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SEDIMENTATION AND SEDIMENT HANDLING IN HIMALAYAN RESERVOIRS

Kulekhani resevoir, dam and sloping intake

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SUMMARY

Dams have been promoted as an important means of meeting needs for water and energy. Due to increasing population in the earth, more dams are needed to be built in the future to meet people’s increased demand for water, food and energy. In contrast to ever increasing demand, global water storage volume is reduced by approximately one percent annually due to reservoir sedimentation.

A more accurate reservoir sedimentation survey is one of the useful means to monitor reservoir sedimentation rate. However, numerous factors affect the accuracy of surveyed data from the reservoir sedimentation survey. Even a small scale error may lead to an erroneous estimate in the total sediment volume, which ultimately may result to an inaccurate computation of reservoir sedimentation rate.

In general, reservoir sedimentation survey in large reservoirs is being used to determine the total capacity of the reservoir, the sediment deposition rate, the capacity-elevation curves and the sediment yield from the watershed. However, in daily peaking reservoirs designed to regulate only for the peak hours of a day, the reservoir surveys is carried out to determine the sediment deposition pattern, the amount of flushed sediment after flushing, the shift in the sediment deposition area and the capacity-elevation curves.

To assess the measurement accuracy, a bathymetric survey was carried out in Øvre Leirfoss tailrace pond and Selbusjøen Lake close to its outlet. Further, similar survey was carried out in Kulekhani and Kaligandaki A reservoirs in Nepal with an aim to determine the effects of survey track lines and survey point density in DGPS bathymetric survey.

The main factors that effect on the accuracy of survey data and volume computations from the bathymetric survey are terrain irregularity, data density and track lines of the survey. These effects can be minimized by increasing survey point data density with the coverage of the entire reservoir area. The sediment yield, the capacity reduction rate and sediment deposition thickness should be assessed with respect to the accuracy of survey technique used to determine the schedule and the frequency of bathymetric survey of the reservoir.

At present, Kaligandaki A and Middle Marsyangdi hydropower plants supply about one third of total hydropower generation in Nepal. Hence, the sustainability of these reservoirs is a key issue. In this respect, bathymetric surveys were conducted in December 2010 in order to prepare base line maps of the reservoir and to monitor the sedimentation process in these reservoirs.

The Kaligandaki A and the Middle Marsyangdi reservoirs have lost 51% and 65% of their total volume, respectively. The total losses within the live storage capacity are in the range of 6.7 percent and 14.1 percent of the total live storage capacity, respectively. Still, both power plants are capable in generating with their full capacity during the peak load demand in Nepal. Hence, reservoir capacity loss within the dead volume in the peaking reservoir does not affect in generation capacity.

Reservoir sedimentation surveys in the Kulekhani reservoir were also carried out using Differential Global Positioning Systems (DGPS) method in 2009 and 2010 by this author with a support received from Norwegian University of Science and Technology (NTNU) Norway and Hydro Lab Nepal. Bathymetric maps were prepared from the survey data. The deviation (variation) in depths between survey data of 2009 and 2004, 2010 and 2004 and 2010 and 2009 are computed using software program “Surfer”. The deviation maps are also plotted.

The reservoir volume based on 2010 survey is 64.89 mill m^3. The Kulekhani reservoir has lost 20.4 mill. m^3 in total and 14 mill. m^3 in it’s live storage capacity during last 27 years of operation. It gives an average annual loss rate of about 1 percent of the original reservoir capacity. The annual average thickness of sediment deposition is 0.16 meter since 1996. Similarly, the annual average sediment deposition volume is 0.56 percent of the total reservoir capacity. The annual average thickness and the sediment deposition volume in the Kulekhani reservoir are less than the average error of the bathymetric survey with the DGPS method. Based on the current sediment yield, capacity reduction rate, sediment deposition thickness and the accuracy of the DGPS bathymetric survey it is concluded that the bathymetric survey in this reservoir should be conducted at an interval of at least five years.

It is noted that the peak energy demand is growing annually by about 10 percent in Nepal, creating acute electricity shortage in the country. Demand is exceeding supply every year. The greatest challenge is to bridge the gap between supply and demand of electricity. At present, Nepal is facing load shedding up to 16 hours a day during the dry season.

The Kulekhani reservoir is the only reservoir project in operation in Nepal, which facilitates seasonal water storage for energy production. The load shedding hours during dry season depends mainly on the availability of water in this reservoir. There is power deficit even during the wet season, resulting in the use of Kulekhani power plants to generate electricity during the wet season. This is due to the fact that the run of river (RoR) plants have generated with lower capacity (up to 50% of the total capacity). The Kulekhani reservoir is therefore not filled to its full capacity during the wet seasons since 2006. If RoR plants were capable in generating their maximum capacity, the Kulekhani power plant would not be needed to operate during the wet season.

In this respect, an excel model is developed for the optimum operation of the Kulekhani reservoir and for some key peaking run of river (PRoR) power plants. The model is based on historic data records. The model focuses on water availability, generation and storage capacities and the power demand. The transmission part of the system is not considered to be a bottleneck in this context.

The model shows that if Nepalese hydropower plants are operated and maintained properly, they can generate their full capacity and the load shedding can be reduced significantly. The Kulekhani reservoir can also be operated at higher reservoir levels. The large amount of load shedding in Nepal is due to the capacity deficiency during the peak hours from October-November to March-April. Development of PRoR instead of RoR helps to reduce the load shedding during this time.

The RESCON (REServoir CONservation) model is a spreadsheet-based program written in Visual Basic programming language and works with macros. This model is used to review the sustainability of the Kulekhani reservoir. Some sensitivity analyses are also carried out.

Based on the RESCON analysis both the flushing and Hydrosuction Sediment Removal System (HSRS) sediment management options seem to be feasible for the Kulekhani reservoir. However, HSRS is found to be best sediment management option in regards with economy.

In RESCON model, the calculated sediment concentration through hydrosuction pipe of HSRS is very low compared to the sediment concentration achieved during the field test in Nepal. Quantity of sediment removed by HSRS method could be higher in comparison with as calculated by RESCON method. Hence, calculation of sediment transport and sediment concentration in RESCON method should be reviewed and updated.

Field tests of the Hydrosuction Sediment Removal System (HSRS) with the Modified Double Layer Sediment Sluicer (MDLSS) were also performed at the settling basin of Sunkoshi small hydropower plant and at the peaking pond of the Sunkoshi hydropower plant in July 2009.

The field tests of HSRS indicates that it is a promising technology and has relatively low-cost and low-power requirement system for the sediment removal from the reservoirs and peaking ponds. The field test study shows that the efficiency of the hydrosuction sediment removal can be increased if it is combined with water jetting to break consolidated sediment deposits. The efficiency and performance of the HSRS can also be improved with the adjustment of the opening between inner and outer pipe according to the consolidation level of sediment deposit.

The study of the sediment management in the Kulekhani reservoir showed that HSRS is best suited for removing sediment deposit from the lower portion of the reservoir. The field test also proved that it is possible to remove even consolidated sediment deposit from the bottom of reservoir.

The HSRS can evacuate sediment deposits from the downstream area of the reservoir (about two km upstream from the dam) and sediment from the upstream parts will be transported from live storage to the downstream. In principle, it is possible to evacuate the total incoming average annual sediment load from the reservoir every year with HSRS, which can help to establish sustainable reservoir with present storage capacity.

Hence, the Hydrosuction Sediment Removal System (HSRS) is economically attractive and viable for sediment handling in the Kulekhani reservoir. However, HSRS is one of the long-term sedimentation removal solutions for the sustainability of the Kulekhani reservoir.

PREFACE AND ACKNOWLEDGEMENTS

This thesis summarises the work had been carried out by the author during the period from January 2008 to April 2012. The research presented in this thesis has been undertaken at the Department of Hydraulic and Environmental Engineering (DHEE), Faculty of Engineering Science and Technology, Norwegian University of Science and Technology (NTNU) under the supervision of Professor Haakon Støle.

I got a quota scholarship from the State Educational Loan Fund of Norway (Lånekassen) for the first three years of my research. I have also received support from DHEE for the first three years. The last one and half years of my scholarship was covered by Trans-Himalayan University Network for Education and Research (THUNDER) project. I have also received financial support from Sediment Systems - Dr. Ing. H. Støle AS. Professor Haakon Støle and Sediment Systems - Dr. Ing. H. Støle AS have provided financial support for the required research equipment, field work and other operational costs for the research work I have carried out in Nepal and in Norway. I have also received support from DHEE and THUNDER to take part in international seminars in Nepal, Norway and South Africa where I presented papers.

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I express sincere gratitude to the NEA for providing opportunity to conduct research work at the Kulekhani, Kaligandaki A, the Middle Marsyangdi and the Sunkoshi hydropower plants and other logistic support. Special thanks to System Operation Department for providing genration data from major hydropower plants of Nepal. I wish to express my sincere thanks to Hydro Lab and Dr. Meg B. Biswakarma for their cooperation and support during this research work. I would like to thank Mr. Yogesh, Mr. Ramesh, Mr. Raju and rest of the staffs at Hydro Lab and Mr. Dhurba Basnet, chief of civil section of Markhu branch of Kulekhani for their great cooperation during reservoir survey and field work.

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Last but not the least, I dedicate this work to my beloved parents, Bishnu Bahadur and Gyani Maya Shrestha. I am always grateful to them and now I am in this position because of their support, encouragement and blessing.

Hari Shankar Shrestha

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ABBREVIATIONS AND SYMBOLS

ψ	Constant value, depends on sediment type
Φ	Dimensionless excess head loss
Ψ	Durand-Condolios group of independent variables
η	Efficiency
ω	Fall velocity
υ	Kinematic viscosity
γ_s	Specific weight of particles
γ	Specific weight of water
ρ	Mass density of clear water
η	Trap efficiency
$\Delta\rho$	Density difference between density current and clear fluid above
η^*	Mechanical efficiency
°C	Degrees centigrade
μ_L	Viscosity of liquid
ρ_d	Density of sediment deposit
ρ_{mix}	Density of mixture
ρ_s	Density of solids (sediments)
ρ_w	Density of water
A	Symbol for area
Acf	Acre feet
A_f	Cross-sectional area of the scoured valley
Ar	Archimedes number,
A_r	Cross-sectional area of the reservoir
B/C	Benefit cost ratio
BM	Bench Mark
BPC	Butwal Power Company
C	Symbol for suspended sediment concentration
C_D	Drag coefficient for d_{50}
C_{DM}	Average drag coefficient
C_h	Chezy roughness coefficient
CIR	Capacity inflow ratio
cm	Centimeter.

C_m	Sediment concentration by mass
cm/s	Centimeter per second.
$C_{monthly}$	Average monthly concentration
C_o	Original capacity of the reservoir
CSR	Capacity Sediment ratio
C_v	Sediment concentration by volume
C_w	Concentration by weight, decimal fraction
D	Diameter of pipe
D_{50}	Symbol for particle size. Size of “average” particle during visual determination of bed material distribution. Size of particle where 50% of sample by weight is finer than D_{50} during particle size distribution analysis.
D_{90}	Symbol for particle size. Size of “bigger” particle during visual determination of bed material distribution. Size of particle where 90% of sample by weight is finer than D_{90} during particle size distribution analysis.
DGPS	Differential Global Positioning System
Dr.Ing.	Doctoral degree in engineering
d_z	Difference in elevation, meter.
EEK	Energy equivalent
f	Darcy-Weisbach friction factor
F_L	Dimensionless limit deposit velocity
FSL	Full supply level
g	Acceleration due to gravity
GPS	Global Positioning System
GW	Giga Watt 1 GW = 10^9 W
GWh	Giga Watt hours
h	Symbol for head, m
ha	Hectare, 1 ha = 10^4 m^2
HD-PE	High-density polyethylene
HMG/N	His Majesty’s Government of Nepal
hrs	Hours
HSRS	Hydrosuction system for the removal of sediment
i	Head loss gradient for water
ICOLD	International Committee of Large Dams
i_m	Head loss gradient for the sediment-water mixture

INPS	Integrated Nepal Power System
IoE	Institute of Engineering
IPP	Independent Power Producer
IRR	Internal rate of return
K	Symbol for conveyance (carrying capacity of the channel)
kg	Kilogram
kg/m^3	Kilograms per cubic metre
KG-A	Kaligandaki-A
KL-I	Kulekhani 1
KL-II	Kulekhani 2
km	Kilometre
km/h	Kilometer per hour
km^2	Square kilometre
km^3	Cubic kilometer
kWh	Kilo Watt hours
kWh/m^3	Kilowatt hour per one cubic meter of water volume (energy equivalent)
L	Symbol for length
LDC	Load Dispatch Center
m	Metre
m/s	Metres per second
m/s^2	Metres per square seconds
m^2	Square metre
m^3	Cubic metre
m^3/s	Cubic metre per second
m^3/year/km^2	Unit of specific sediment yield in terms of volume, cubic meter per year per square kilometer
MAR	Mean Annual Runoff
masl	Metres above mean sea level
M_{dep}	Annual sediment deposit
M_f	Mass of sediment flushed
mill.	Million
mill. m^3	Million cubic meter.
M_{in}	Sediment inflow rate
mm	Millimetre
MW	Mega Watt

n	Manning's number/ Number of years into the future
N/m^2	Newtons per square metre
NA	Not available
NEA	Nepal Electricity Authority
NOK	Norwegian kroner
NPV	Net present value
NRs	Nepalese Rupees
NTNU	Norwegian University of Science and Technology
O&M	Operation and maintenance
omc	Operation and maintenance coefficient
ppm	Unit of sediment concentration, parts per million: I mg sediments in 1000 g of water and sediment mixture.
PRoR	Peaking Run of River
Q	Discharge
Q_{av}	Symbol for average flow
Q_f	Symbol for flushing discharge, m^3/s
Q_s	Sediment Discharge
r	Social discount rate
R	Hydraulic radius
ref.	Reference
RESCON	Symbol for Reservoir Conservation
RoR	Run of River
S	Second
S	Slope (%)
S	Symbol for bed slope
S_f	Symbol for energy slope
S_o	Average bed slope of reservoir
SOD	System Operation Department
SPSS	Slotted Pipe Sediment Sluicer
Sq.miles	Square miles
S_s	Relative density of solid; ρ_s/ρ_w
SSL_r	Suspended sediment load in the river (tonnes)
SSS	Saxophone Sediment Sluicer
t	Life expectancy of individuals
T	Symbol for time (hours)

TE	Trapping efficiency
T_f	Symbol for duration of flushing
TWh/year	Unit of energy, Tettawatt hour per year
USD	United States dollars
V	Velocity
V	Average velocity
V_{in}	Mean annual inflow
V_L	Limit deposit velocity
V_{Lmax}	Maximum limit deposit velocity
v_r	Settling velocity of particles
W	Symbol for channel width
W_{bot}	A representative bottom width of the reservoir
WCD	World Commission of Dams
WECS	Water and Energy Commission Secretariat
W_f	Flushing width
W_{res}	A representative reservoir width
X_i	Fraction of particles
Y	Symbol for stage height
yr	Year

1 INTRODUCTION

1.1 BACKGROUND

Dams have been promoted as an important means of meeting perceived needs for water and energy services and as long-term strategic investments with the ability to deliver multiple benefits. Increasing population and increasing consumption make the demand for water storage inevitably rising despite the increasing use of alternative water sources and more efficient use of water. The advantages of dams and reservoirs are considerable and stored water in reservoirs has improved the quality of life worldwide. Due to increasing population in the world, more dams are needed in the future to meet their demand for water, food and energy. According to Lempérière (2006), the overall benefits of dams during the 21st century will be five times in comparison with what they have been since 1950. White (2010) reported that a modest annual growth rate of 1.3% in reservoir storage in 2020 would be required to meet the water demand.

The global water storage is reduced annually because of sedimentation. The estimates as presented by Mahmood (1987), Atkinson (1996) and White (2010) show a wide range of reservoir storage losses. The annual loss of world storage estimated by White (2010), 0.5% per annum, is half that estimated by Mahmood (1987) and Basson (2010), who quote a figure of 1% and 0.96%, respectively. The same percentage of storage loss was quoted by Atkinson (1996) and by Morris & Fan (1998). The global water storage and the reservoir sedimentation in global prospective are presented in Appendix A.

The future trend in reservoir storage as reported by White (2010) is shown in Figure 1-1. The upper curve is the projected demand for reservoir storage. The central curve shows how reservoir storage develops if the rate of dam construction in 2000 to 2010 continues to 2030. The lower curve shows how reservoir storage will diminish due to sedimentation if no new reservoirs were to be constructed (White, 2010). Worst-case scenario is deficit of about 3,000 km^3 in 2030; i.e. about 40% of current capacity.

