated in the Kotmale catchment above Kotmale dam is connected to the Mahaweli River as released from the Kotmale reservoir. Furthermore, Kotmale spillway discharge is the crucial input variable of the study. Therefore, the Kotmale releases and Kotmale powerhouse releases were added to the model as source 1 and source 2, respectively. The Kotmale reservoir release and Kotmale powerhouse release data are available in MCM/day. However, on the days when spillway discharge is present, the hourly discharge data is available for both the spillway and powerhouse. Locations of the Kotmale discharge and Kotmale powerhouse are shown in the Figure 2. Once the HEC-HMS basin model is set up, it is shown as given in the Figure 5. The important junctions, river reaches, sources and basins which are crucial in the further analysis are given Figure 9.

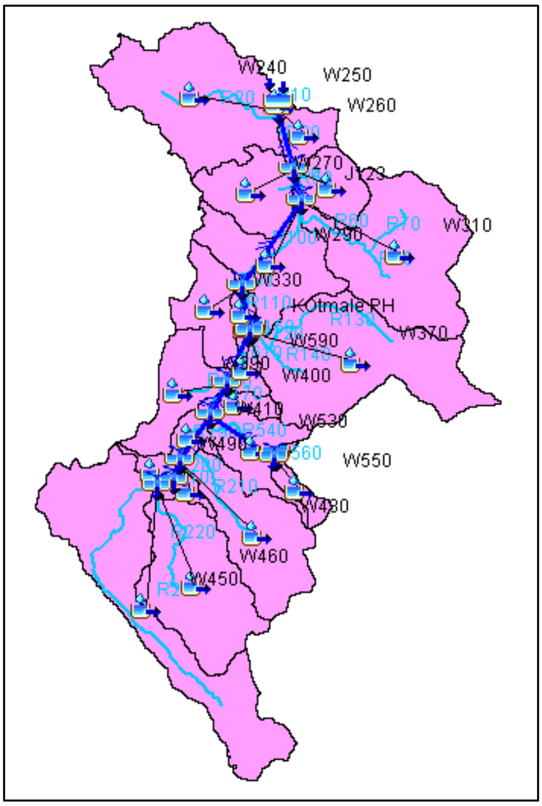

Figure 5 HEC-HMS Model with Sub-basins, Sources, reaches, links and Junctions

The basin model and meteorological model with calculated initial parameters, which were obtained from HEC-GeoHMS tools, were exported from Arc GIS to HEC-HMS software.

2.3 HEC-HMS Model Setup

Once the Basin Model was created, the HEC-HMS model was defined with processes as indicated in Figure 6. In this study, the Natural Resources Conservation Service (NRCS), previously Soil Conservation Service (SCS) curve number method, SCS transform method, and recession method were used for loss, transform, and base flow processes in the rainfall-runoff model respectively. The Muskingum routing method is used for the routing.

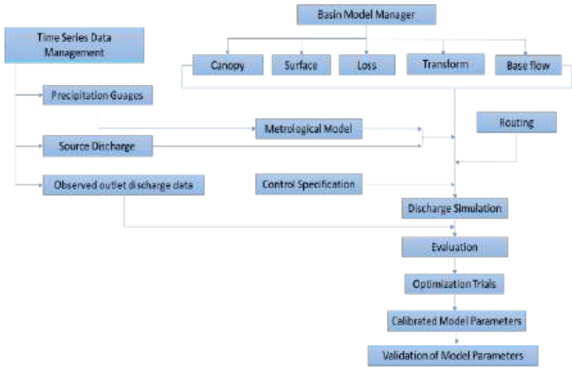

Figure 6 HEC-HMS Model Setup

The selected precipitation gauges were connected to the Meteorological model under time series data management. Source discharges and recorded outlet and junction discharges were also fed to the model under time series data management. Then the control specifications were defined and simulation was done. To calibrate the model, optimization trials were run for different parameters for each sub-basins separately. Once the model validation did not provide satisfactory outcomes with calibrated parameters, the model calibration was repeated and process was continued until the model validation also provided satisfactory outcomes.

2.4 Calibration and Validation of the model

The model was calibrated to both Nawalapitiya and Peradeniya River gauging stations in the Mahaweli River to improve the accuracy of the model.

For calibration of the model, the peak flow event of 825.2 m^3/s that occurred on the 12th of May 2012 at Peradeniya was selected. As input to the model, hourly data starting from 11-05-2012 01:00 hr to 16-05-2012 23:00 hr were selected for rainfall gauging stations and river gauging stations. No releases from the Kotmale reservoir were found for the selected time slot. The release from the Kotmale power station was added to source 2 as hourly data in m^3/s.

The normalized objective function, the Nash–Sutcliffe efficiency, and the percentage bias are quantitative measures for the simulation skills. These metrics are commonly used to evaluate rainfall–runoff models [7]. Accordingly, after several trials with model optimization, the model was calibrated first to the Nawalapitiya gauging station and then with that parameter values to the whole catchment up to Peradeniya. Values of 0.971 and 0.84 were obtained for Nash–Sutcliffe efficiency for Peradeniya and Nawalapitiya, respectively. Comparison of observed and calculated outflow graphs at Peradeniya for model calibration and validation are given in Figure 7 and Figure 8, respectively.

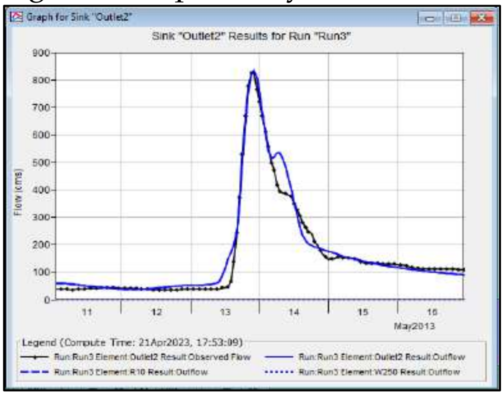

Figure. 7 Model Calibration at Peradeniya Outlet

For validation of the model, the peak flow event of 239.31 m^3/s that occurred on the 7th of September 2013 at Peradeniya was selected. As input to the model, hourly data starting from 06-09-2013 00:00 hr to 08-09-2013 23:00 hr was selected for rainfall gauging stations and river gauging stations. For the selected period no releases from the Kotmale reservoir were recorded. The release from the Kotmale power station was added to source 2 as hourly data in m^3/s. Values of 0.915 and 0.569 were obtained for Nash–Sutcliffe efficiency for Peradeniya and Nawalapitiya respectively.

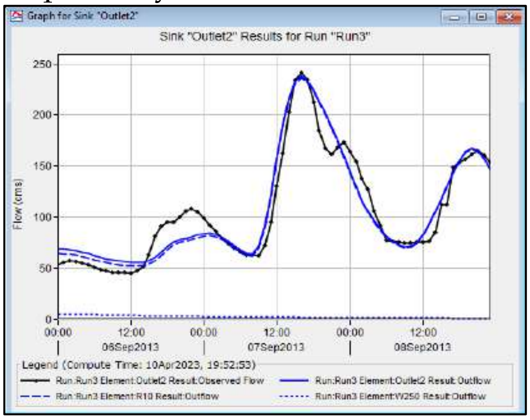

Figure 8 Model Validation at Peradeniya Outlet

2.5 Sensitivity Analysis

To operate the Kotmale reservoir by reducing flood peaks at the confluence, the flood hydrograph at the confluence and then the variation of that graph with outflow-generated form Kotmale reservoir have to be obtained.

As indicated in Figure 9, the sub-catchment W530 of the HEC-HMS model is the catchment below Kotmale Dam which runs up to the confluence of Kotmale Oya. The releases from Kotmale Oya, both spillway releases and bottom outlet releases which are given as source-1 are directed to the confluence through the river reach R540. For the considered period release from the bottom outlet was zero. The confluence of Kotmale Oya is given by J93 which is junction 93 in the model. River reach R190 is the reach flowing through the W410 sub-catchment, which is just above the confluence of Kotmale Oya. The entire outflow generated in the Mahaweli catchment starting from the Hatton area up to the confluence is directed to J93 through R190 river reach. The catchment area from Kotmale Dam to the confluence is 93.01 sq.km while the Catchment area starting from Hatton to the Confluence of Kotmale Oya is 191.7 sq.km.