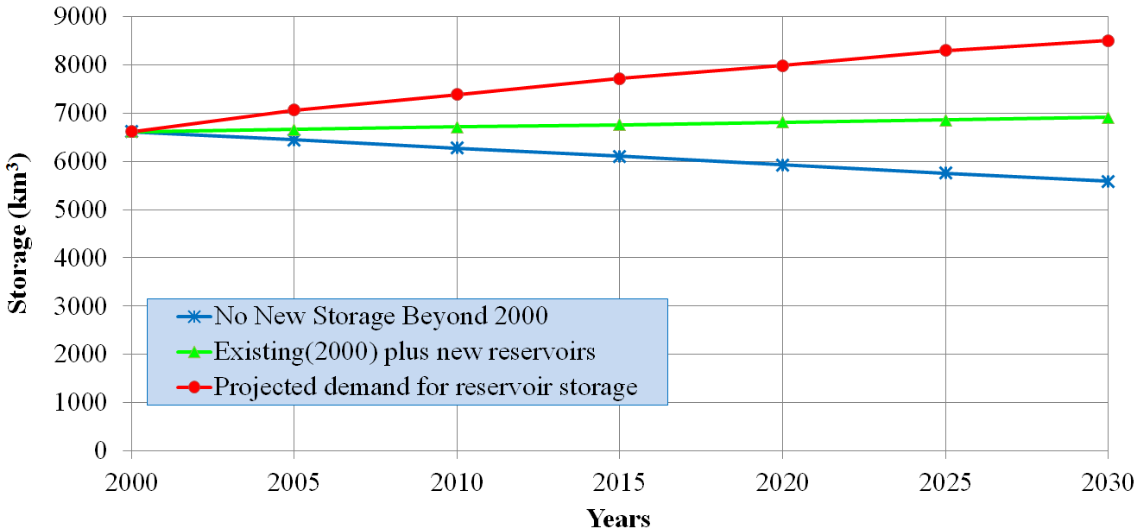


Figure 1-1: Future trends in reservoir storage (White, 2010)

Palmieri *et al.* (2003) report that the annual replacement cost due to the reservoir sedimentation is about 13 billion US dollars. Similarly Basson (2010) reports that the replacement cost is about USD 15 billion (excluding downstream impacts) and with downstream impacts considered the annual cost of about USD 17 billion.

Estimation of sediment inflow and deposition in a reservoir is not easy task as there are large variations in sediment production rates from place to place and from time to time. Data on long-term sediment yield are therefore very important for water resource development in any Himalayan river. Plans for new reservoirs must be based on sediment data and on general knowledge on the physiographical characteristics of the river basin. Therefore, it is important to monitor the sedimentation process of the Kulekhani reservoir and other reservoirs in order to learn more about sediment yield in Nepal.

1.2 OBJECTIVE OF THE STUDY

Reservoirs in Nepal are losing their capacity faster than estimated. It is important to monitor the sedimentation process of the reservoirs which can be used as base for planning and design of new reservoirs and mitigate the existing problems of sediment deposit in the reservoirs.

The Kulekahi reservoir has lost about 30% of its original capacity in 28 years of operation and it is losing its capacity at the average rate of 1% annually. If this process of sediment deposition continues, a stage may be reached when the whole reservoir is filled by sediment and converted to a simple Run of River (RoR) plant.

On the other hand Nepal is facing load shedding of 16 hours a day in the dry season. Kulekhani hydroelectric power plant is so far the only reservoir type hydropower plant in Nepal. Therefore, the water volume in Kulekhani reservoir plays a significant role in Nepalese energy sector. Therefore, its preservation is a vital issue.

The sustainability of the Kulekhani reservoir and reservoirs of other power plants in Nepal depends on sediment management. Before adopting suitable sediment management techniques for the evacuation of deposited sediment from the reservoir, it is necessary to understand sedimentation processes in the reservoir.

On this background, the objectives of this research are:

- Assess the accuracy of bathymetric surveys with Differential Global Positioning System (DGPS).
- Develop base line maps of the Kaligandaki and Middle Marsyangdi reservoirs, using Differential Global Positioning System (DGPS) for the sedimentation monitoring in these reservoirs.
- Continue monitoring of reservoir sedimentation in the Kulekhani reservoir.
- Develop an alternative operation regime for the Kulekkhani reservoir, which will optimize the use of limited energy resources in the Kulekhani reservoir within the context of Nepalese power grid and assess the availability of water to remove deposits from the reservoir.
- Field test of Hydrosuction Sediment Removal System (HSRS) to remove sediment from the peaking pond of Sunkoshi hydropower project and the settling basin of Sunkoshi small hydropower project and evaluate its applicability for Kulekhani reservoir.

- Assess the feasibility/applicability of HSRS to remove sediment deposits from Kulekhani reservoir based on the field test.

1.3 ORGANISATION OF THE THESIS

This study is based on literature studies, analysis of generated data and reservoir operation data, development of alternative operation regime for the major hydropower plants of Nepal and field studies conducted by the author from 2009 to 2011. Each chapter in the thesis is self-contained.

Chapter 1 gives a brief introduction to the problem of reservoir sedimentation. The objective and organisation of the thesis are set out in this chapter.

Chapter 2 presents the effects of survey point density and survey track lines on the accuracy of DGPS bathymetric survey based on the surveys carried out in some reservoirs in Nepal and Norway.

Chapter 3 provides information of the baseline survey carried out in the Kaligandaki and Middle Marsyangdi reservoirs using Differential Global Positioning System (DGPS).

Chapter 4 highlights the sedimentation monitoring in Kulekhani reservoir. It includes bathymetric surveys carried out in the past and by the author in 2009 and 2010. It also deals with the capacity loss based on the bathymetric surveys.

Chapter 5 includes the development of an alternative operation regime for the Kulekkhani reservoir, which will optimize the use of limited energy resources in the Kulekhani reservoir within the context of Nepalese power grid.

Chapter 6 reviews the sustainability of the Kulekhani reservoir using RESCON method including sensitivity analyses.

Chapter 7 contains the experimental installation and results of the field experiment of the modified hydrosuction system carried out in the peaking pond of Sunkoshi hydropower project and the settling basin of Sunkoshi small hydropower project.

Chapter 8 assesses the sustainability of the Kulekhani reservoir and some economic aspects. Economic value of storage capacity and cost of sediment removal are estimated in this chapter.

Chapter 9 summarises the conclusions and recommendations for further studies.

Brief description of reservoir sedimentation and sediment handling, energy situation in Nepal and relevant data are presented in appendices.

2 ACCURACY OF DGPS BATHYMETRIC SURVEYS

2.1 INTRODUCTION

Sediment deposition rates and distribution pattern in reservoirs can be obtained from repeated bathymetric surveys of the reservoirs. Accurate reservoir sedimentation survey is one of the appropriate means for monitoring reservoir sedimentation. In sedimentation surveys, the objective is to obtain accurate volume of the difference between successive surveys. This makes computation of sedimentation rate extremely sensitive to even small errors in the volume estimates, especially when volume changes are relatively small because of a short inter survey period or low sedimentation rate, or when reservoirs are new and have not yet lost significant percentage of their total volume. Errors of only few percent in the total volume estimate can produce errors of several tens of percent in the computed sedimentation rate (Morris and Fan, 1998).

The bathymetric surveys can be done as range surveys, contour surveys and airborne laser scanning photogrammetry survey. Contour surveys use more complete topographic or bathymetric information to prepare a contour map of the reservoir. They are the most accurate technique for determining volume and provide the most complete information on sediment distribution (Morris and Fan, 1998). The Differential Global Positioning Systems (DGPS) is one way of the contour surveys.

However, numerous factors affect the accuracy of the surveyed data in the Differential Global Positioning Systems (DGPS) survey method. The resultant accuracy of a single depth point is represented by a 3-D error ellipsoid as shown in Figure 2-1. The dimensions of this ellipsoid are indicated by the standard errors in all three dimensions S_X, S_Y, and S_Z.

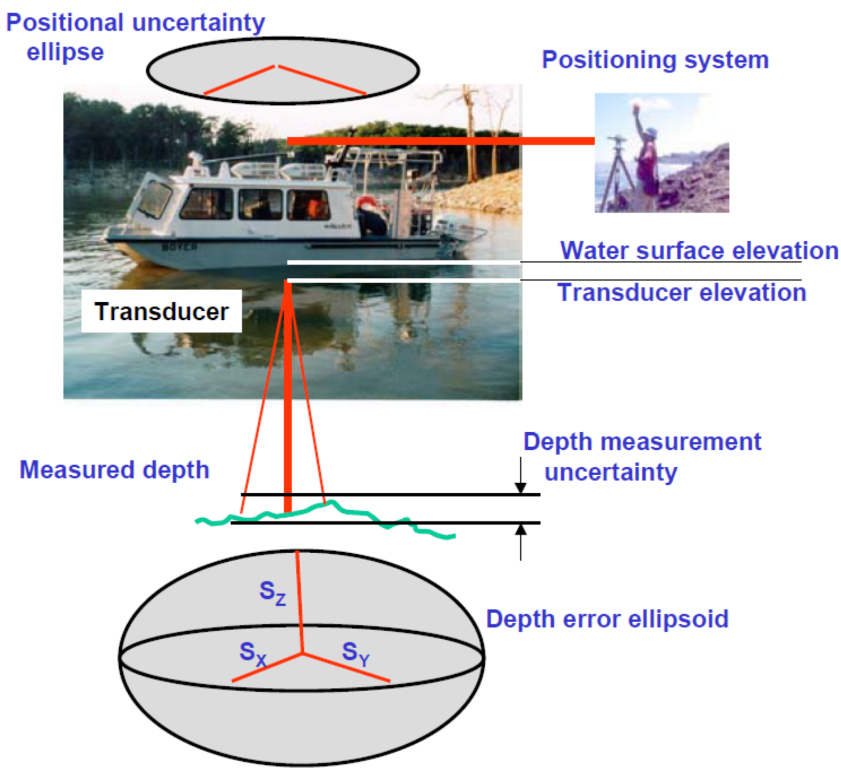


Figure 2-1: Three-dimensional uncertainty of a measured depth (USACE, 2004)

Some of the factors, which affect the accuracy of the bathymetric survey, are:

- Accuracy of instruments
- Representation of the topography
- Density of data points during surveying
- Survey track line orientations
- Fluctuation of reservoir water level
- Stability of the boat
- Alignment of the echo sounder
- Speed of the boat during survey
- Analysis techniques

The accuracy of a bathymetric survey is difficult to monitor relative to conventional land-based surveys due to the lack of available control checks. USACE (2004) reported that bathymetric surveys could exhibit different results when measurements are repeated at different times. This variation becomes obvious when comparing cross sections (and resultant end area volumes) measured by different survey crews and/or systems, or even the same crew and system in many cases. It is extremely difficult to independently verify the accuracy of a dynamic bathymetric measurement (USACE, 2004).

The effects of terrain irregularities on reservoir volume computations depend on the density of data coverage. Even if survey data points were error-free, large volume errors due to terrain irregularity would still exist due to lack of measurements between the two consecutive survey sections. In general, when survey point spacing is closer, the uncertainty is lower. In addition, for surfaces where terrain irregularities are small, estimated volume uncertainties will be lower than areas with the same line spacing and greater bathymetric variations. The only way to minimize the error is to decrease the survey line spacing.

Department of hydraulics and environmental engineering, NTNU bought two sets of equipment that will be used for bathymetric survey. Verification of the measurement accuracy from the equipment is essential before it will be used for any measurements. To assess the measurement accuracy of the new equipment, the bathymetric surveys were carried out in Øvre Leirfoss tailrace pond and Selbusjøen Lake (close to the outlet of the Lake).

Further, the surveys were carried out in the Kulekhani and Kaligandaki A reservoirs to determine the effects of survey track lines and survey point density in DGPS bathymetric surveys.

2.2 METHODOLOGY

The adopted methodology in this study includes field survey, use of different equipment, repeated surveys with different survey point density and track lines, processing and analysing the collected data, preparation of bathymetric map, computation of volumes and comparing the differences in depths and volumes between the data sets of surveys. Brief description of the methods adopted for assessment of the accuracy of DGPS bathymetric surveys are given below.

2.2.1 SURVEY METHODOLOGY

The Differential Global Positioning Systems (DGPS) is used for the bathymetric surveys. The principle behind DGPS is to use a reference location of the base receiver to correct for the position error of the unknown rover position. The equipment consist of a field computer with GEODOS software, the GARMIN GPS35 with GPS L1 antenna, the Topcon Legacy H receiver with RTCM output, SATEL 441 MHz radio transmitter, the rover (Garmin Etrex) and an echo-sounder, which together define the absolute x, y, z coordinates of the reservoir bottom during traverses. As shown in Figure 2-2 the boat is equipped with GPS, echo sounder and field computer while its reference station is located on a known position (known easting, northing and elevation) and a radio-link provides communication between the instruments at the base station and in the boat.

Position Measurement

DGPS surveying requires the use of at least two GPS receiver systems functioning simultaneously, as shown in Figure 2-2. One receiver is designated as the base and the second is the rover. The base system collects raw data from all available satellites. This raw data is sent to a radio transmitter. The transmitter broadcasts the raw data to the end user. Some of the major errors and uncertainties are effectively minimized when simultaneous observations are made at the receiver stations.

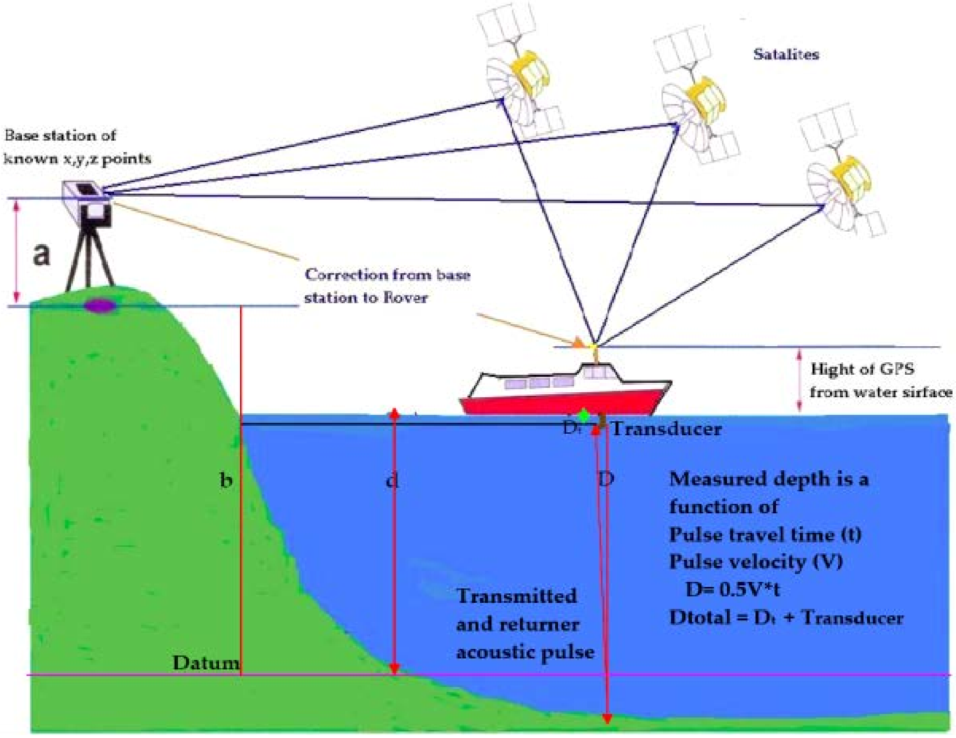


Figure 2-2: Schematic presentations of the working principles of DGPS (Belete, 2007)

As shown in Figure 2-2, this system allows the receiver station set over a known point to compute corrections (difference between observed and known position) for four or more satellites and transmit the correction to the rover so that the observed readings will be corrected accordingly.

Once configured and made operational, the radio in the rover system receives the transmission from the base station and passes the received raw data to the rover receiver via the serial port. Simultaneously, the rover GPS receiver collects its own raw data at its current position. The raw data from the base GPS receiver and the raw data from the rover GPS receiver are processed by the rover receiver. Finally, the rover receiver computes the rover position using the known base position and the computed vector (horizontal angle, vertical angle and distance between the base and rover receiver), (Belete, 2007).

Depth Measurement

An echo sounder is used for depth measurements. An echo sounder is a portable recorder designed to measure water depth by projecting a high-energy acoustic signal. The signals are sent downward, from just below the water surface and receiving them when the signals reflect back from the reservoir bottom. As shown in Figure 2-2, measuring the depth of water depends on the speed of sound through water. For known water temperature and salinity conditions, the speed of sound through water is known so the time interval between the projected and receiving signals are related directly to the water depth. The present bathymetric survey system uses sonic soundings to record continuous profiles of the bottom of the reservoir.