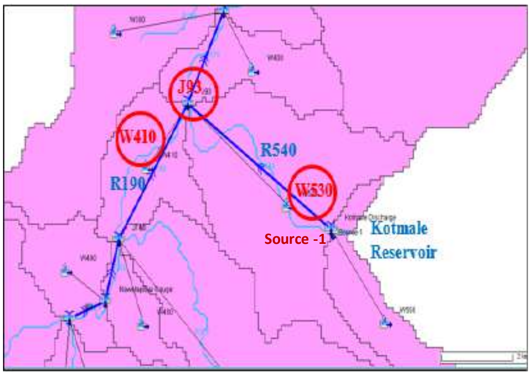

Figure 9 Important Basins, Junctions and River Reaches in HEC-HMS Model

2.6 Comparison of lag time

The lag time of each sub-basin plays an important role in the shape and time to peak of a resultant outflow hydrograph of a particular location. When the actual situation is considered as discussed in Figure 9, two sub-basins connect at the confluence of Kotmale Oya. Therefore, the hydrograph of the combined flow of R190 and W410 compared with the hydrograph from the combined flow of W530 and reached R540 as given in Figure 10 using the outputs hydrographs of the same model by which model calibration was performed. In the model-calibrated data set, the spillway discharges were zero for the considered period. The graph was generated using time series data table at Junction 93 of the model used for calibration.

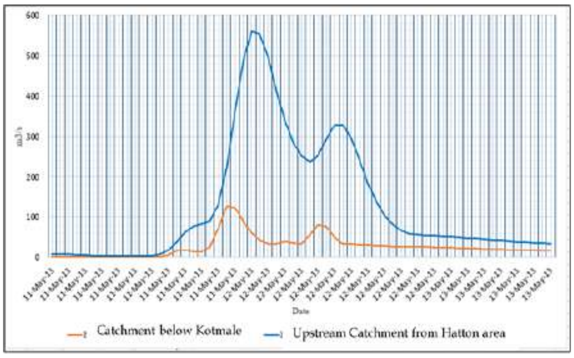

Figure 10 Comparison of lag time of main river at Kotmale Confluence and from Kotmale Oya

It is observed that the peaks of hydrographs for two catchments have three hours of difference for the selected rainfall event. Therefore, the peak flood hydrograph of the below Kotmale catchment reached the confluence of Kotmale Oya before the discharge from the upstream catchment of the Mahaweli River from the Hatton side.

Therefore, passing spillway discharge through the confluence before the flood from upstream of Mahaweli River starting from Hatton reaches the confluence would reduce the peak discharge of the hydrograph at the confluence. Coinciding flood peaks from both sub-basins increase the peak discharge of the resultant flood hydrograph at the confluence point. This could be done by advancing spillway releases from the Kotmale reservoir and hence allowing retention volume at the reservoir to hold the upcoming flood from its catchment. However, a base case scenario with spillway discharges is needed to see the real impact on the outflow hydrograph at confluence as a result of advanced spillway releases.

2.7 Further Analysis

A separate peak flood event was selected where the Kotmale reservoir releases are also available on an hourly basis for further analysis. The model with calibrated parameters was run for the selected event with actual rainfall data of the selected rainfall gauging stations and actual spillway discharge data. Accordingly, the model was run from 09-06-2013 to 00:00 hr to 16-06-2013 23:00 hr. Figure 11 shows that the outflow at junction 93 has drastically increased with releases from the Kotmale reservoir, which is indicated by the outflow graph of R190.

In the sensitivity analysis, this event was considered as the base event to compare the effect of the outflow hydrograph with the variation of spillway discharge of the Kotmale reservoir. To obtain the effect of time of the spillway releases on the flood peak at the confluence, the releases of Kotmale reservoir are floated along the timeline to see the variation in the flood peak at junction 93, the confluence of Kotmale Oya. The model was run with the same reservoir release advanced in 24 hrs, 36 hrs, and 48 hrs separately. The change in total outflow hydrographs was observed for those cases and compared with the base case of actual reservoir releases.

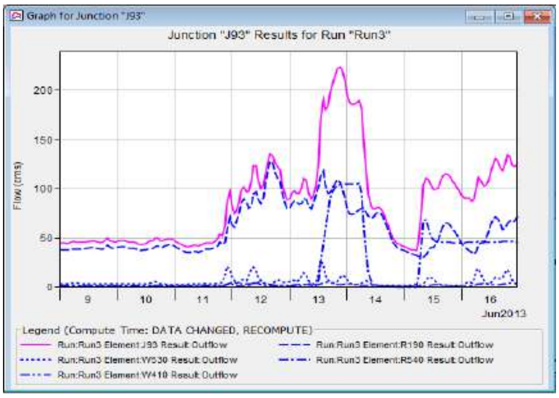

Figure 11 Outflow hygrographs at junction 93 for base case scenario

3. Results

In the graphs given in Figure 12, it is observed that the flood peak at the confluence of Kotmale is increased by 6.7 m^3/s when reservoir releases are advanced by 24 hrs. In graphs 13 and 14, the flood peak at confluence has reduced by 21.6 m^3/s and 34.6 m^3/s for releases of spillway discharges advanced by 36 hrs and 48 hrs, respectively. The occurrence of flood peaks is also advanced by 8 hr and 5 hr, respectively.

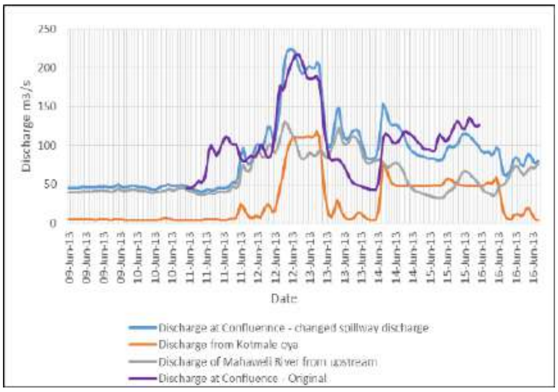

Figure 12 Advancing Spillway Discharge by 24 hrs

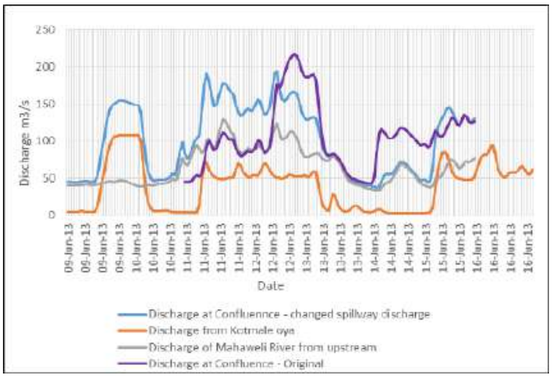

Figure 13 Advancing Spillway Discharge by 36 hrs

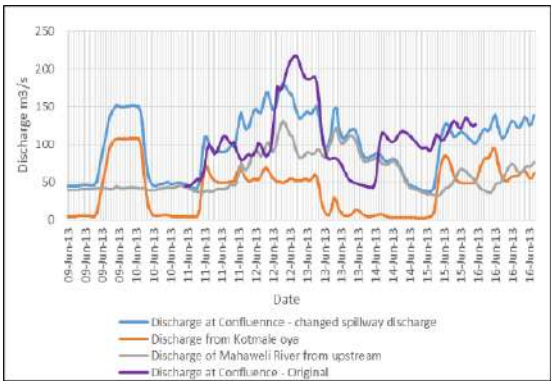

Figure 14 Advancing Spillway Discharge by 48 hrs

For the base scenario, the maximum hourly spillway release is 227.38 m^3/s. Accordingly, the reduction of peak discharge at confluence compared to the base case scenario was 9.5 % and 15.2 % concerning peak spillway when the spillway releases are advanced by 36 hrs and 48 hrs, respectively.

4. Discussion

In the study, a rainfall-runoff model was developed for a selected critical portion of the Mahaweli Catchment where the outflow from Kotmale reservoir being one of the most important flow-regulating and flood-controlling reservoirs is included. Since the Kotmale reservoir is the second uppermost reservoir in the Mahaweli cascade system, it plays an important role in the real-time operation of the reservoir cascade system. Releasing water from reservoirs of Mahaweli cascade system for irrigation and hydropower generation is controlled by the weekly decisions issued by the Water Management Secretariat. However, in the current situation, only the buffer volumes of reservoirs, rate of rise in reservoir level, and back-calculated inflow generated using reservoir levels are taken into account when operating the spillways of Mahaweli cascade system.

Due to limited access to data, especially the reservoir bed survey and operation data for the Upper Kotmale reservoir, this study was limited to the selected study area. Also, the primary concern of the study was to analyse the performance of the catchment above the confluence of Kotmale Oya starting from the Hatton area with spillway discharge from the Kotmale reservoir. However, a single model included with the catchment of Kotmale reservoir would do a similar sensitive analysis and ease the decision-making process of reservoir operation.