2.2.2 USE OF DIFFERENT EQUIPMENT

Øvre Leirfoss tailrace pond and Selbusjøen Lake (close to the outlet) located nearby Trondheim were surveyed using the old (previously verified equipment) and a new set of equipment. The old set of equipment was already used in different reservoir surveys in Norway, Nepal, Tanzania and Ethiopia. The verification of the equipment was done by Sangroula (2005) and Belete (2007) in Baklidammen (reservoir) and Lake Jonsvatnet in Trondheim, Norway in 2004 and 2005, respectively. According to Belete (2007) the position accuracy and depth accuracy of the equipment was in the range of 0.2 m to 1.0 m and 2 mm to 30 mm, respectively.

The survey data from the each measurement is used to prepare the contour maps and to calculate reservoir volume with help of the software “Surfer 8”. Then accuracy of the entire survey has been assessed by comparison of the contour maps, calculated volumes and deviations in depths.

2.2.3 SURVEY DENSITY AND TRACK-LINE

To determine the effect of the survey track line and survey point density, the surveys in Kulekhani and Kaligandaki A reservoirs were carried out with different data density, with different distance between two consecutive survey points and track lines, with and without shoreline survey and including and excluding longitudinal survey along the middle part of the reservoir. The accuracy of the survey is checked by comparing the contour maps, calculated volumes and deviation in depths of the repeated surveys. Further difference in volume is calculated for different data point density for the same survey.

2.2.4 DATA PROCESSING AND ANALYSING

The software “Surfer 8” is used for data processing, analysis, preparing bathymetric maps and volume calculation in this study. Surfer is one of the commonly used programs for map generation and volume computation. Surfer is a grid-based graphic program which can interpolate irregularly spaced x, y and z (easting, northing and elevation) data into a regularly spaced grid. The bathymetric surface is interpolated with Kriging and as a

Triangulated Irregular Network (TIN). Kriging is a geostatistical method for interpolating a surface from point data. Elevations are interpolated from the original data points to a regularly spaced grid. The interpolation is based on a global weighting scheme derived from the data variance. The grid is then used to produce maps and volume calculation (Golden Software, 2002).

Processing of the survey data begins with importing data into a Microsoft Excel file from the field computer. Each data point has a row comprising easting, northing, depth and time. The easting and northing define the position of the point and the depth defines the depth of the point from the transducer face. Water levels were also recorded during the survey.

The depth data are first converted to bottom elevations. In order to convert the raw depths recorded by the sounder into bottom elevations, the depth at each reading are subtracted from the corresponding water surface elevation during the same time of the bathymetric survey. The value for bottom elevation of the reservoir is calculated as, Bottom elevation = Water surface elevation – (Depth reading + Transducer Face Depth), where the Transducer Face Depth represents the depth of the transducer face below water level in meters (this value is 0.35 m).

During the bathymetric survey in the reservoir, some erroneous data were collected due to limitation of the echo-sounder, some obstacles between the reservoir bed and echo-sounder, and problem in battery. When the water depth from the echo-sounder was less than 0.5 m, the recorded depth was always more than 50 m. Similarly, when the battery was weak and the echo-sounder met any object (weed, floating debris, wooden log etc.) the recorded depth was always differed from the actual depth. Therefore, the processing of bathymetric survey data started with the removal of erroneous data, and through the selection of valid data, a ‘cleaned’ data set was prepared for further processing. The software “Surfer 8” is used to remove the erroneous data. The procedure is explained below.

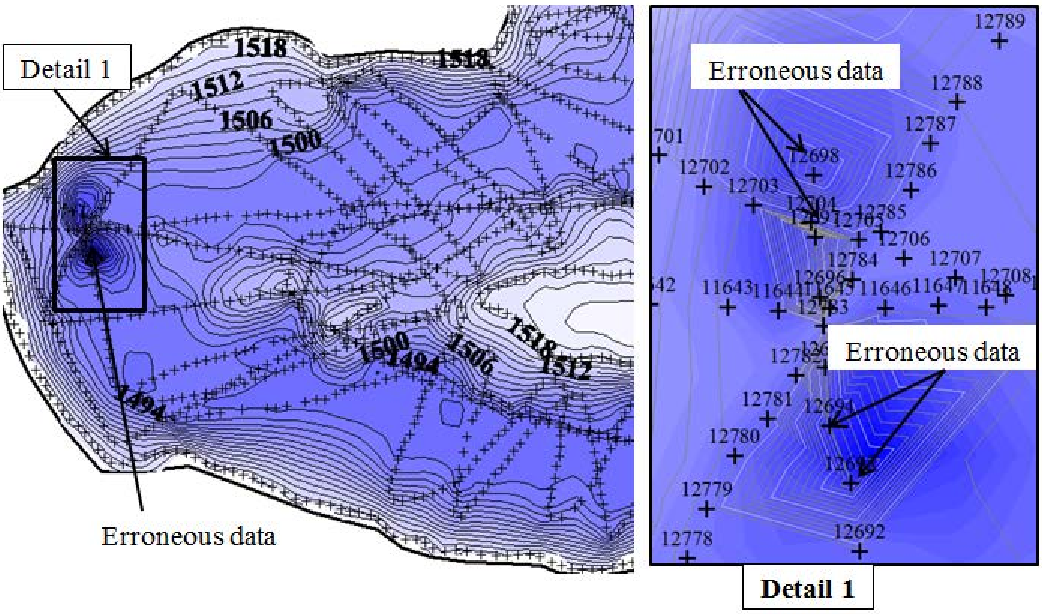


Figure 2-3: Contour map from raw data

The initially processed data file from the Microsoft Excel were transferred and placed into a Surfer worksheet to prepare a grid data file. Then first contour map was generated from the raw data (Figure 2-3). With the help of the contour map, unprocessed erroneous data was identified and removed from the data set. Then the processed data was used to prepare updated contour map and checked if erroneous data still existed or not. This process was repeated until the ‘cleaned’ data set was prepared for further processing.

Finally, the processed clean data file was transferred and placed into the Surfer worksheet to prepare the grid data file. From the grid data file, maps were generated (Figure 2-4), the volume and difference in depths between the different surveying of the reservoirs were also computed by Surfer programme.

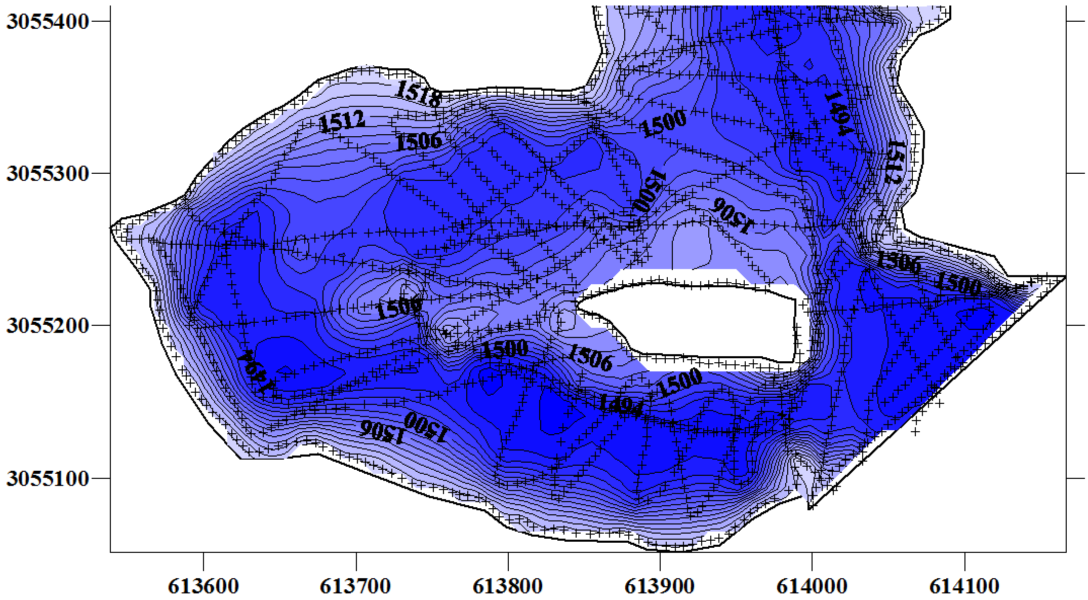


Figure 2-4: Contour map from processed data

2.3 FIELD SURVEY

The surveys were carried out in the following locations:

- Øvre Leirfoss Tailrace Pond
- Outlet area of Selbusjøen Lake
- Middle part of the Kulekhani Reservoir
- Kaligandaki A Reservoir

Table 2-1: Area, maximum depth and base station details of the survey areas

Location	Survey Area, (m^2)	Maximum depth, (m)	Base station details		
			Easting	Northing	Elevation (masl)
Øvre Leirfoss	8,500	9.40	570,486.84	7,027,910.96	73.99
Selbusjøen	184,700	51.54	575,675.92	7,014,394.13	163.04
Kulekhani	253,500	35.35	613,765.25	3,056,017.90	1,529.55
Kaligandaki A	135,300	17.98	459,529.13	3,095,685.20	584.39

Area, maximum depth and base station details of the survey area are tabulated in Table 2-1. The survey area varied from 8,500 m^2 to 253,500 m^2 and maximum depth varied from 9.4 m to 51.54 m. Similarly, elevation of the reservoirs varied from 39.2 masl to 1,521.2 masl.

Figure 2-5: Base station Transducer and GPS and fitted on boat and GARMIN Fish Finder at Øvere Leirfoss

The equipment consisted of a field computer with GEODOS software, a Differential Global Positioning System (DGPS) and an echo-sounder. The GPS and antenna were mounted on the tripod at the base stations. The coordinates and elevation of the base stations are given in Table 2-1. Depth measurements were taken from the boat. The boat was equipped with the echo-sounder and transducer, the GPS system mounted on board and a PSION WB field computer with GEODOS software for data recording (Figure 2-5). The data were stored electronically in the field computer. The echo-sounding instrument consisted of GARMIN Fish Finder and transmitting and receiving transducer mounted on the side of the survey boat with power supply from a battery in the boat. The transducer was fitted at the bottom of the boat, 35 cm below the water surface whereas the rover was located about 2 m above the water surface.

2.3.1 FIELD SURVEY IN ØVRE LEIRFOSS TAILRACE POND

Field test of the equipment was carried out in the tailrace pond of Øvre Leirfoss in Trondheim on 4th and 5th November 2008. Dr. Kiflom Belete and the author were involved in this field survey. All points were recorded at spacing of 5 m. The survey track lines and the surveyed area are shown in Figure 2-6.

The water levels during the survey, number of survey points, survey area and survey point density with old set, Set 1 and Set 2 are tabulated in Table 2-2. Density of the point is calculated per 10,000 m^2 (100 m x 100 m).

Table 2-2: Field survey details in Øvre Leirfoss

S. No.	**Description of surveys**	**Water level, (masl)**	**Survey points**		**Area, (m^2)**	**Survey points per 10,000 m^2**
			Raw	**Processed**		
1	Old set	39.21	188	182	8,540	213
2	Set 1	39.20	110	104	8,261	126
3	Set 2	39.24	140	135	8,183	165

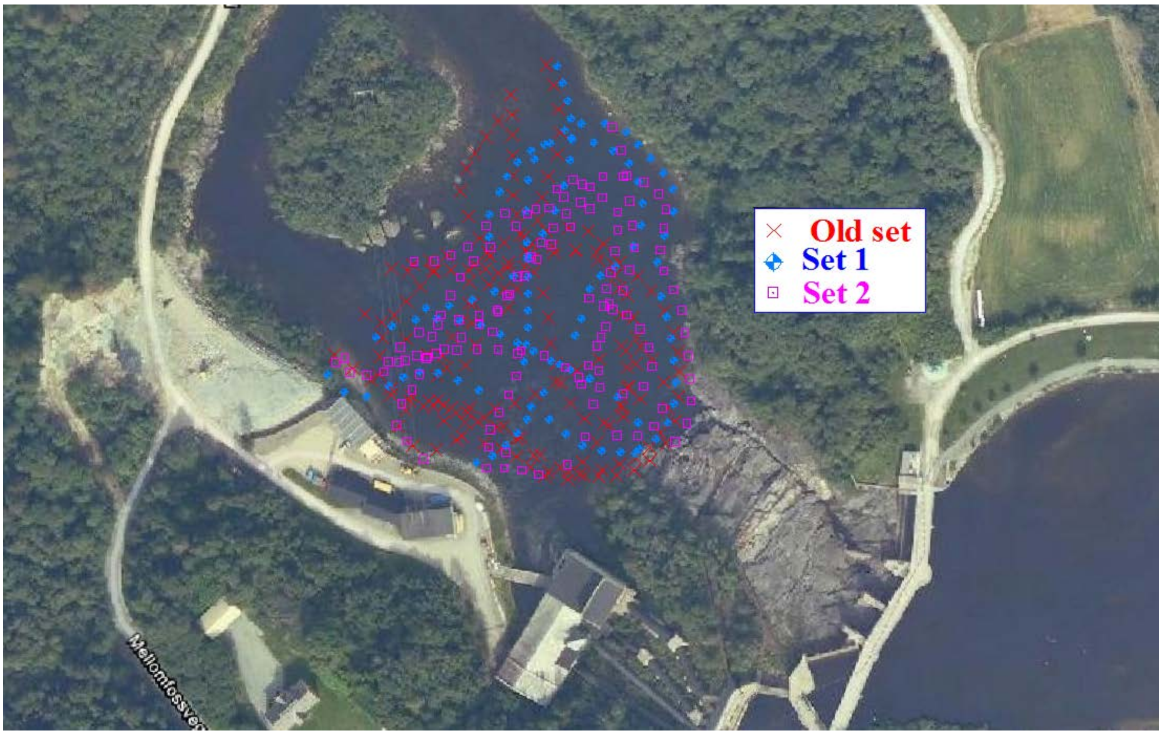


Figure 2-6: Survey track lines and the surveyed area in Øvre Leirfoss (picture from http://kart.gulesider.no/)

2.3.2 FIELD SURVEY IN SELBUSJØEN LAKE

The survey area is located in the downstream part of Selbusjøen Lake near Brøttemsmoen area at about 20 km south of Trondheim. Dr. Kiflom Belete had carried out the field survey for entire area of the Selbusjøen Lake in July 2009 with the old set but only 480 points were extracted from the survey data points of the survey area. Dr. Kiflom Belete and the author carried out the bathymetric survey with the new sets on 13th August 2009. All points were recorded at distance of 5 m for the sets 1 and 2, but 10 m for the old equipment. The survey for the old equipment was carried out independently but the survey with the second set approximately followed the track lines of the survey with the first set equipment. The survey track lines and the surveyed area in the Selbusjøen are shown in Figure 2-7.

The water levels during the survey, number of survey points, survey area and survey point density with old set, Set 1 and Set 2 are tabulated in Table 2-3.

Table 2-3: Field survey details in the Selbusjøen Lake

S. No.	Description of surveys	Water level, (masl)	Survey points		Area, (m^2)	Survey points per 10,000 m^2
			Raw	Processed		
1	Old set	159.99	485	480	184,750	26
2	Set 1	159.99	2,134	2,130	181,091	118
3	Set 2	159.99	1,866	1,848	184,709	100

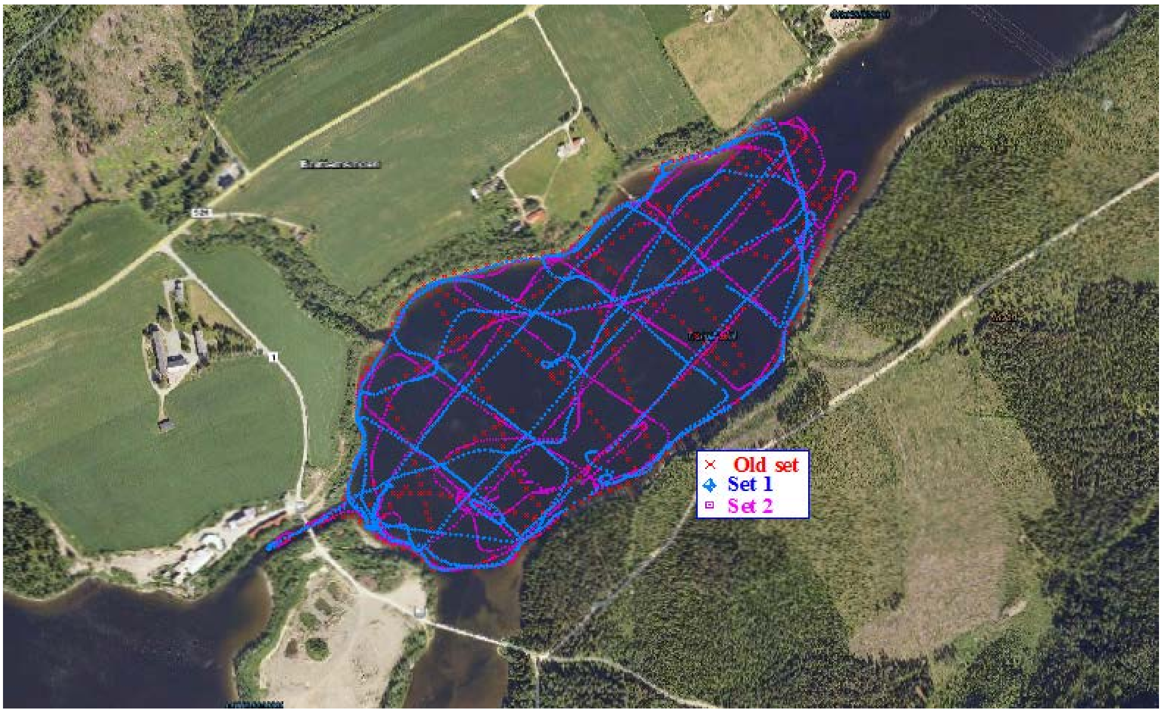


Figure 2-7: Survey track lines and the surveyed area in Selbusjøen (picture from http://kart.gulesider.no/)

2.3.1 Field Survey in Kulekhani Reservoir

The survey area is located about 3 km upstream of the dam and about 850 m downstream of the check dam (Figure 2-8). The area is within live storage zone of the reservoir and highly sensitive area in terms of sediment deposition rate and its distribution as the area is only 850 m downstream of the check dam where one of the tributary Chitlang Khola enters the reservoir. Bathymetric survey in Kulekhani reservoir for the accuracy of DGPS bathymetric survey purpose was carried out on 30th November and 1st December 2010. On 30th November, "Survey 1" was carried out along the shoreline and cross sections in addition to longitudinal surveys along the middle part of the survey area. The "Survey 2" was carried out on 1st December. The survey points in Survey 1 are denser than in Survey 2 with different track lines. A local boat was used for the survey. The boat was about 5 m long and 1 m wide at the center. The survey track lines and the surveyed area in the Kulekhani reservoir are shown in Figure 2-8 (water level of the reservoir was lower than 1515 masl when the picture was taken). All the points were recorded at a distance of 5 m.