In this study, the historical rainfall data and spillway data were used to perform the sensitivity analysis to demonstrate methods by which the developed rainfall-runoff model could be run with predicted or real-time rainfall data to make decisions in operating the Kotmale reservoir in a manner that the flood peak at the Kotmale Oya confluence is reduced. Therefore, this study sets an example for methods that the forecasted flood hydrographs with rainfall data could be used in reservoir operations effectively, especially to avoid disastrous flood peaks in downstream river reaches.

The sensitivity analysis performed shows that rainfall prediction for 1 day, 1 ½ days, and 2 days would give a significant difference in flood peaks at the considered outlet. Since such rainfall prediction data are already available and being analysed currently at DRCC, Polgolla, the model can be used to support decision making in the reservoir once the most reliable source for rainfall predictions of the catchment is identified. However, in reality the scheduled water requirement for irrigation and hydropower generation shall be considered when advancing spillway releases.
In the literature review, it was found that HEC-RTS models have been widely used all over the world for real-time runoff generations from catchments and hence to control floods. Therefore, a similar model shall be suitable for DRCC to operate a reservoir cascade system in Mahaweli River. The rainfall-runoff model developed in HEC-HMS can be imported to the HEC-RTS software to further analyse the catchment performance [8]. Therefore, the rainfall-runoff model developed in the study can be used to develop an HEC-RTS model and shall be further improved with hydrological and weather data that will be available in future
Throughout the study, the absence of data required for the development of an event-based rainfall runoff model such

as hourly rainfall data within the catchment and hourly stream flow data in sub basin was noticed. Thus, to forecast the real-time outflow hydrographs of the catchment, an adequate number of rainfall gauges are to be installed within the catchment. Installation of additional gauging stations in the Mahaweli River and tributaries will also support the continuous calibration of the models with the new data acquisitions. Therefore, the study emphasises the importance of the installment of instruments that collect hydrological data in lesser time intervals than daily data. It is a crucial requirement for developing a real-time rainfall-runoff model that can be used in decision-making in reservoir operation.

Acknowledgments
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Applicability of electrical resistivity for identification of seepages in earthen dams in Sri Lanka (A case study in Thissa dam)

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Abstract

Most of the dams constructed across rivers in Sri Lanka are earthen dams. The main purpose of most of the dams in Sri Lanka is to reserve the excess water to use in dry periods. The Irrigation Department (ID) manages most of the large dams in Sri Lanka. Most of the dams that are managed by ID are earthen dams and some of them were constructed in kings' periods. In old dams, excessive seepages through the dam body are often observed. Clay and/or Cement grouting is used as the main treatment method for excessive seepages through a dam. Earlier, the dams with seepages were investigated and the treatments were recommended based on the borehole details the cost for which is comparatively high. Ground Electrical Resistivity Surveying (GERS) is an economical method which is currently used by ID to identify the high seepage locations.

Excessive seepages were visible in Thissa dam in Sri Lanka and the dam was investigated using geophysical methods before and after grouting. The resistivity survey was used to identify the seepage locations in the earthen dam using the anomalies of the Electrical Resistivity Tomography (ERT). The recommendations were given by the Engineering Geology Division (EGD) of the ID and the grouting treatments were also carried out by EGD.

During treatments in Thissa dam, both overburden soil and rock were grouted using clay and cement. Test holes were drilled to verify the grouting process. After the treatments, it was identified that the grouting plan was successful. The seepage could be considerably reduced by using clay and cement grouting. This paper discusses the applicability as well as the limitations of electrical resistivity surveying for identification of seepages.

Keywords: Clay-cement grouting, Seepage, Geophysical investigation, Ground Electrical resistivity survey

1. Introduction

The Thissa reservoir is situated in Thissamaharama in Hambanthota district about 250 km South from Colombo in Sri Lanka (Figure 1). The total storage volume of the reservoir is 4.32 million cubic meters and it is used mainly for irrigation purposes. The reservoir and dam were thought to have been constructed in the 3rd century BC, either by Mahanaga of Ruhuna or his successor Yatala tissa of Ruhuna. The maximum height of the dam is 4.88 m and its length is about 1493 meters. The irrigable area under Thissa reservoir is about 1113 acres. Geologically the Thissa reservoir is located in litho-tectonic unit of Vijayan Complex in Sri Lanka.

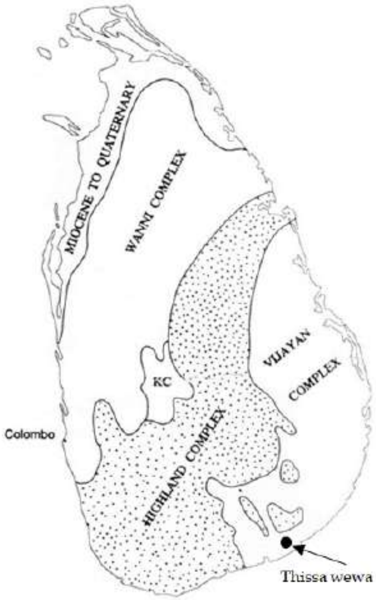


Figure 01: Location of Thissa reservoir

2. Literature review

Seepages or leakages through an earthen dam body are very common and it may even result in the dam failure. Proper monitoring and necessary actions have to be taken to control the seepage through the dam within acceptable limits. There are several methods to control the seepage; however, grouting is one of the popular methods used in treatment of high seepages. To make the grouting plan, it is important to identify the seepage paths as intense grouting have to be implemented for the selected locations. Borehole investigations provide important data for this purpose but it may miss some important information when the seepage water follows irregular paths inside the dam body. GRES provides a continuous profile which aids identification of seepage paths through the dam.

In the resistivity method an electrical current is introduced into the ground and the resulting potential distribution is measured. Typically one pair of steel electrodes is used to inject current and another pair is used to measure potentials (Lin et al. 2013). Resistivity distribution can be identified by determination of resistivity by using Ohm's Law ($V= IR$ where V is the voltage difference between two potential electrodes, I is the current and R is the resistivity.

GERS is a well-established technique for a broad variety of purposes in engineering and environmental ground investigations. It has been used for leakage detection on many occasions, with mixed results (e.g.Buselli and Lu 2001; Butler and Llopis1990; Panthulu et al. 2001; Sjodahl et al. 2005). A more effective, but more demanding approach is to carryout repeated measurements or long-term monitoring (Johansson and Dahlin 1996; Johansson et al. 2005).

3. Ground Electrical Resistivity Survey and grouting process

It was after few years of detecting several leakages along downstream (DS) face of the Thissa dam, the investigations were started and three locations (Figure 2) with excessive seepages could be identified. It was noticed that the roots of old dead trees exist at the seepage locations.

Figure 2 : Seepage locations

As a quick and economical investigation method in geophysics, the GERS was used here to identify leakage paths. Three sections were investigated using GERS as shown in Figure 2. The surveyed length of the section A is 80m (from 0+120 to 0+200), section B is 240m (0+580 to 0+820) & section C is 80m (1+100 to 1+180). Subsurface profiles were developed based on the resistivity values of bund and subsurface (Figure 3 to 5).

The variation of resistivity is related to the type of subsurface material, water content of the area, degree of compaction, and the structural features and weathering pattern of the rock.

In section A, the subsurface delineated up to around 17m in depth from Bund Top Level (BTL). A low resistivity zone was identified in between 0+160 & 0+178, which extends up to 8m depth.

In section B, the subsurface delineated up to around 45m in depth from Bund Top Level (BTL). A low resistivity zone was identified in between 0+710 & 0+820 which extends up to 15m depth.

In section C, the subsurface delineated up to around 17m in depth from Bund Top Level (BTL). A low resistivity zones were identified throughout the whole section, which extends up to 10m depth. According to the results obtained from resistivity survey most of the subsurface materials have relatively low resistivity values which is shown by bluish colour in the developed subsurface profile (Figure 3 to 5). When the conductivity is considered, this blue colour zone has comparatively higher conductivity. Conductivity is directly proportionate to the moisture content. Therefore, this blue colour area can be identified as higher seepage prone area.

It was recommended to grout the identified bund sections (Section A, B & C) maximum up to 12m depth in overburden with clay and cement, and in rock, up to 24m with cement grout to arrest water leakages as necessary. Two rows of grout holes were planned in zigzag pattern. Spacing between two holes in a row is 3m and the spacing between two rows is 1.5m having a 2.12m shortest distance between two nearest holes.