The water level of the reservoir, the number of points, the survey area and the survey point density for Survey 1 and Survey 2 are tabulated in Table 2-4.

Table 2-4: Field survey details in the Kulekhani reservoir

S. No.	**Description of surveys**	**Water level, (masl)**	**Survey points**		**Area, (m^2)**	**Survey points per 10,000 m^2**
			Raw	**Processed**		
1	Survey 1	1,521.19	2,356	2,339	254,330	92
2	Survey 2	1,521.15	2,118	2,100	255,383	82

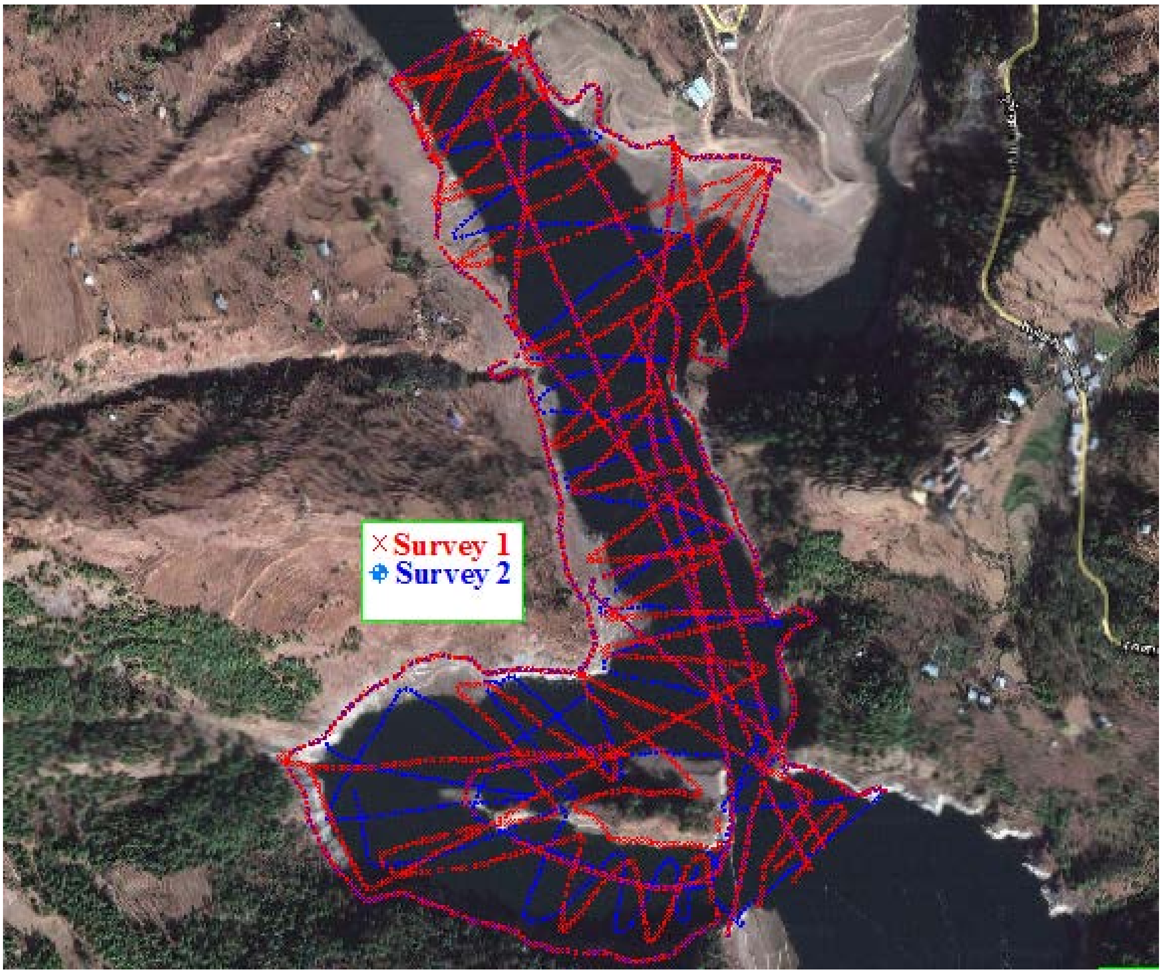


Figure 2-8: Survey track lines and the surveyed area in Kulekhani reservoir (picture from Google Earth at situation with low water level)

2.3.2 Field Survey in Kaligandaki A Reservoir

The survey area includes headworks area of the reservoir and it was extended to 300 m upstream to Kaligandaki and Andhi Khola rivers from their confluence. The Bathymetric survey in Kaligandaki A reservoir for the accuracy of DGPS bathymetric survey purpose was carried out on 14th, 15th and 17th December 2010. Hydro Lab, NEA team, Dr. Durga Sangroula and the author were involved in this survey. On 14th and 15th Survey 1 was carried out with denser intensity than surveyed on 17th December (Survey 2). The survey track line includes shoreline, cross sections and centreline. The survey track lines and the surveyed area in Kaligandaki A reservoir are shown in Figure 2-9. Total 1,344 and 663 points were recorded during the first and second surveys, respectively. All the points were recorded at interval of 5 m.

The water level, survey points, area, density and volume in the Kaligandaki A reservoir are tabulated in Table 2-5.

Table 2-5: Field survey details in the Kaligandaki A reservoir

S. No.	Description of surveys	Water level, (masl)	Survey points		Area, (m^2)	Survey points per 10,000 m^2
			Raw	Processed		
1	Survey 1	520.92	1,355	1,344	135,341	99
2	Survey 2	521.88	680	663	134,072	49

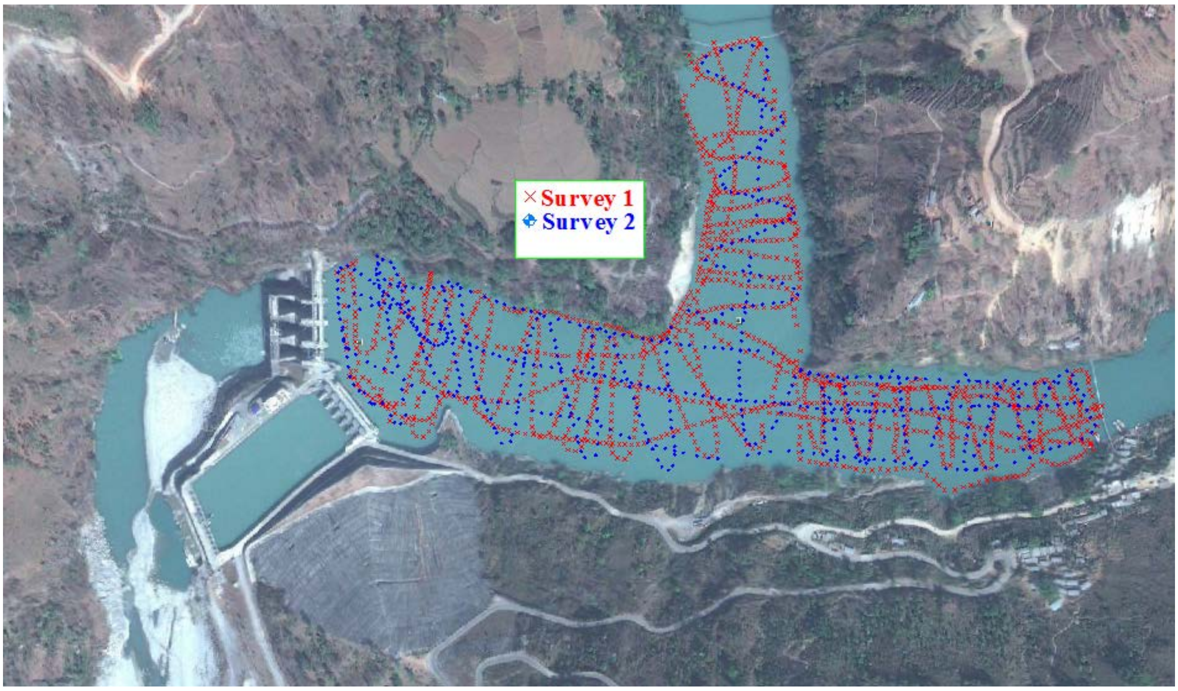


Figure 2-9: Survey track lines and the surveyed area in Kaligandaki A reservoir (picture from Google Earth)

2.4 RESULTS AND DISCUSSION

The bathymetric surveys of all the reservoirs were analysed, maps generated and storage capacities for all the surveyed area of reservoirs were calculated using the software “Surfer 8”.

Further maps are generated for the residuals (difference in depth) between the survey data with different equipment. Residuals command is used to obtain vertical difference between elevation (Z value) in the data file and interpolated elevation (Z value) on a gridded surface. A residual is the difference between the elevation (Z value) of a point in a data file and the interpolated elevation (Z value) at the same easting and northing (XY) location on a gridded surface. The residuals (difference in depth) are calculated at all measured data points and the maps are generated. Surfer uses a bilinear interpolation method to calculate Z values at points that do not coincide with grid nodes (Golden Software, 2002).

2.4.1 Map Generation

Øvre Leirfoss Tailrace Pond

The bathymetric maps of Øvre Leirfoss tailrace pond were developed for each survey data. The generated maps are shown in Figure 2-10 and Figure 2-11. To compare data and track line of the surveys each map were overlaid and prepared a combined map. The combined map is shown in Figure 2-11. The maps show the track lines and the contours.

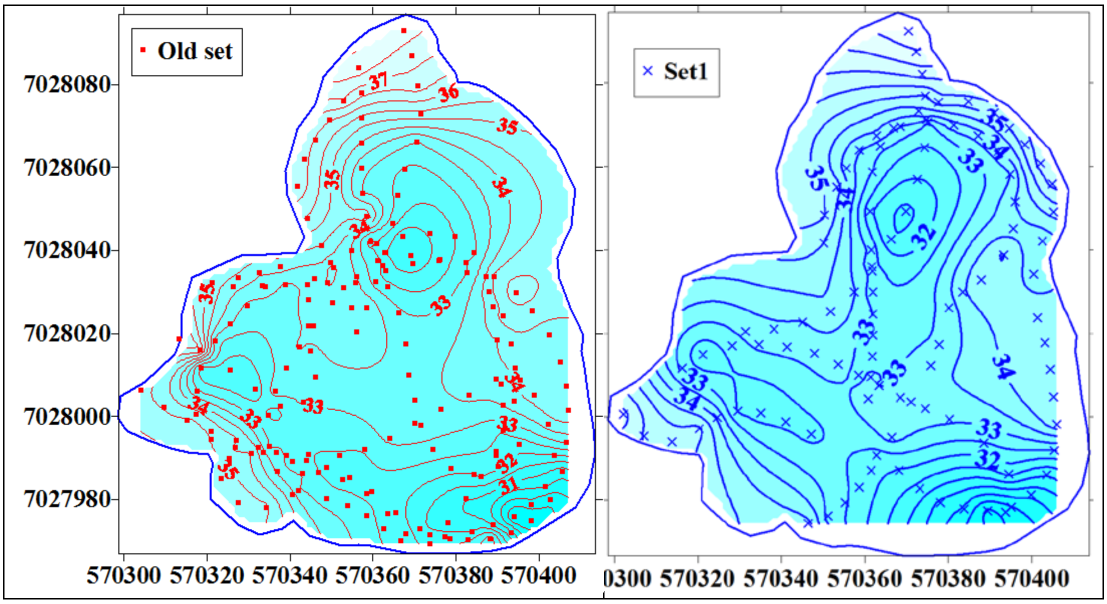


Figure 2-10: Bathymetric map of Øvre Leirfoss tailrace pond with Old set and Set 1

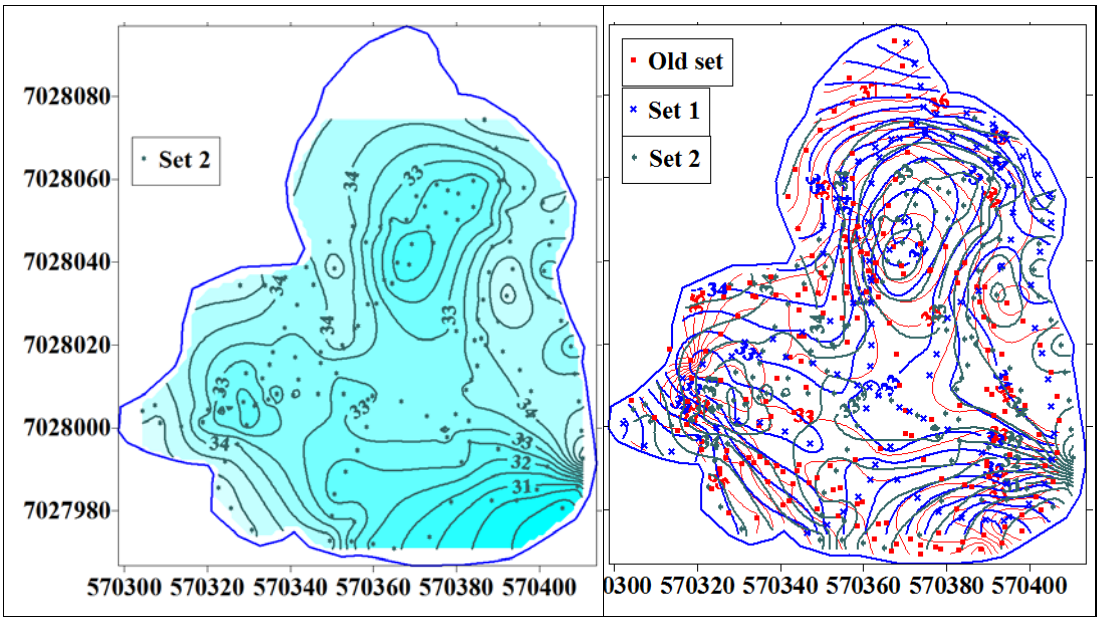


Figure 2-11: Bathymetric map of Øvre Leirfoss tailrace pond with Set 2 and overlaid map

The maps of residuals (difference of depths) between survey data sets of Old set and Set 1, Old set and Set 2 and Set 1 and Set 2 are shown in Figure 2-12, Figure 2-13 and Figure 2-14, respectively.