Grouting was carried out from 0+092 to 0+226 in section A, from 0+624 to 0+816 in section B and from 1+050 to 1+188 in section C. Grout taken amounts are shown in Figure 6, Figure 7 and Figure 8. It is clear from the figures that those stretches which have taken high amount of grout coincide with the identified low resistivity zones from GERTs. In section A, the identified low resistivity zone was between 0+160 and 0+178. Equally, high amount of grout was taken from 0+148 to 0+181 as shown in figure 6. In section B, the identified low resistivity zone was between 0+710 and 0+820. Equally, high amount of grout was taken from 0+700 to 0+812 as shown in figure 7.

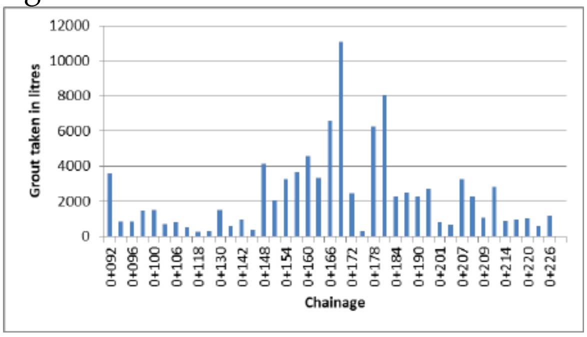


Figure 6: Grout taken in Section A

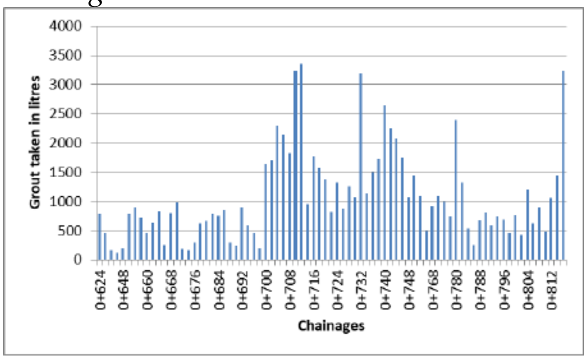
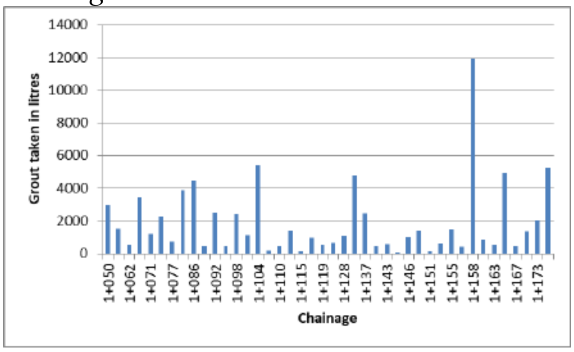
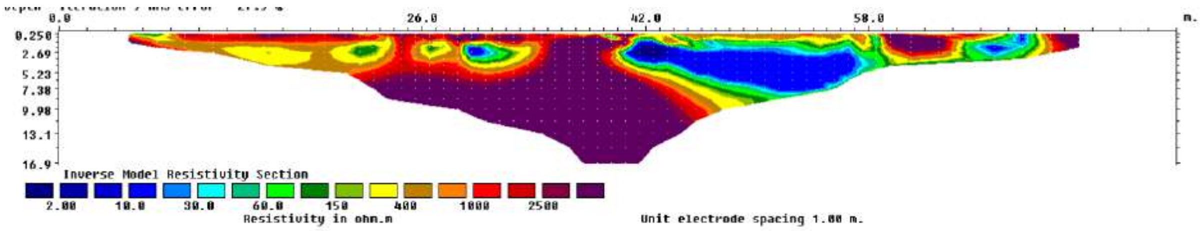


Figure 7: Grout taken in Section B



Drilling and Grouting was initiated in the upstream face. The ratios for cement grouting in rock was determined as 3:1 (water: cement by mass). However, the ratios varied as 5:1 and 2:1 as per the site condition and the grout taken amounts. The size of the drill holes was BX. All the drill holes were drilled at the dam crest. The downstream areas were visually monitored for changes in the discharge through water leakages to get an idea on the success of the grouting plan. Further, the grouting pressure was applied to match with the ground condition and the grouting depth.

Figure 8: Grout taken in Section C



In section C, low resistivity zones were identified throughout the section and equally, the amount of grout taken throughout this section is also high as shown in figure 8.

4. Success of grouting

The seepages which were visible in the downstream in the selected sections before the treatments are not visible after the treatments (Figure 6 & 7). It was noticed that the highest leakage which was near 1+140 was completely dried out. Further, the leakages near the sluice barrel were noticed as ceased.

Figure 3: Resistivity survey results in Section A (before grouting)

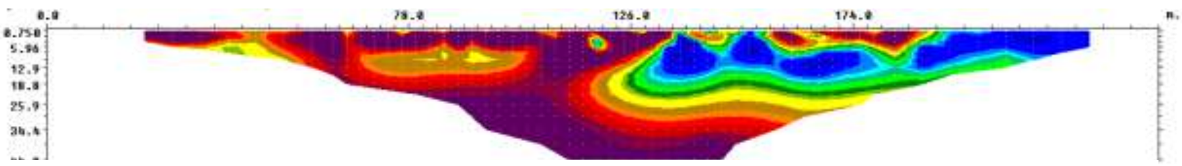


Figure 4: Resistivity survey results in Section B (before grouting)

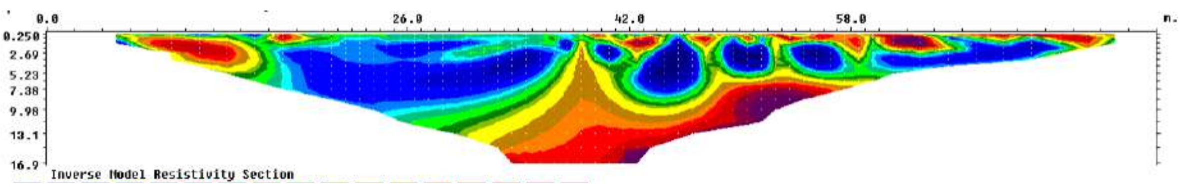


Figure 5: Resistivity survey results in Section C (before grouting)

Figure 6: Before grouting at section C

Figure 7: After grouting at section C

5. Conclusions

Indirect methods of geotechnical investigations were very much economical with reliable results for this kind of a project. It can be used to optimize the available financial budget. Especially for functioning dams, treatments should be carefully planned ensuring their safety and GERS profile are very helpful in gathering accurate and adequate information to prepare a practical plan for treatment of dams, therefore the accuracy of the investigations is a significant factor for the safety of the dam. Further, this method of investigation and treatments by grouting can be used for considerable control of seepage through the dam. The more expensive methods such as bore hole drilling could then be directed to the points of greatest importance.

During the grouting operations at the Thissa dam site different methods and techniques were applied concerning ground investigations, grouting works and controlling the outcome of the works. All these showed their applicability and efficiency.

The experiences gained, strongly points out that the use of GERT to identify seepages can be recommended as an economical method of identification and the results can be used in preparation of the grout plan.

Acknowledgements

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Emergency Relocation of the Left bank Sluice of Nachchaduwa Reservoir Built by King Moggallana II, A Case Study of Multidisciplinary Team of Irrigation Engineers to Overcome Challenges in Operation and Maintenance of Ancient Reservoirs

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Abstract
Nachchaduwa reservoir was built during the 6th century AD, by King Moggallana II, across Malwathu Oya mainly for drinking water supply and irrigation purposes while it also serves as a flood control reservoir to safeguard the city of Anuradhapura. The reservoir has been operated under the Irrigation Department since 1903 and this paper describes the technical aspects considered in finding a solution for the leakage observed in the left bank sluice in year 2010. The remedial measures and the long-term solution were proposed by the multidisciplinary team of the Irrigation Department consisting the branches of Engineering Geology, Engineering Materials, Design, Dam Safety, Regional Director's Office - Anuradhapura and Divisional Irrigation Engineer's Office – Nachchaduwa. As a pioneer government department, design and construction were solely done by the staff of the Irrigation Department during 2012-2014. It is of utmost importance to mention here that the solution made ensured no damage to archaeological value of the ancient reservoir and at the same time no cultivation season was foregone during the repairs.