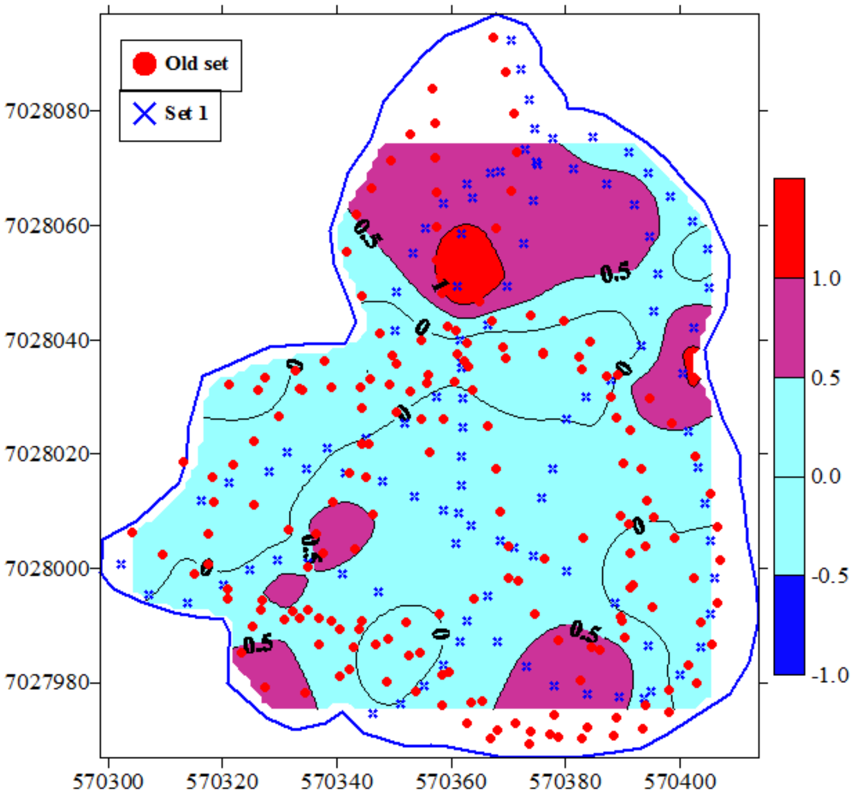


Figure 2-12: Residuals map between survey data sets of Old set and Set 1 (Øvre Leirfoss)

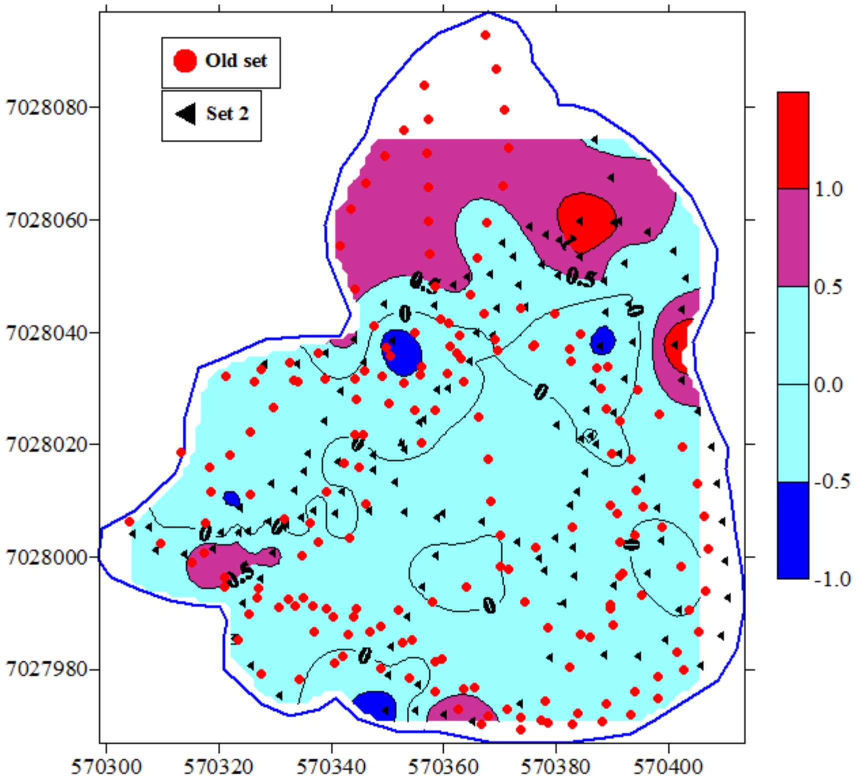


Figure 2-13: Residuals map between survey data sets of Old set and Set 2 (Øvre Leirfoss)

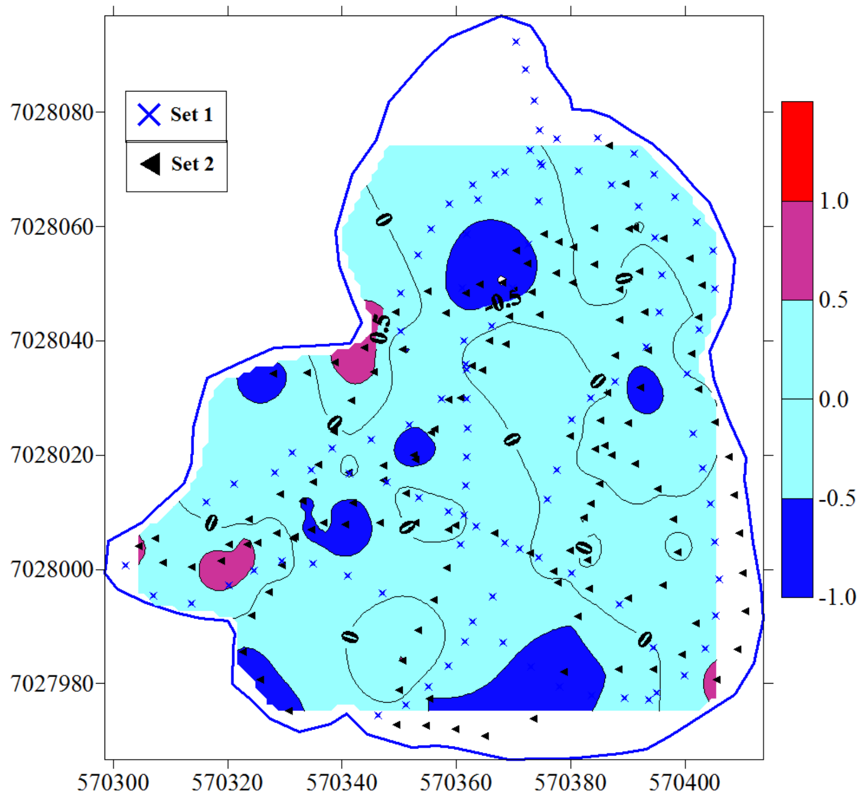


Figure 2-14: Residuals map between survey data sets of Set 1 and Set 2 (Øvre Leirfoss)

Figure 2-12 to Figure 2-14 show that the deviation in the depth between the survey data sets from different equipment varied from -1 m to 1 m. However, the majority of the deviations are within -0.5 m to 0.5 m. The deviation in depth is higher in deep areas and in the areas where survey points and track-lines do not coincide.

Selbusjøen Lake

The bathymetric maps of Selbusjøen Lake (close to the outlet) are shown in Figure 2-15 and Figure 2-16. To compare the data and track lines of the surveys, each map was overlaid and a combined map was prepared. The combined map is shown in Figure 2-16. The maps show the track lines along the shoreline, middle part and cross sections including contours.

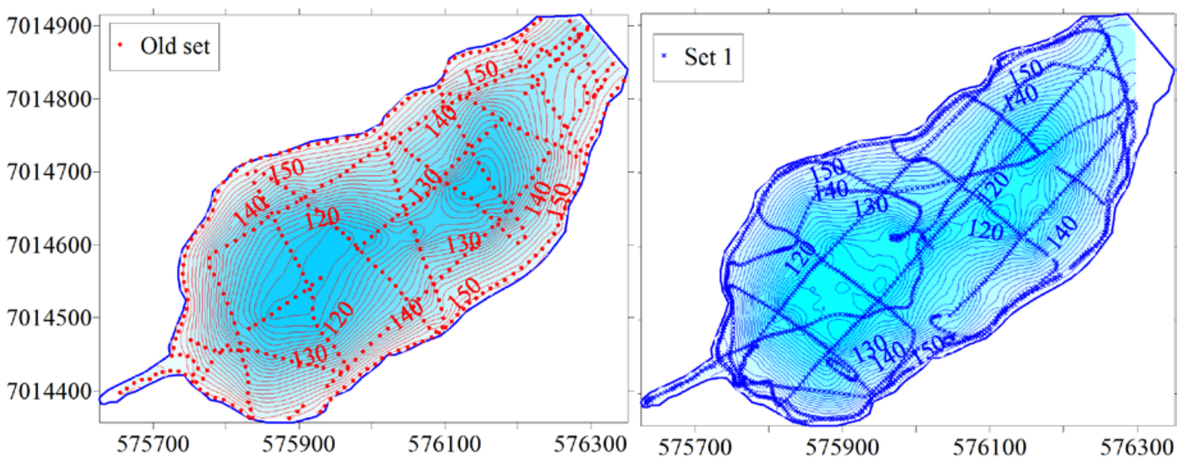


Figure 2-15: Bathymetric map of Selbusjøen Lake with Old set and Set 1

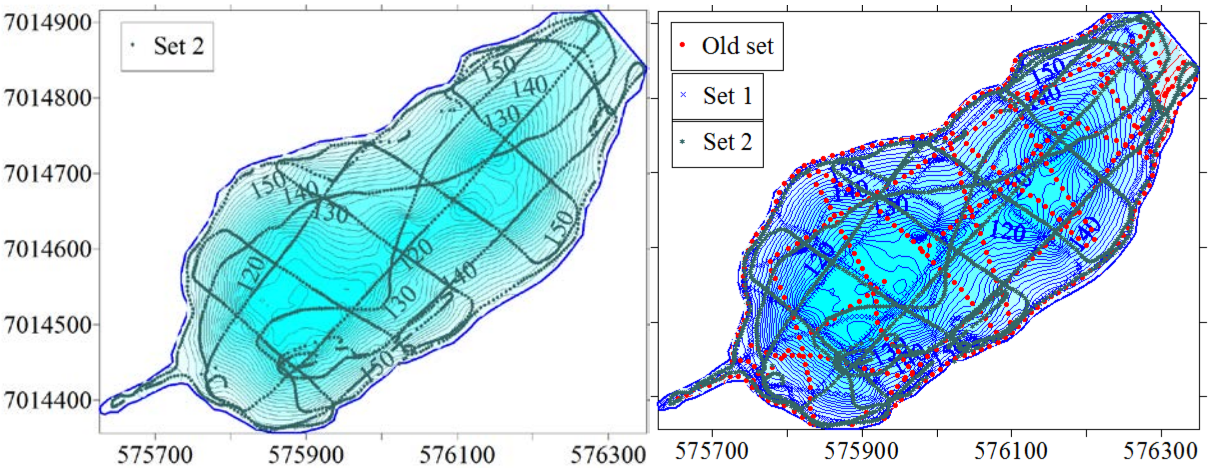


Figure 2-16: Bathymetric map of Selbusjøen Lake with Set 2 and overlaid map

The maps show that the contours are matching and consistent with the data set of each other.

Maps of residuals (difference of depths) between survey data sets of Old set and Set 1, Old set and Set 2 and Set 1 and Set 2 are shown in Figure 2-17, Figure 2-18 and Figure 2-19, respectively.

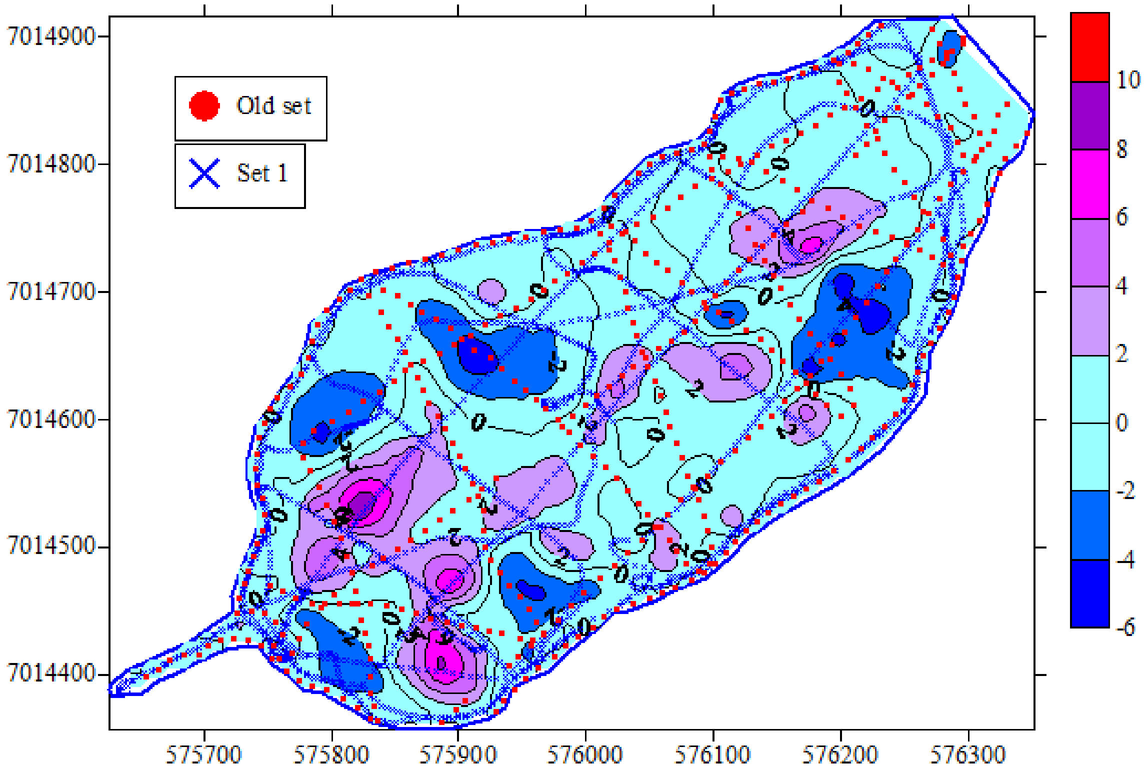


Figure 2-17: Residuals map between survey data sets of Old set and Set 1 (Selbusjøen)

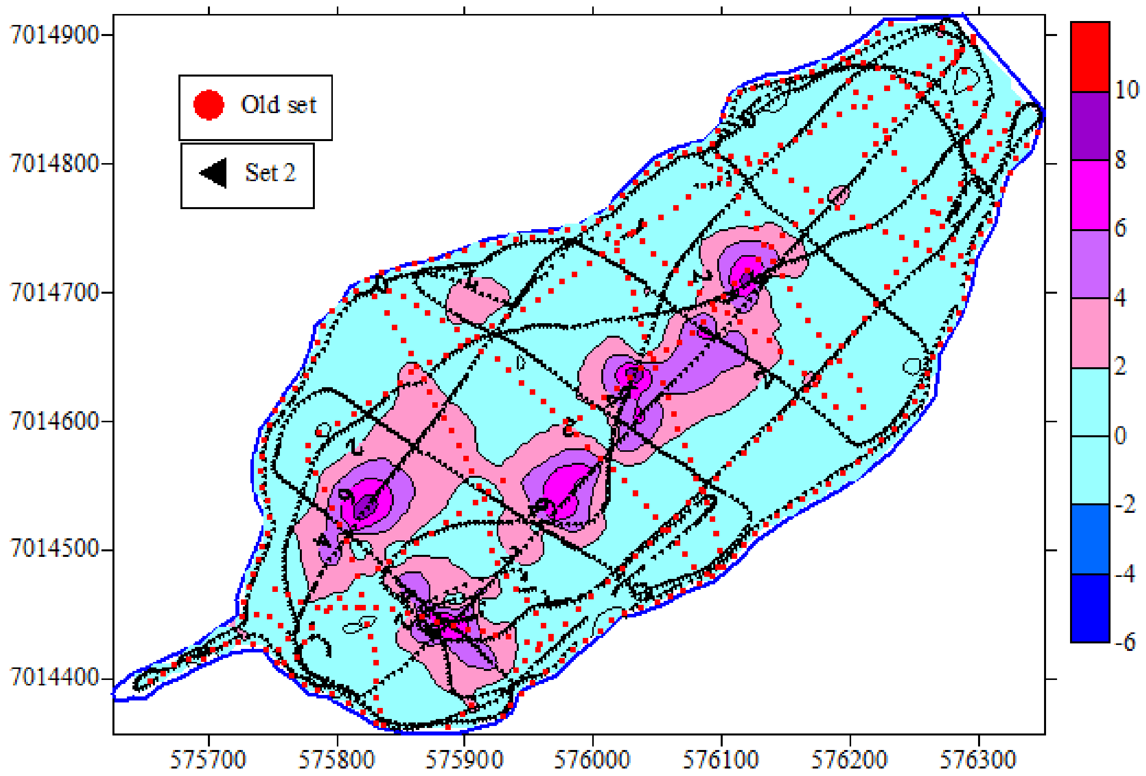


Figure 2-18: Residuals map between survey data sets of Old set and Set 2 (Selbusjøen)