Keywords: Ancient Reservoirs

1. Introduction

1.1 Historical Background

Nachchaduwa Reservoir currently operates under the purview of Divisional Irrigation Engineer (Nachchaduwa) and Director of Irrigation (Anuradhapura). This reservoir was built by the King Moggallana II and was restored by the Irrigation Department in 1903 after being abandoned for several centuries. It was again breached due to the massive floods in 1957 and restored in 1958 by the Irrigation Department with Irrigation Department expertise and local funds.

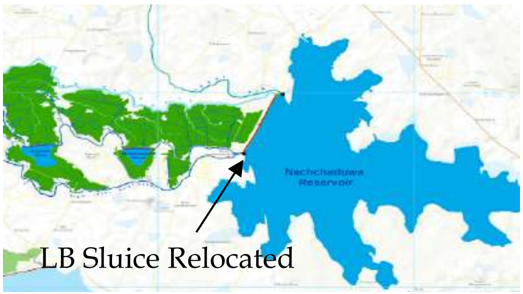


Figure. 1: Scheme Map

1.2 Scheme data

Coordinates	: 166 443 E, 338 374 N
Type of the Reservoir	: Major
Catchment Area	: 59 745 Ha
River Basin	: Malwathu Oya
Irrigable Extent	: 2225 Ha

2.0 Significance of Relocation of the Existing Left Bank Sluice

2.1 Observation of a Leakage

A sudden leakage was observed across Nachchaduwa Tank bund near the LB sluice structure on 2009.12.28. Therefore, technical staff of specialised services (Engineering Geology, Engineering Materials and Design) together with Asset Management branch inspected the site together with RDI(Anuradhapura) and made remedial actions as specified below in order to stop the leakage.

1. Lowering the water level of the reservoir in order to minimise the risk of any kind of failure of the earth bund.
2. Laying of a filter at the downstream end of the earth bund to eliminate the erosion or piping.
3. Treatment by Cement Grouting.

2.2 Investigation and Recommendation

Strengthening of the area associated with the leakage was done by the Engineering Geology branch using cement grouting as a remedial measure. After the discussions among the specialists of Design Branch, Engineering Geology Branch and Dam Safety Branch, investigations were carried out by the Engineering Geology Branch. Bore hole detail of DH 13 and 14 which were taken at the centre of the existing sluice revealed that there was a completely weathered rock just below the sluice level for a depth of 4m (92m MSL - 88m MSL). Acceptable and the treatable rock was encountered at the level of 87.1m MSL. ***Figure. 2*** shows the location of the bore holes and ***Figure. 3*** shows the unfavourable area.

2.3 Constraints on Emergency Relocation

2.3.1. Constraint 1

It was not possible to forego a season as it could severely affect the farmer community and the economy. And the construction had to go on while the water issue was on going.

2.3.2. Constraint 2

There was a restriction on demolishing of the existing sluice and the bund as it has an enormous archaeological value when considering the history of the most ancient hydraulic engineering civilisation in the world.

2.3.3. Constraint 3

Construction of a new sluice by demolishing the existing one was unnecessary as it requires under water construction.

2.4 Alternative Design Solutions

2.4.1 Alternative 1

It was proposed to insert a steel casing to the existing sluice barrel and to protect the same with cathodic action with other sacrificial metal. However, it was not possible to ensure proper compaction around the annular space between existing barrel and new steel casing hence this solution was not implemented.

2.4.2. Alternative 2

It was proposed to form a new stretch of bund at the downstream of the existing sluice and install the sluice at the new bund aligning with the existing sluice where foundation conditions are sound.

2.5 Detailed Geological Investigation along the proposed bund

As it was essential to ensure favourable geological conditions, 7 number of bore holes (DH 1,2,3,4,5,6 and 8) on the proposed alternate bund at the downstream side were done to check the geological conditions of the underlying strata (See ***Figure. 2***). The results revealed of better foundation conditions (See ***Figure. 4***).

2.6 Final Conclusion for a Design after considering Constraints

Considering all the constraints stipulated in 2.4 and geological investigation reports, it was decided to continue with the Alternative 2 stipulated in 2.4.2.