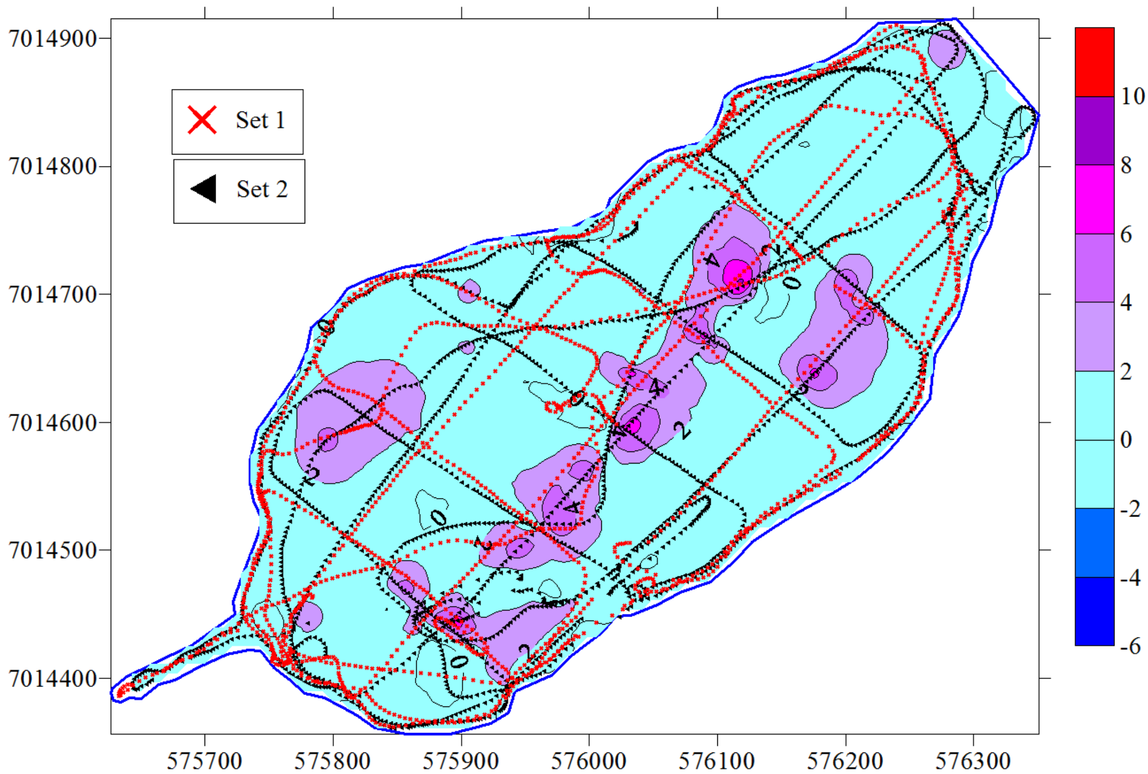


Figure 2-19: Residuals map between survey data sets of Set 1 and Set 2 (Selbusjøen)

The maps of residuals in Selbusjøen Lake show that the deviation in the depth between survey data sets from different equipment varied from -6 m to 10 m. However, the majority of the deviations are within -2 m to 2 m.

Kulekhani Reservoir

The bathymetric maps of Survey 1 and Survey 2 with combination of all data set of survey in the middle part of the Kulekhani reservoir are shown in Figure 2-20. The maps show track lines along the shoreline, middle part and cross sections including contours.

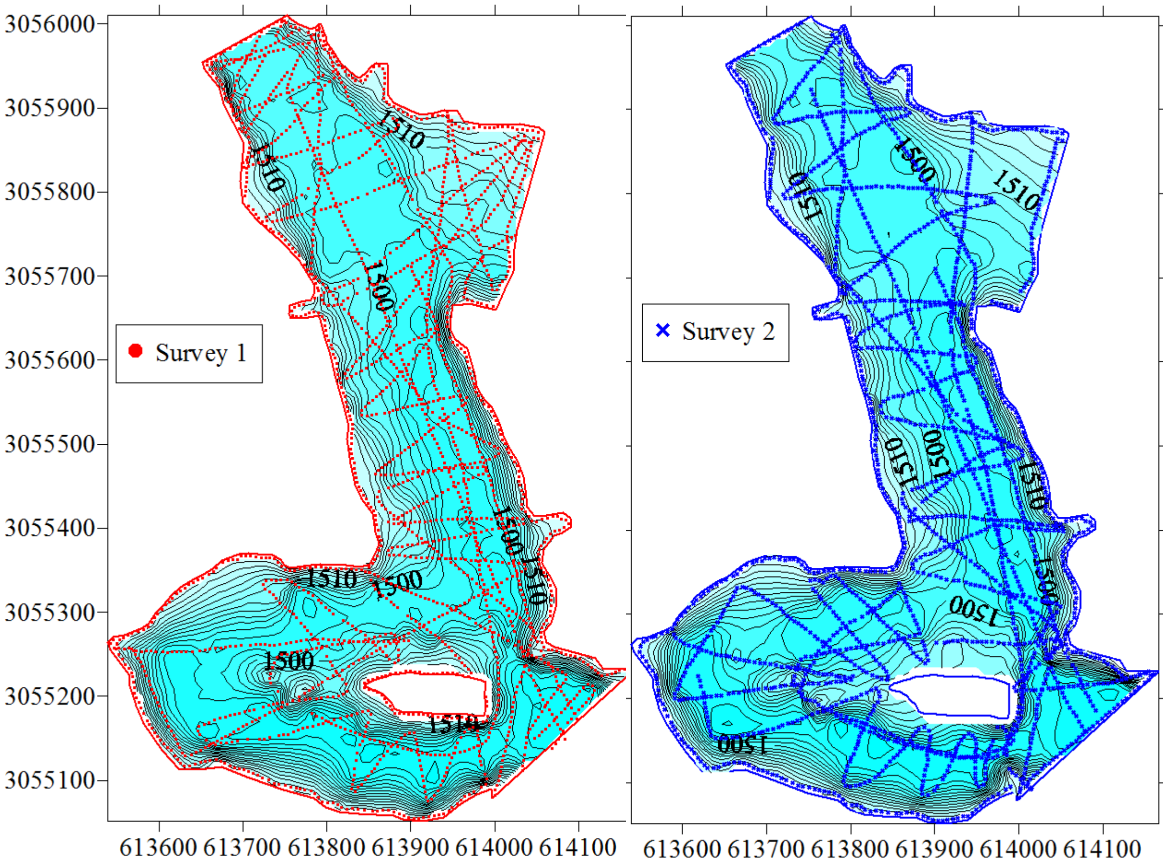


Figure 2-20: Bathymetric map at middle part of Kulekhani reservoir with Survey 1 and Survey 2

Further, the map of the residuals (difference in depth) is generated for survey data sets between Survey 1 and Survey 2. The map of residuals is shown in Figure 2-21.

Figure 2-21 shows that the deviation in the depth varied from -2 m to 3 m. However, majority of the deviations are within -1 m to 1 m.

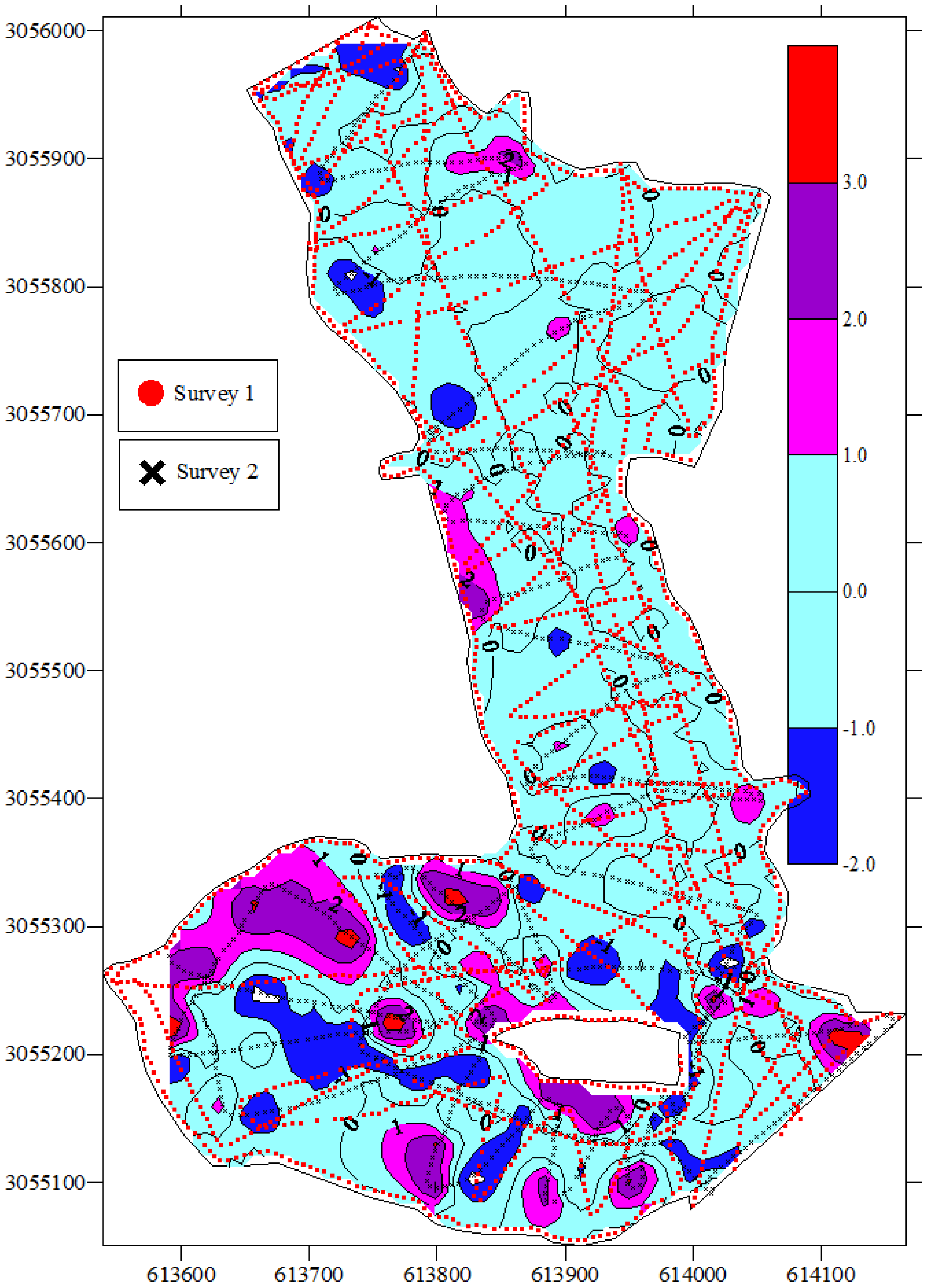


Figure 2-21: Residuals map between survey data sets of Survey 1 and Survey 2 (Kulekhani)

Kaligandaki A Reservoir

The bathymetric maps of Survey 1 and Survey 2 carried out in Kaligandaki A reservoir are shown in Figure 2-22 and Figure 2-23. The maps include track lines and contours.

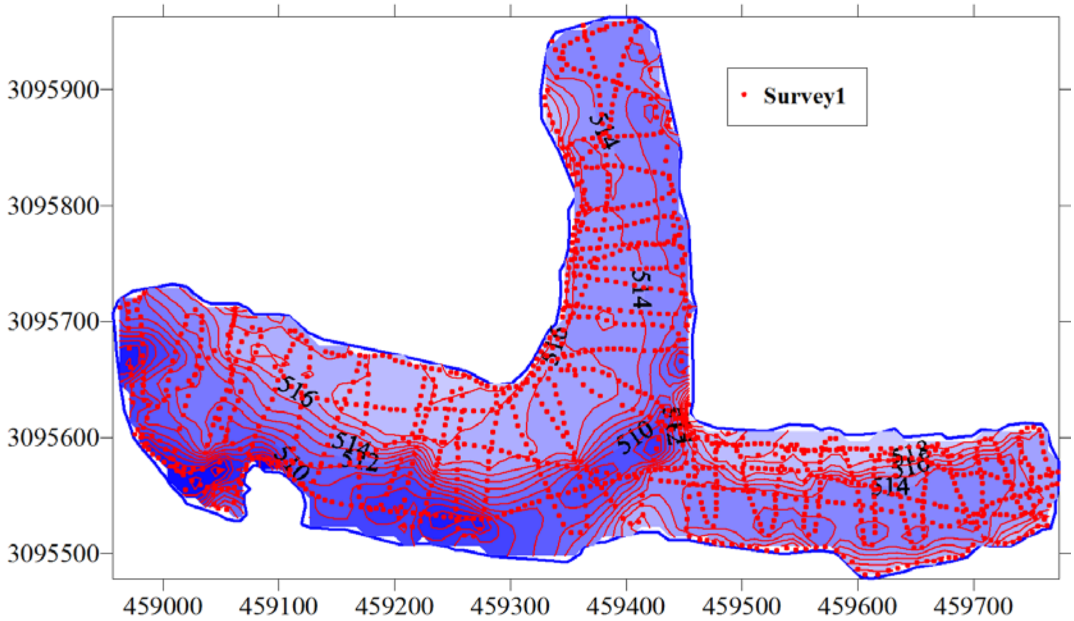


Figure 2-22: Bathymetric map of Kaligandaki A reservoir for Survey 1

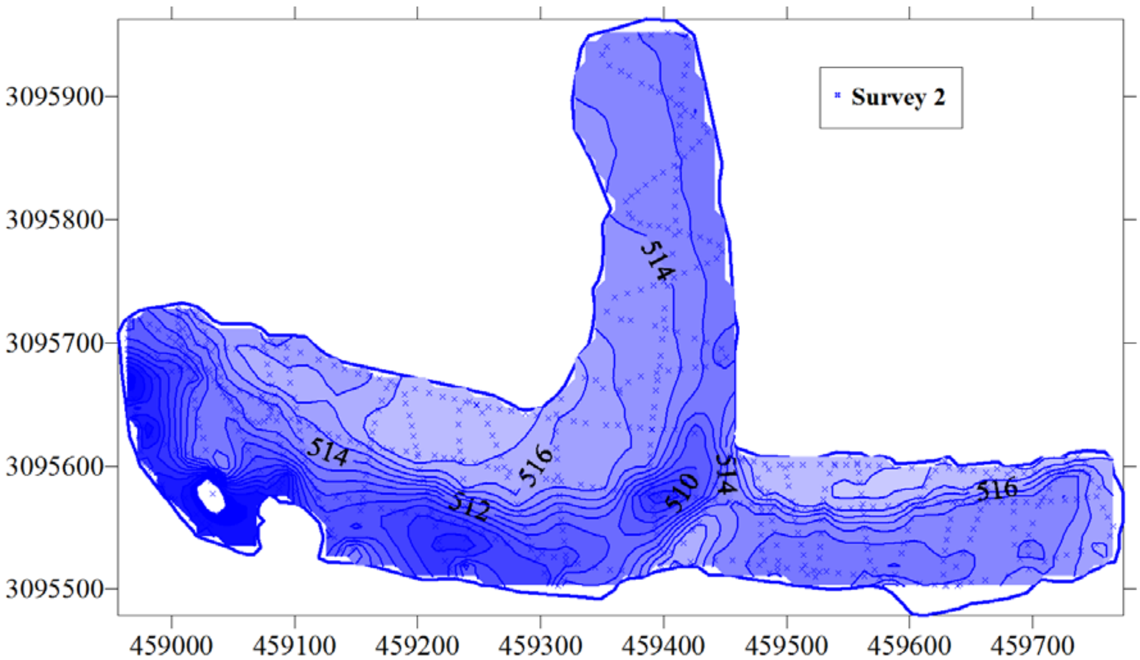


Figure 2-23: Bathymetric map of Kaligandaki A reservoir for Survey 2

The map of residuals (difference of depths) between survey data sets of Survey 1 and Survey 2 is shown in Figure 2-24.

Figure 2-24 shows that the deviation in the depth varied from -3 m to 3 m. However, the majority of deviations are within -1 m to 1 m.

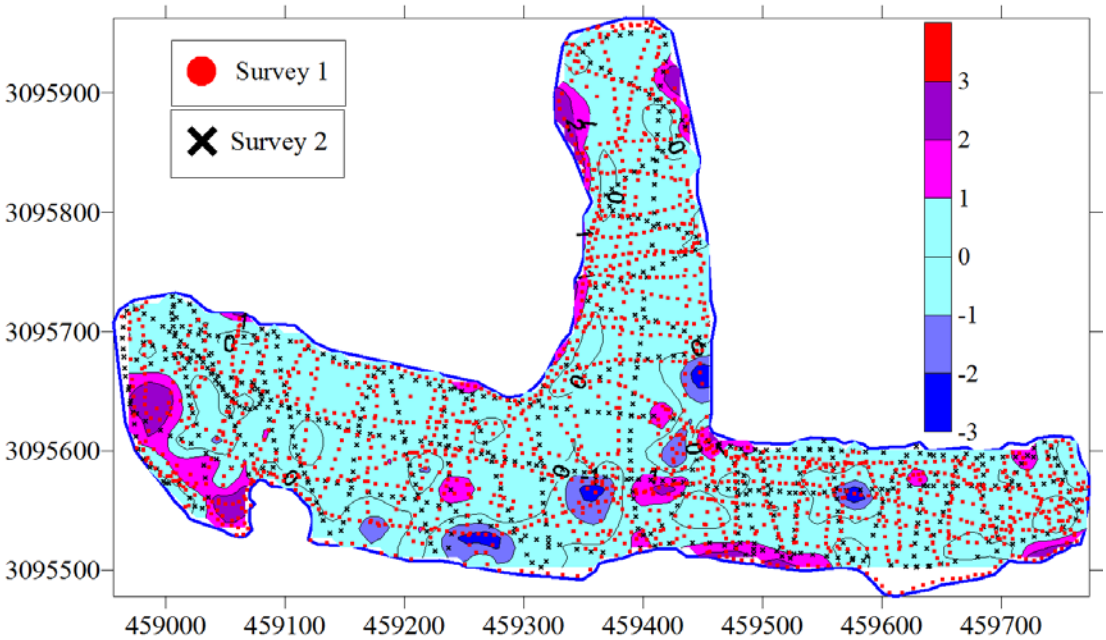


Figure 2-24: Residuals map between survey data sets of Survey 1 and Survey 2 (Kaligandaki A)

The residual maps show that higher difference in depths between two surveys data sets are located where the survey points (track-lines) do not coincide. This is due to the fact that the method of interpolation of Surfer program, that uses a bilinear interpolation method to calculate Z values at points do not coincide with grid nodes.