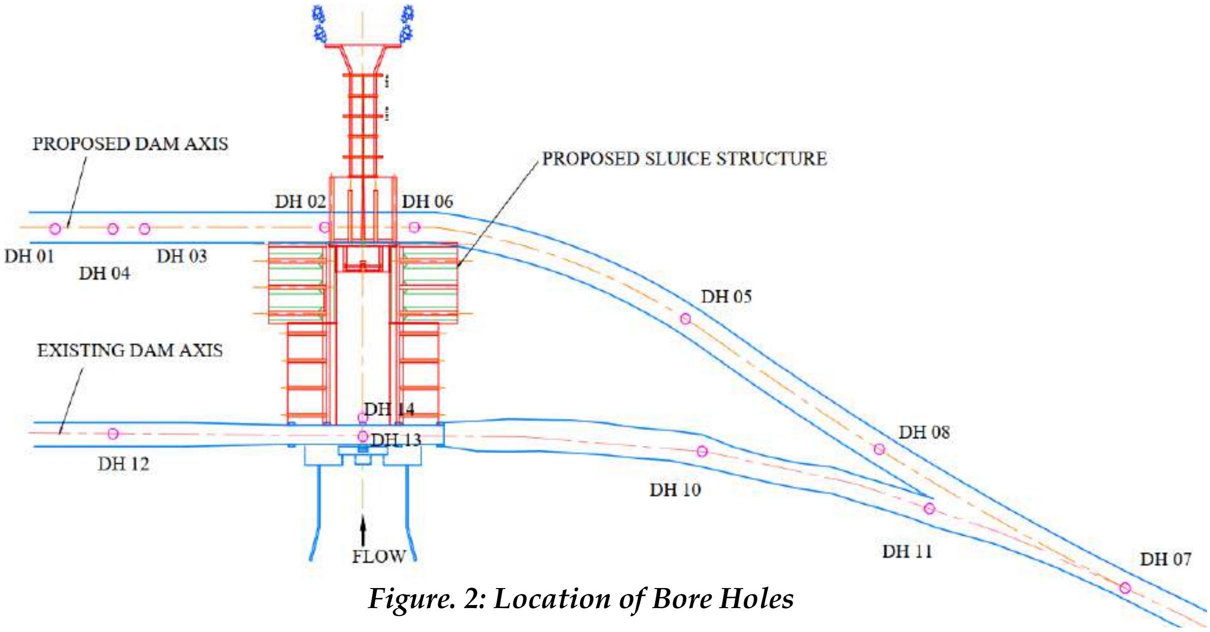


Figure. 2: Location of Bore Holes

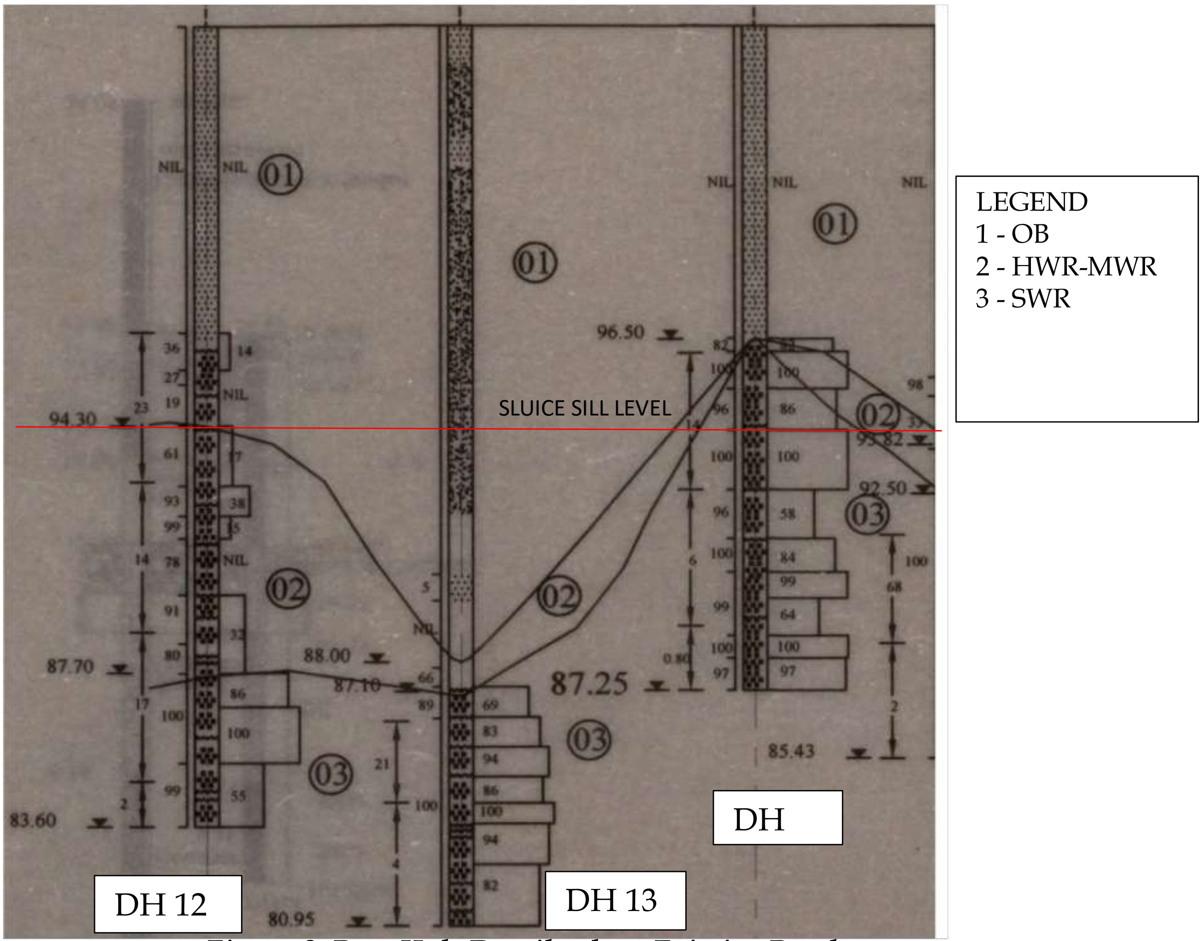


Figure. 3: Bore Hole Details along Existing Bund

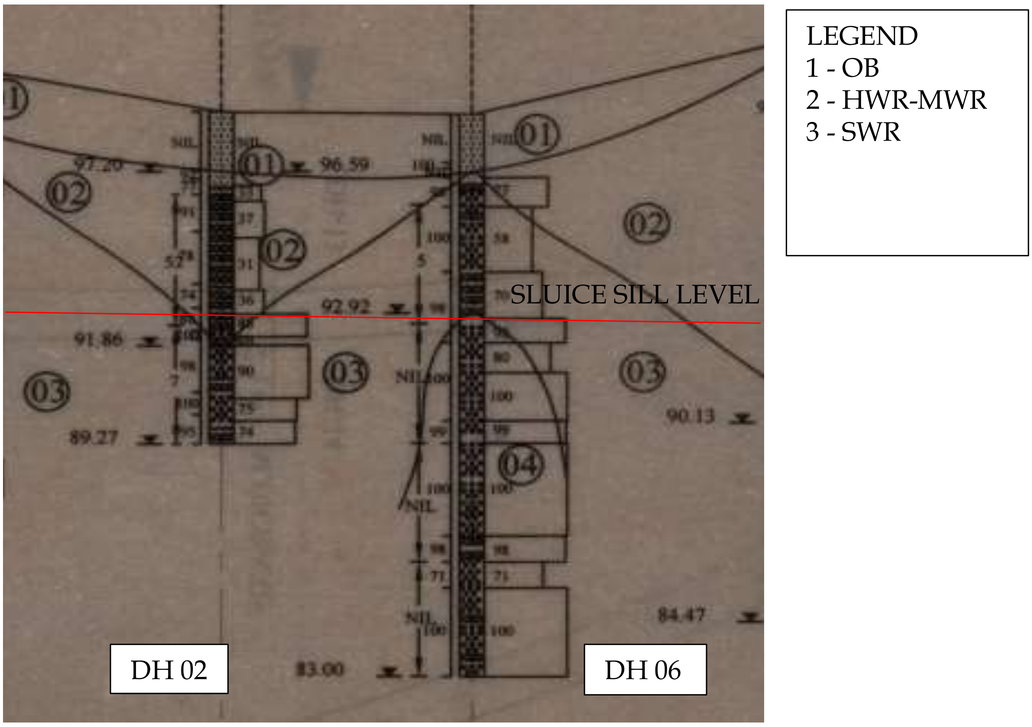


Figure. 4: Bore Hole Details along Proposed Bund

3.0 Conceptual Design

The newly formed earth section designed by the Engineering Materials Division has upstream and downstream slopes as 1: 2.5 and 1: 2.0 respectively together with 4.5m bund top width. The existing BTL is 104.7m MSL and the proposed one is taken as 106.0m MSL during this rehabilitation.

The new structure consists of a pool section created in between new abutments and the existing old bund at upstream side of the new dam. The control gates were installed upstream of the new bund together with a sluice tower and the water is conveyed across the new bund along newly constructed sluice barrels (Refer ***Figure. 9***). Abutments (Unit 1 and 2 of ***Figure. 10 & 11***), Sluice Tower, Retaining Wall (Unit 3 of ***Figure. 10 & 11***) and sluice barrels (Unit 4, 5 and 6 of ***Figure. 10 & 11***) are the design components of the new structure.

3.1 Structural Design

Authors (1 and 2) worked as the Irrigation Engineer and the Deputy Director respectively in the Design Branch, Head Office of the Irrigation Department and author 1 carried out the design under the guidance of author 2 and issued construction drawings during 2010 in order to start the construction work without delay.

3.1.1 The Abutments and the Retaining wall

The abutments of either side of the pooling section consists of retaining walls (Unit 1 and Unit 2 of ***Figure. 11***). Unit 3 is the retaining wall at downstream side of the sluice tower which supports the earth dam and allow openings directing water to sluice barrels. The height of the abutments and retaining walls ranges from 7.0 m to 13.2m and the counterfort type retaining walls were accommodated as it is the economical type of retaining walls when the retaining height exceeds 6m according to the literature.

3.1.1.1 Design of Abutment - Unit 1

Design criteria of Unit 1 and 2 are the same and ***Details of Design of Unit 1*** is presented here.

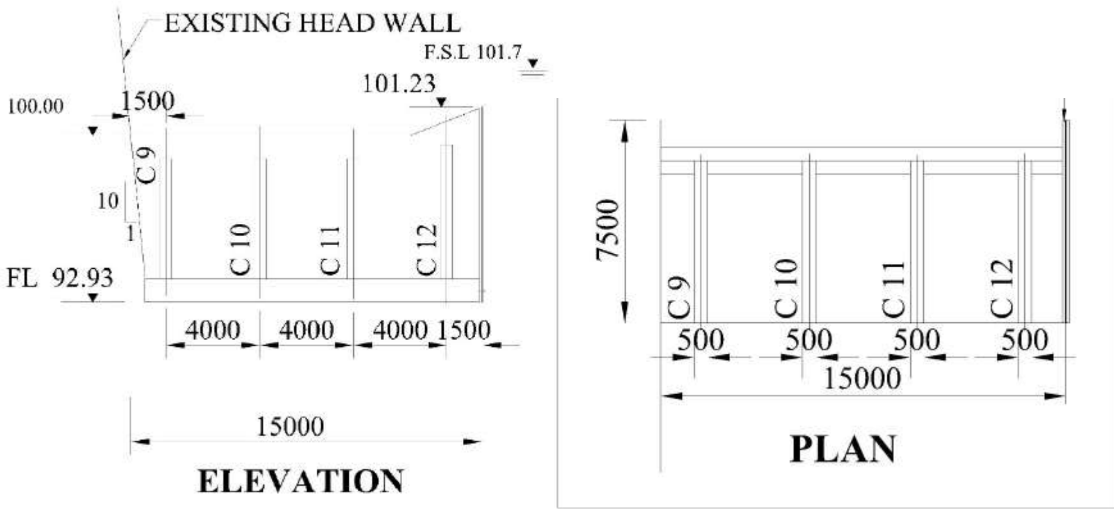
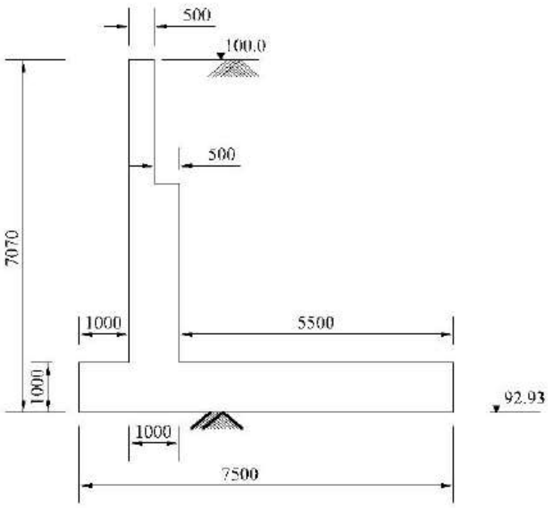


Figure. 5: Details of Abutment – Unit 1

Initially, some dimensions for the retaining wall were assumed as shown below.



3.1.1.2 l ***Figure. 6: Initial Dimensions assumed for the Retaining Wall***

Three load cases were considered as mentioned below.

Case 1: When the Reservoir is at FSL

Case 2: When the Reservoir is at Minimum Level

Case 3: When there is a sudden drawdown of the reservoir

Case 3 was considered for the analysis as it is the most critical case affecting for the stability of the retaining wall. Surcharge of $10kN/m^2$ is considered as an additional load to represent machinery load acting on the earth fill.

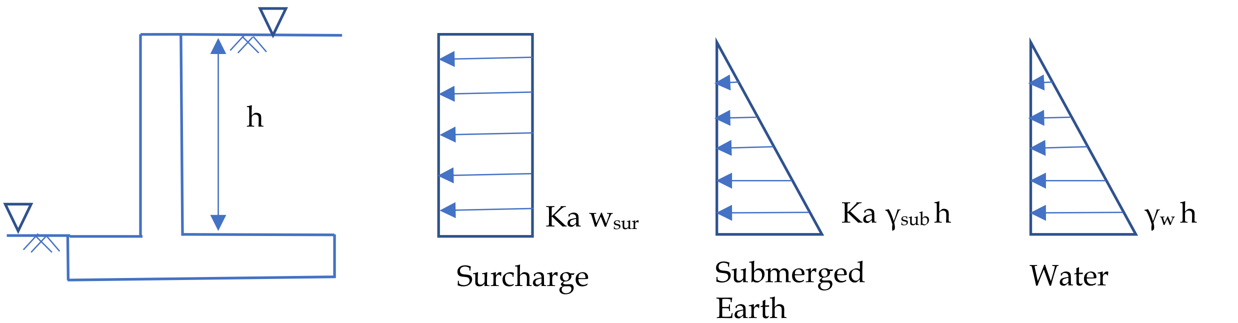


Figure. 7: Horizontal Pressure Loads acting at Case 3 on the Retaining wall

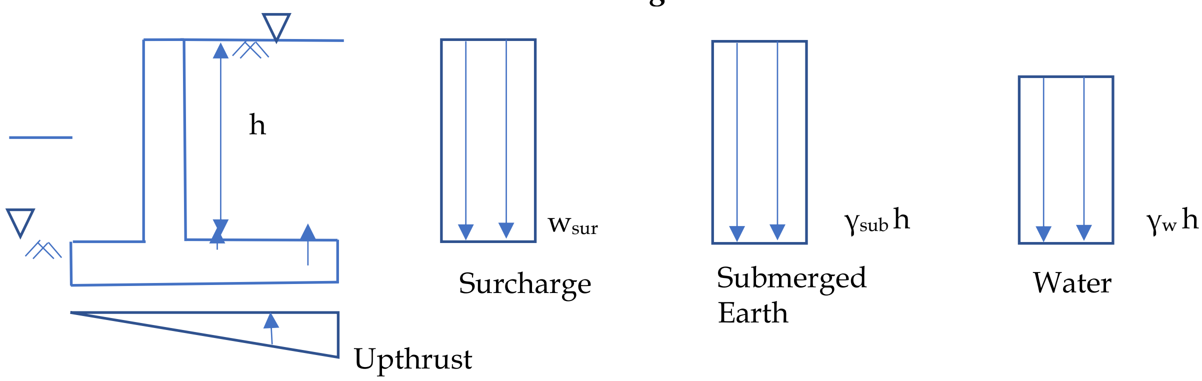


Figure. 8: Vertical Pressure Loads acting at Case 3 on the Retaining Wall

3.1.1.3 Stability Criteria

1.Overturning (Safety factor against overturning should be greater than 1.5)

2. Sliding (Sliding factor should be greater than 1.5)

3. Bearing (Maximum Bearing Pressure should be less than the Bearing Strength of the rock)

4. Tension (No tension is developed at the foundation level – satisfied middle third rule)

The section of the retaining wall was finalised when the above criteria was satisfied.

3.1.1.4 Addition of Counterforts

The counterforts were introduced with 4m intervals as it is not economical to design the abutment as a cantilever retaining wall when the retained height is more than 6m.

3.1.1.5 Design of the Stem

It is clear that the horizontal loads on the stem are linearly increasing along the downward direction, therefore a few horizontal strips with constant load are considered in designing. The stem was designed as a one way spanning continuous slab supported by the counterforts. The design was carried out considering the Bending and Shear criteria.

3.1.1.6 Design of the Heel Slab

The load at the heel level is constant and the heel slab was designed as a one way spanning continuous slabs supported by the counterforts. The design was carried out similar to the stem considering the Bending and Shear criteria.

3.1.1.7 Design of the Counterfort

The counterforts were designed for bending and shear for the loads coming on to them.

3.1.2 Sluice Tower and the Barrels

The foundation of the Sluice tower was designed for the self-weight of the structure, gates and live load acting on it. The barrels of the sluice were designed for the self-weight and the weight of the earth acting on them.