2.4.2 CAPACITY ESTIMATION

Øvre Leirfoss

The volume of the reservoir is estimated using Surfer 8 software. Figure 2-25 shows surface area-elevation and capacity-elevation curves of surveyed area of Øvre Leirfoss tailrace pond based on survey with different instrument set used for the surveying are tabulated in Table 2-6.

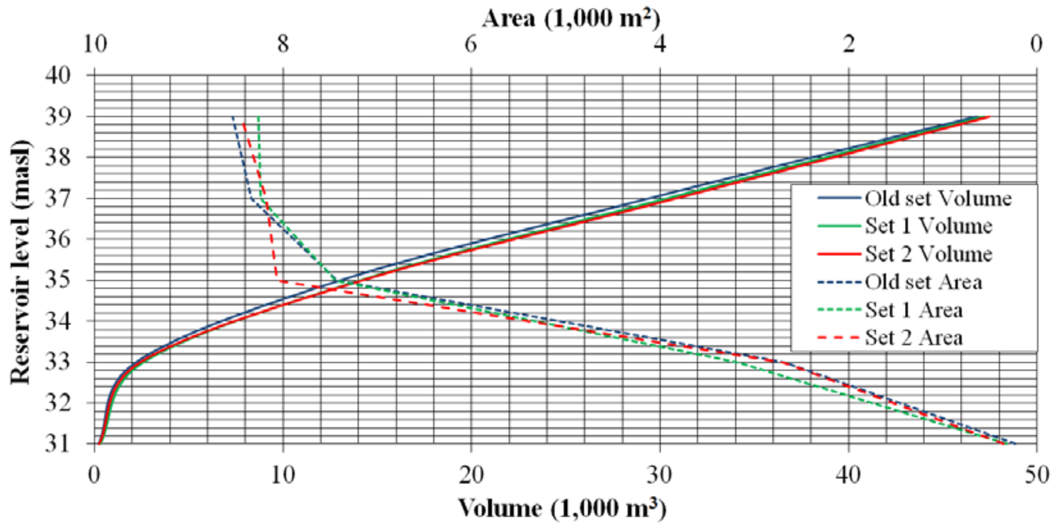


Figure 2-25: Area-elevation and capacity-elevation curves of surveyed area of Øvre Leirfoss

Table 2-6: Reservoir area and capacity of surveyed area of Øvre Leirfoss

Surveys	**Old set**		**Set 1**		**Set 2**	
Elevation (masl)	**Area (1,000 m^2)**	**Volume (1,000 m^3)**	**Area (1,000 m^2)**	**Volume (1,000 m^3)**	**Area (1,000 m^2)**	**Volume (1,000 m^3)**
31.0	0.24	0.21	0.33	0.29	0.36	0.24
33.0	2.68	2.20	3.17	2.58	2.69	2.42
35.0	7.42	13.03	7.44	14.08	8.06	14.20
37.0	8.34	29.40	8.24	30.36	8.18	30.83
39.0	8.54	46.78	8.26	47.19	8.45	47.48

A relative percent difference for the elevation-capacity curves comparing bathymetric survey data set with different equipment at the Øvre Leirfoss tailrace pond are shown in Figure 2-26. It should be noted that large percentage difference may occur even though the magnitude of the difference is small when area and volume is small. At the Øvre Leirfoss tailrace pond, volume differences are 0.9, 1.5 and 0.6 percent for the survey data set between Old set and Set 1, Old set and Set 2 and Set 1 and Set 2, respectivly for water level at 39 masl (Figure 2-26). But the volume difference in percent is higher at lower water level even though the magnitude of difference is small.

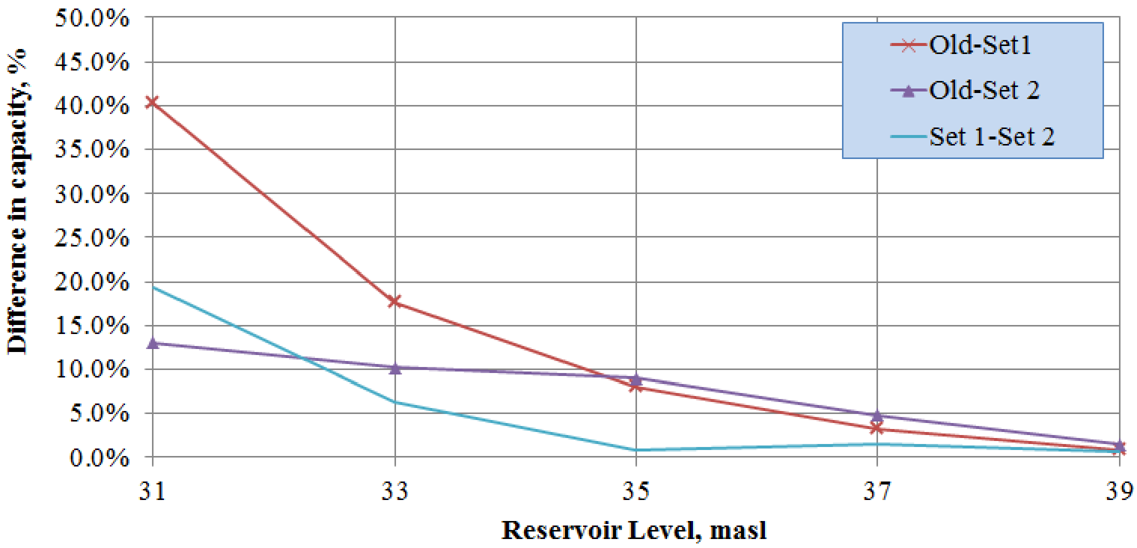


Figure 2-26: A relative percent difference of elevation-capacity curves (Øvre Leirfoss)

Selbusjøen

The surface area-elevation and capacity-elevation curves of Selbusjøen based on survey with different instrument set used for the surveying are shown in Figure 2-27 and tabulated in Table 2-7.

The area and capacity elevation curves show that the area and capacity curves of all data sets follows same line and approximately overlay each other.

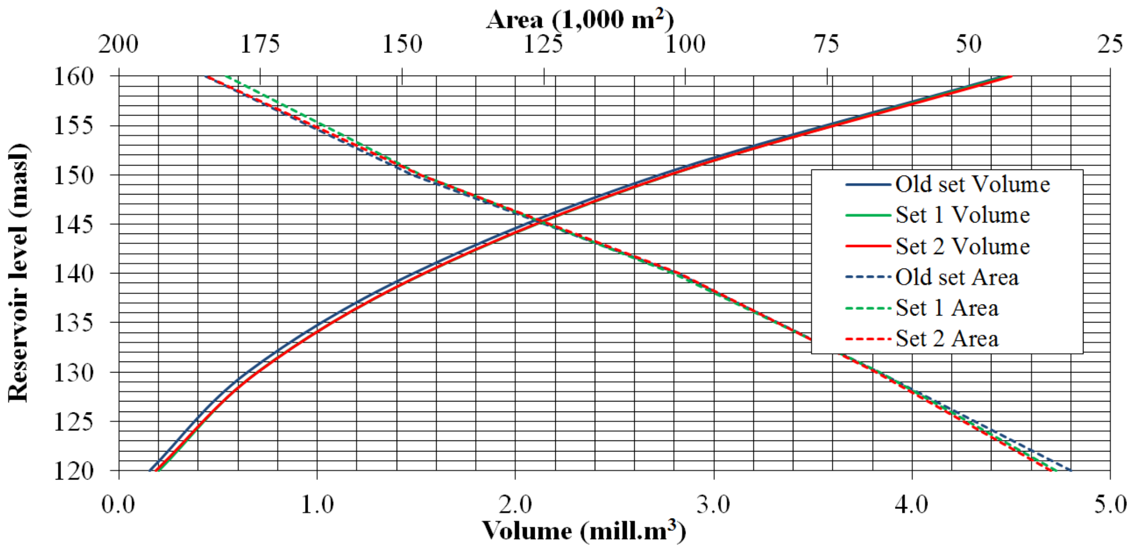


Figure 2-27: Area-elevation and capacity-elevation curves of Selbusjøen

Table 2-7: Reservoir area and capacity of Selbusjøen

Surveys	Old set		Set 1		Set 2	
Elevation (masl)	Area (1,000 m²)	Volume (mill. m³)	Area (1,000 m²)	Volume (mill. m³)	Area (1,000 m²)	Volume (mill. m³)
120.0	31.97	0.16	34.57	0.20	35.60	0.19
130.0	65.83	0.65	65.76	0.70	66.34	0.70
140.0	101.49	1.48	102.20	1.54	101.16	1.54
150.0	148.15	2.73	146.67	2.78	146.78	2.77
160.0	184.75	4.46	181.09	4.48	184.71	4.50

A relative difference in percent for the elevation-capacity curves comparing the bathymetric survey data set with different equipment at Selbusjøen are shown in Figure 2-28. The volume differences are 0.4, 0.9 and 0.5 percent for the survey data set between Old set and Set 1, Old set and Set 2 and Set 1 and Set 2 respectivly for water level at 160 masl.

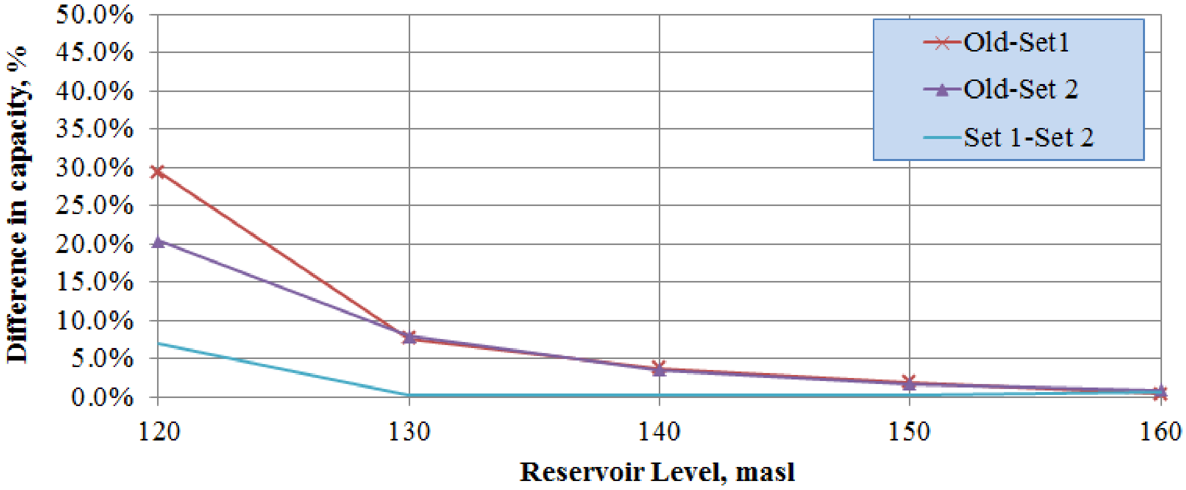


Figure 2-28: A relative percent difference of elevation-capacity curves (Selbusjøen)

Kulekhani Reservoir

The surface area-elevation and capacity-elevation curves of Kulekhani reservoir based on Survey 1, Survey 2 and combination of both survey are tabulated in Table 2-8 and shown in Figure 2-29.

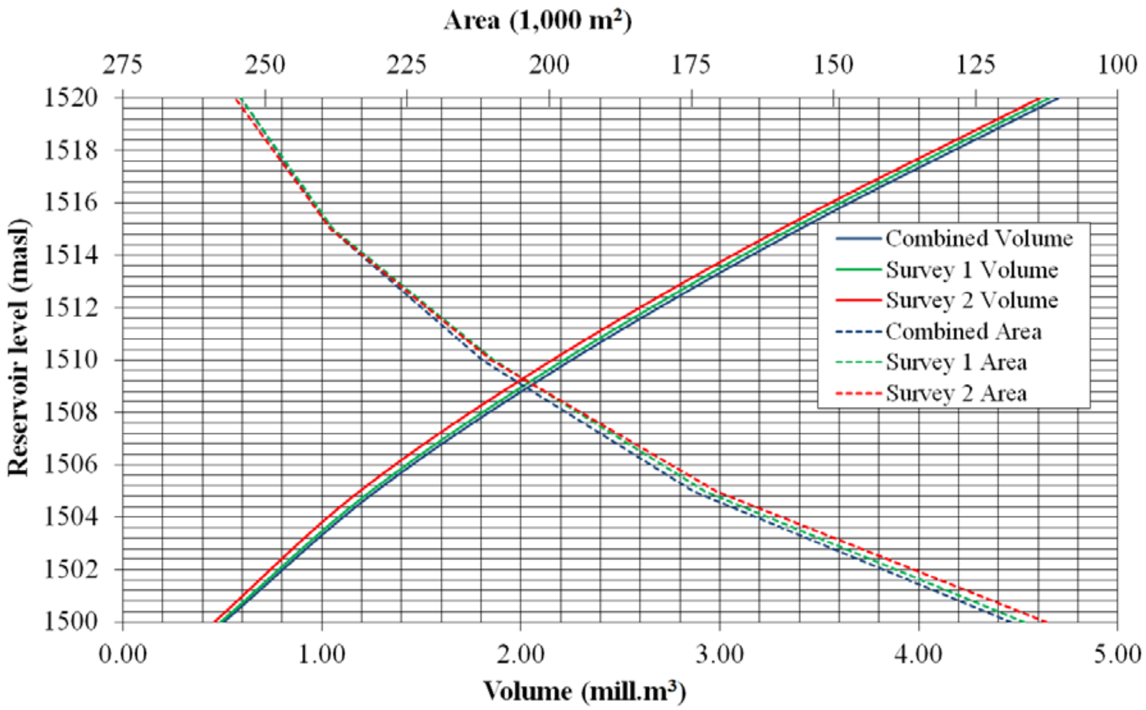


Figure 2-29: Area-elevation and capacity-elevation curves of Kulekhani reservoir

Table 2-8: Reservoir area and capacity of Kulekhani reservoir

Surveys	**Combined**		**Survey 1**		**Survey 2**	
Elevation (masl)	**Area (1,000 m^2)**	**Volume (mill. m^3)**	**Area (1,000 m^2)**	**Volume (mill. m^3)**	**Area (1,000 m^2)**	**Volume (mill. m^3)**
1500	118.7	0.51	116.3	0.49	112.5	0.46
1505	174.6	1.27	172.6	1.25	170.5	1.19
1510	211.7	2.25	209.6	2.22	210.1	2.16
1515	238.5	3.40	238.1	3.36	238.4	3.31
1520	254.3	4.70	254.3	4.66	255.4	4.61

A relative difference in percent for the elevation-capacity curves comparing the bathymetric survey data set with different surveys at the Kulekhani reservoir are shown in Figure 2-30. The volume differences are 1.0, 2.0 and 1.0 percent for the survey data set between Combined and Survey 1, Combined and Survey 2 and Survey 1 and Survey 2 respectivly for water level at 1520 masl. But the volume difference is only 9.4 percent at lowest water level. This difference is small comparing to the difference from other surveys in Øvre Leirfoss, Selbusjøen and Kaligandaki.