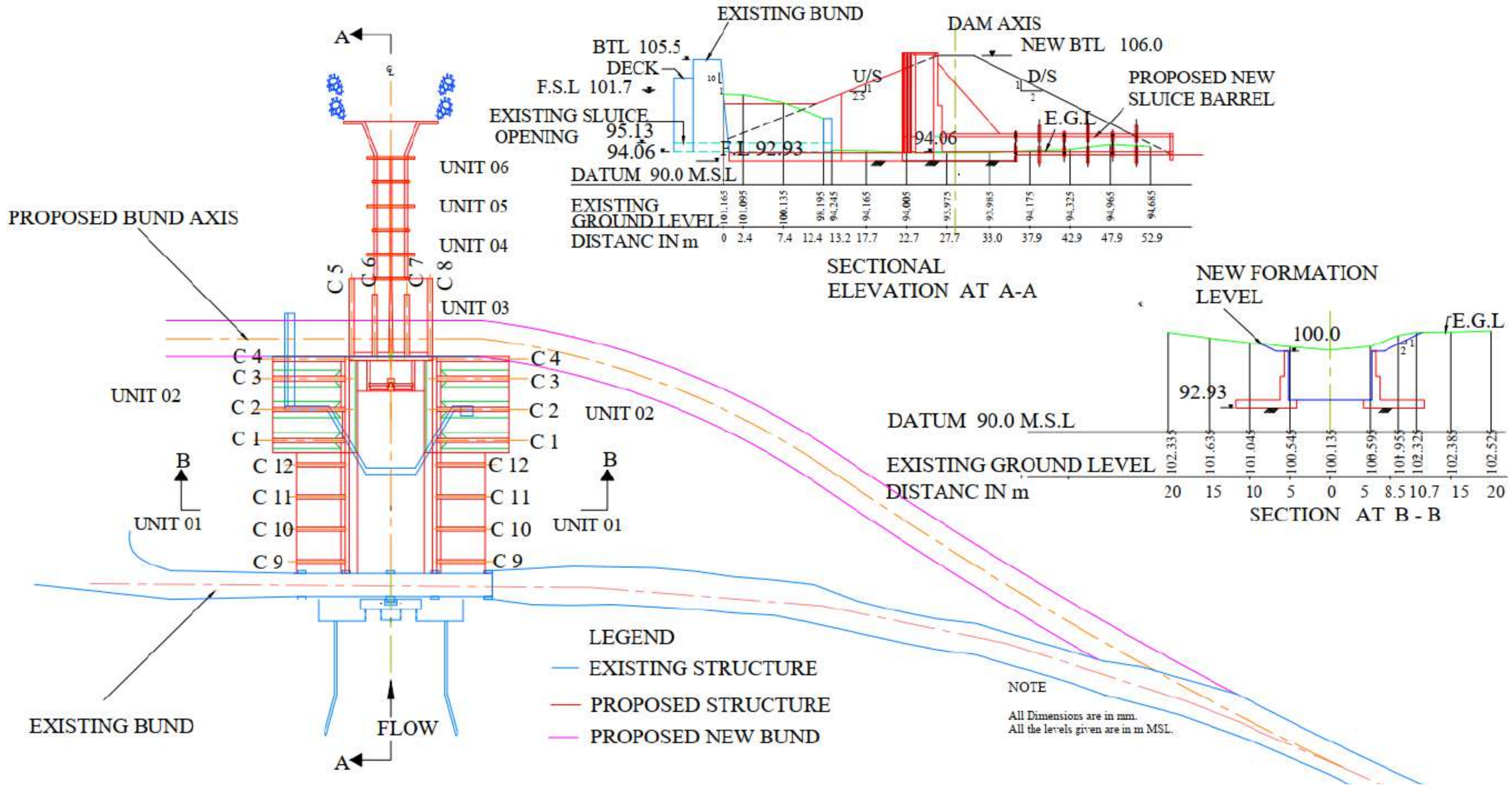
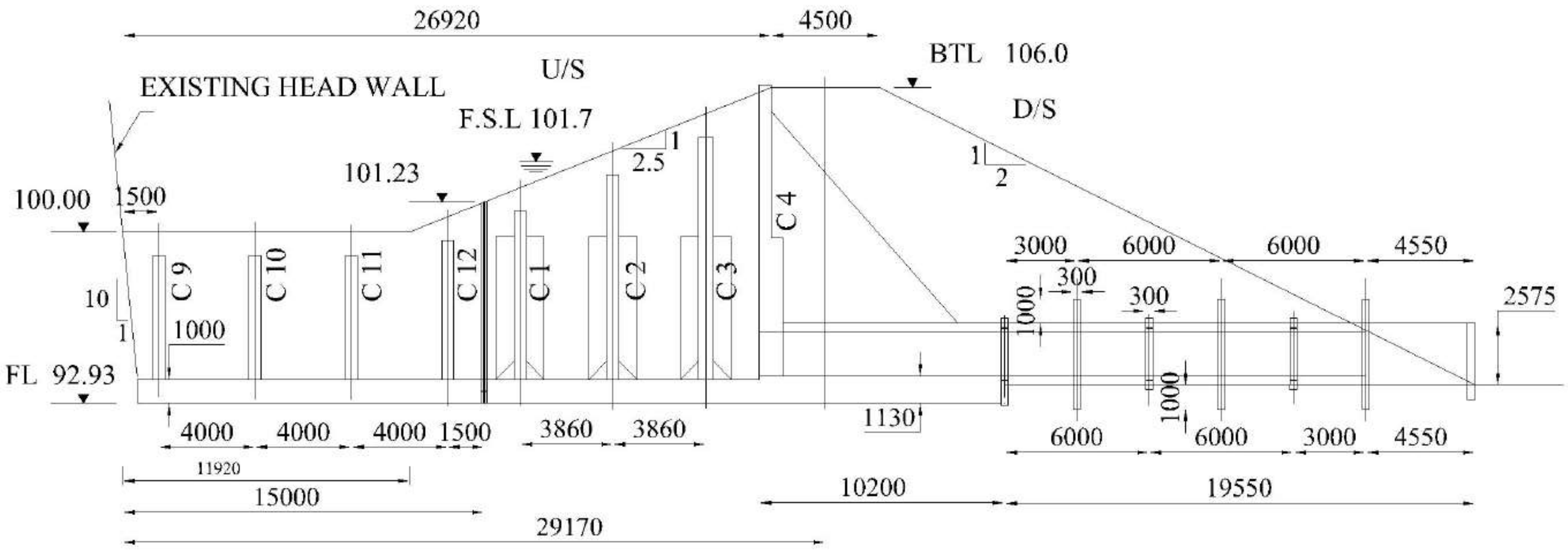


Figure. 9: General Arrangement of Proposed Sluice of Nachchaduwa Tank



SECTIONAL ELEVATION AT C-C

Figure. 10: Sectional Elevation of Proposed Sluice of Nachchaduwa Tank

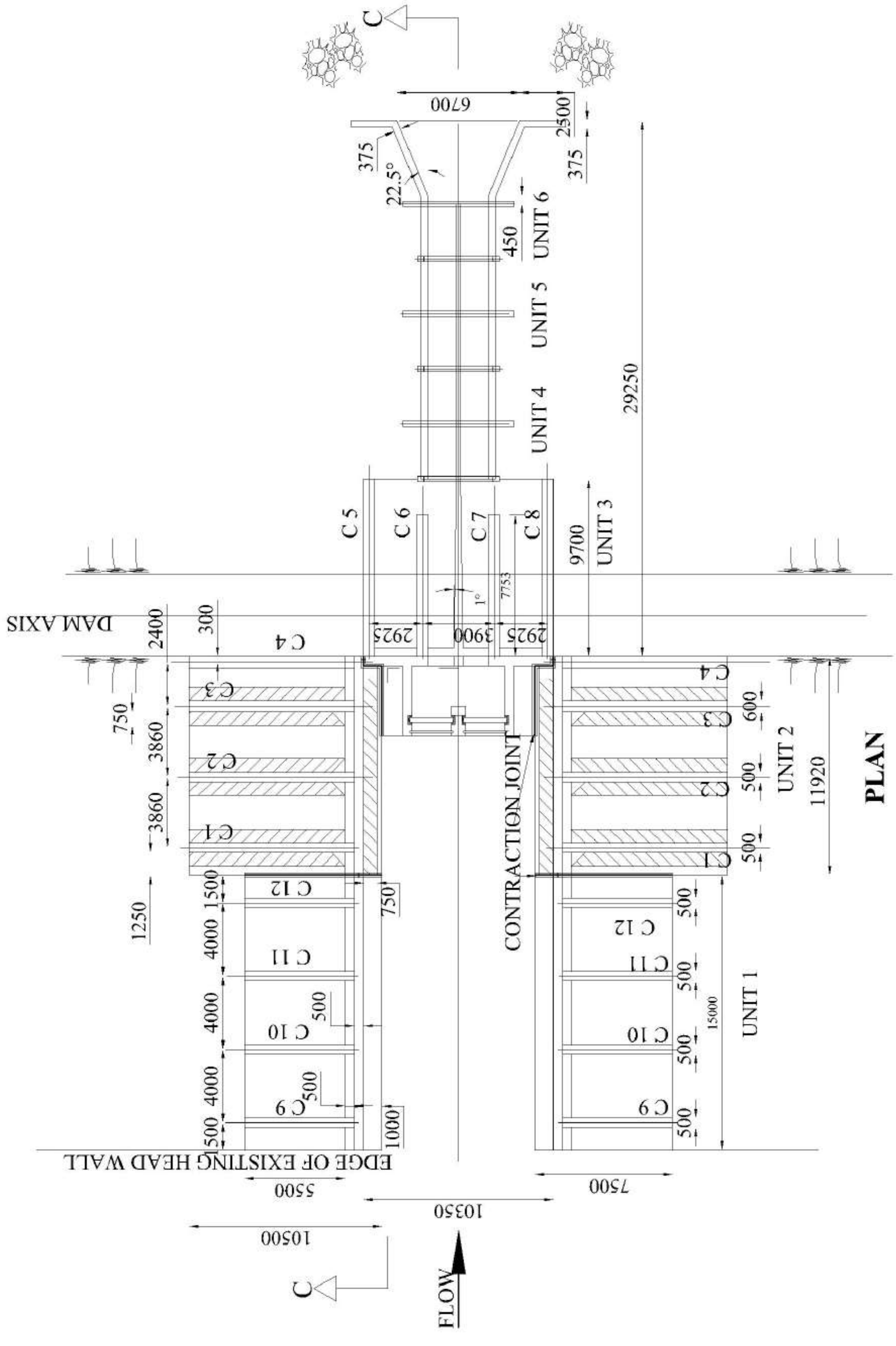


Figure. 11: Sectional Plan of Proposed Sluice of Nachchaduwa Tank

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Construction work was carried out by the Anuradhapura range/Nachchaduwa Resident Engineer during 2012 - 2014 on direct labour method at the cost of Rs. 130 Million .

Figure. 12a: A Site View during Construction

Figure. 12b: Reinforcement Work during Construction

Figure. 12c: Counterforts during Construction

Figure. 12d: A view after the Construction

5.0 Conclusions

With reference to current engineering prac
planning, design and construction of such a structure by the Irrigation Department itself is most creditable and this has been the ancient practice of the Irrigation Department and unless such practices are adopted, Sri Lanka will find it almost impossible to go forward in its economic ladder. This demonstrates the fact that Irrigation Department (even other government institutions) has capable engineering human resources to undertake any water resources or any other engineering work without the service of local or foreign consultants or contractors. You will always find construction cost and time taken for completion of a project is minimal when undertaken by the Irrigation Department.

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References

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[2] Engineering Geology Reports and relevant documents available in the Engineering Geology Branch.


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