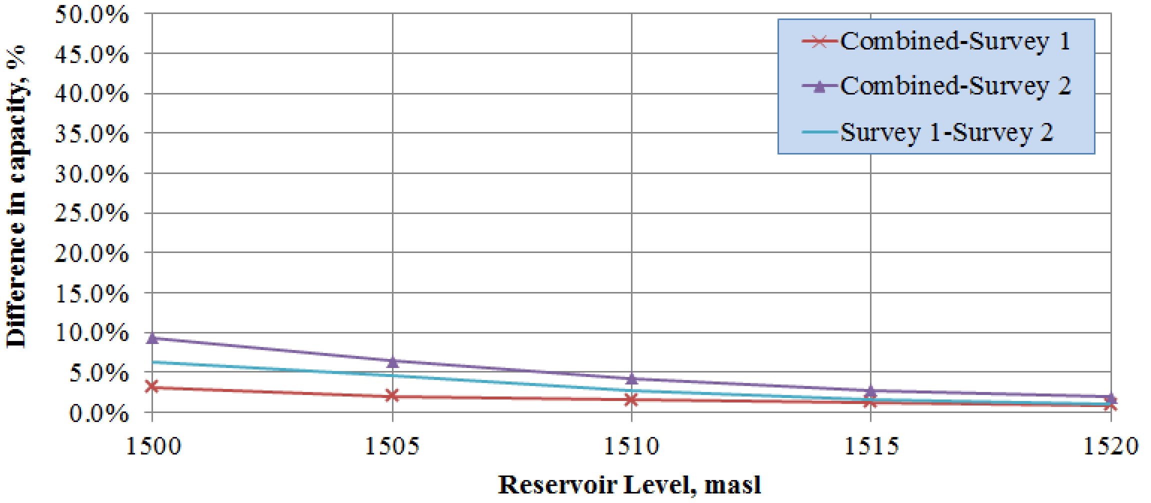


Figure 2-30: A relative percent difference of elevation-capacity curves (Kulekhani reservoir)

Kaligandaki A Reservoir

The surface area-elevation and capacity-elevation curves of Kaligandaki A reservoir based on Survey 1, Survey 2 and combination of both survey are shown in Figure 2-31 and tabulated in Table 2-9.

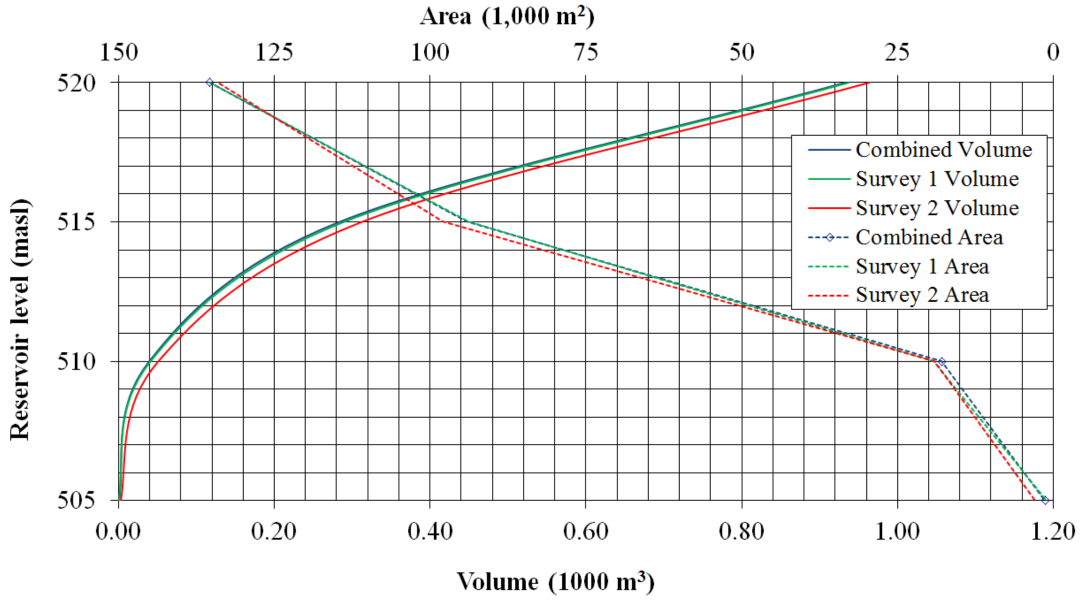


Figure 2-31: Area-elevation and capacity-elevation curves of Kaligandaki A reservoir

Table 2-9: Reservoir area and capacity of Kaligandaki A reservoir

Surveys	Combined		Survey 1		Survey 2	
Elevation (masl)	Area (1,000 m^2)	Volume (mill. m^3)	Area (1,000 m^2)	Volume (mill. m^3)	Area (1,000 m^2)	Volume (mill. m^3)
500	0.00	0.00	0.00	0.00	0.00	0.00
505	1.35	1.04	1.02	0.70	2.85	3.08
510	17.94	39.51	19.22	41.15	19.03	51.22
515	94.11	283.52	93.83	289.37	98.02	312.32
520	135.35	932.09	135.34	936.80	134.07	963.08

A relative difference in percent for the elevation-capacity curves comparing the bathymetric survey data set with different surveys at the Kaligandaki reservoir are shown in Figure 2-32. The volume differences are 0.5, 3.3 and 2.8 percent for the survey data set between Combined and Survey 1, Combined and Survey 2 and Survey 1 and Survey 2, respectivly for water level at 515 masl. But the volume difference is highest (up to 343%) at lowest water level comparing to the difference in other surveys in Øvre Leirfoss, Selbusjøen and Kulekhani.

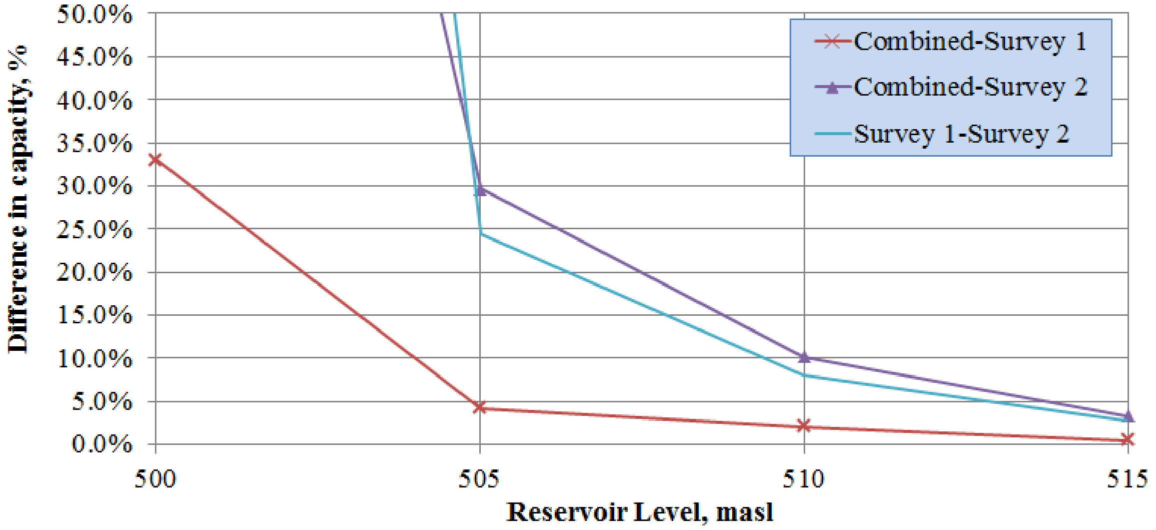


Figure 2-32: A relative percent difference of elevation-capacity curves (Kaligandaki)

2.4.3 Assesment of Depth Accuracy

To evaluate the accuracy of measured depths from bathymetric survey, difference in depth (residuals) were compared to data sets with different equipment and surveys. The difference in depths between the different survey data sets were calculated using the software "Surfer 8" as explained in Section 2.4. The maps of residuals (difference in depths) between the different surveys are already presented in Section 2.4.1. The summary of difference in depths between different survey data sets are presented in Table 2-10.

Table 2-10: Summary of difference in depths between the different survey data sets

Location	Maximum depth, m	Difference between	Difference in depth calculation		
			Maximum	Minimum	Average
Øvre Leirfoss	9.3	Old set-Set1	1.57	-0.51	0.21
		Old set-Set2	1.48	-1.03	0.15
		Set1-Set2	1.07	-1.10	-0.08
Selbusjøen	48.6	Old set-Set1	9.64	-6.39	0.24
		Old set-Set2	10.89	-11.47	0.49
		Set1-Set2	8.64	-9.32	0.26
Kaligandaki	18.0	Survey 1-Survey 2	3.06	-3.04	0.24
Kulekhani	31.6	Survey 1-Survey 2	2.00	-1.74	-0.02

The maximum difference in depth varied from 1.07 m in Øvre Leirfoss to 10.89 m in Selbusjøen. The average difference in depth varied from -0.02 m in Kulekhani to 0.49 m in Selbusjøen.

Further numbers of points are counted for the similar deviation in depths between the different survey data sets and tabulated in Table 2-11 and shown in Figure 2-33.

Table 2-11: Numbers of points presented in the similar deviation in depths

Location	**Øvre Leirfoss**		**Selbusjøen**		**Kaligandaki**		**Kulekhani**	
Maximum depth, m	9.3		48.6		18		31.6	
Difference in depth, m	**Surveyed points, numbers**							
	No.	**%**	**No.**	**%**	**No.**	**%**	**No.**	**%**
0 to1	124	96	1,096	52	1,088	85	1,332	84
1 to 2	5	4	440	21	128	10	202	13
2 to 3	-	0	236	11	60	5	51	3
3 to 4	-	0	169	8	7	1	-	0
More than 4	-	0	170	8	-	0	-	0
Total	129		2,111		1,283		1,585	

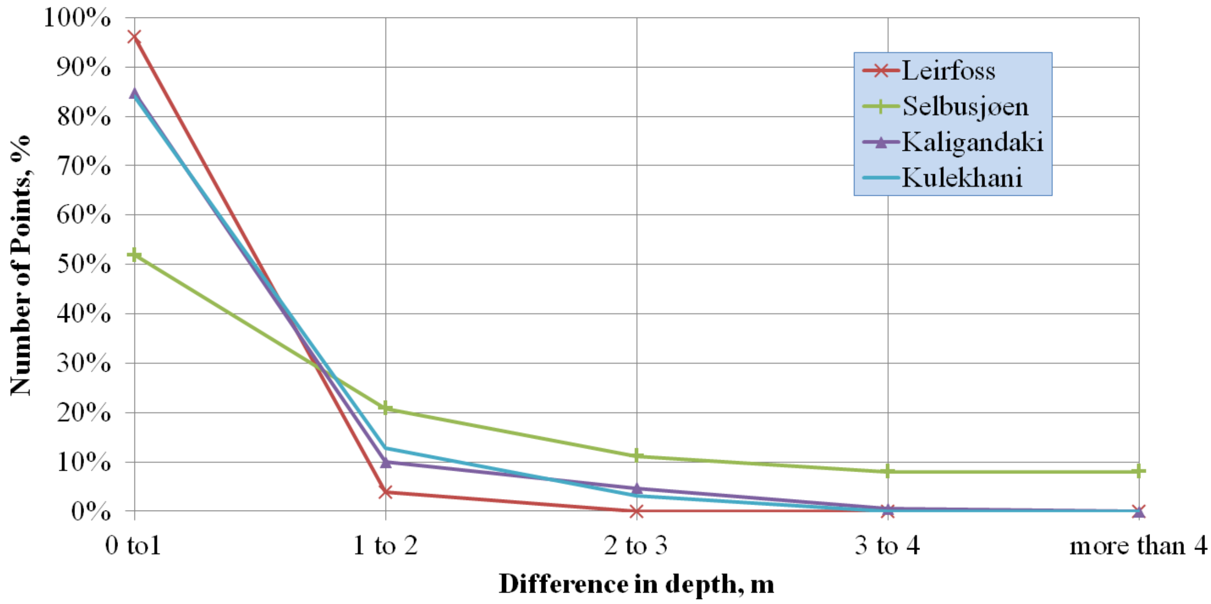


Figure 2-33: Number of points versus deviation in depths

Table 2-11 and Figure 2-33 show that the majority of the survey points are located within the difference in depth of 1 m except in the survey data set of Selbusjøen. However, some points are located more than 2 m of difference in depths. In the case of the data set of Selbusjøen survey, 52% of the total survey points are located within the difference in depth of 1 m. The percentage of number of points is gradually lowered to 8% at the difference in depth more than 4 m.

Further number of points are searched in different water depths for different deviation in depths in survey data set of Selbusjøen and shown in Figure 2-34.

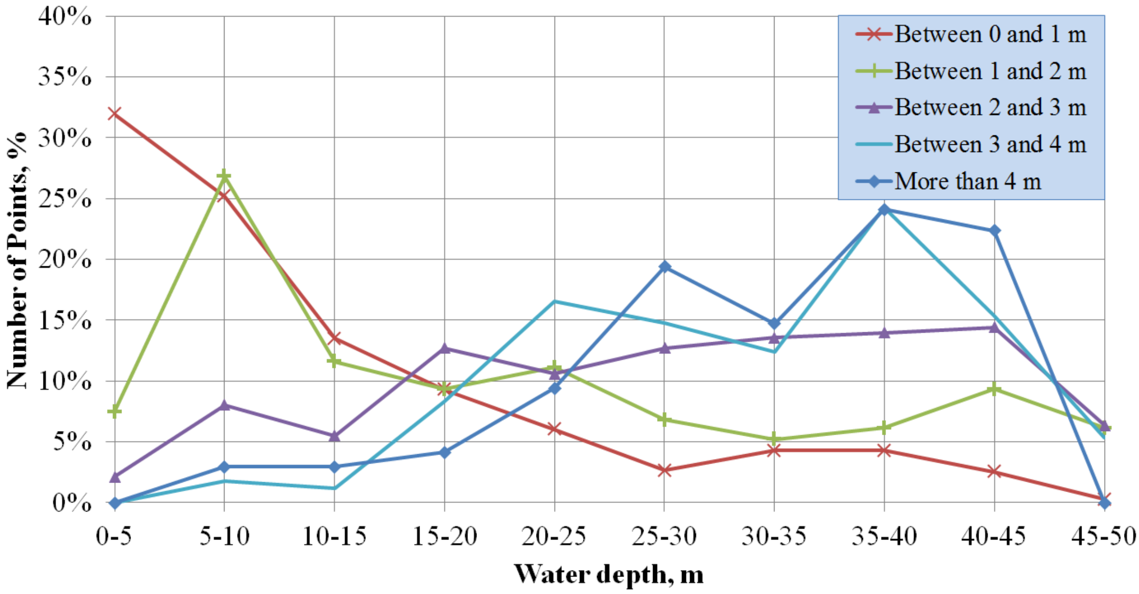


Figure 2-34: Number of points versus water depth for different difference in depth in Selbusjøen

Figure 2-34 shows the highest percentage of survey points with small difference in depth are located at shallow depths, and gradually decreases towards the deep area. It shows that the precision of any depth measurement degrades, with increment of the water depth. The accuracy of an echo-sounding measurement decreases with depth due to a number of electronic and physical factors inherent in the sound travel-time measurement process, i.e., changes in water temperature, salinity, etc.

Another reason for higher difference in depth is due to non-coincidence of the survey points. Because the Surfer program uses a bilinear interpolation method to calculate Z values at points that do not coincide with grid nodes.

The average difference in depth varied from -0.02 m in Kulekhani reservoir (Maximum depth 31.6 m) to 0.49 m in Selbusjøen Lake (maximum depth 48.6 m). The variation in deviation in depth is due to the variation of water depth, non-coincidence of the survey points and survey point density. But the average deviation in depths in Øvre Leirfoss and Kaligandaki survey data are 0.15 m and 0.24 m, respectively with a maximum depth deviation of 9.5 m and 18 m, respectively and are less than in Kulekhani reservoir (31.6 m). This is due to the variation in survey point density difference and non-coincidence of the survey points in Øvre Leirfoss and Kaligandaki. It demonstrates that the accuracy of the bathymetric survey with DGPS method can be increased with the increment of the survey point density with coverage of all area of the reservoir.

Further, the grid data were generated for the grid spacing of 3.5 m, 7 m and 14 m of the same survey data to evaluate the effect of the grid spacing on the accuracy of the measured depths. The calculated deviations in depth for grid spacing 3.5 m, 7 m and 14 m are shown in Table 2-12. The maximum and minimum deviation in depth is found up to 4.56 m and -6.86 m, respectively. The average deviation in depth varied from -0.05 m to 0.03 m.

Table 2-12: Difference in depth for the different grid spacing

Location	Surveys	Difference between	Difference in depth calculation, (m)		
			Maximum	**Minimum**	**Average**
Øvre Leirfoss	Old set	Normal(7) - Close(3.5)	0.34	-0.22	0.01
		Normal(7) - Wide(14)	0.40	-0.45	-0.03
		Close(3.5) - Wide(14)	0.48	-0.53	-0.04
	Set 1	Normal(7) - Close(3.5)	0.23	-0.13	0.01
		Normal(7) - Wide(14)	0.42	-0.55	-0.02
		Close(3.5) - Wide(14)	0.50	-0.71	-0.04
	Set 2	Normal(7) - Close(3.5)	0.58	-0.64	0.02
		Normal(7) - Wide(14)	2.25	-0.71	0.02
		Close(3.5) - Wide(14)	2.80	-0.88	-0.01
Selbusjøen	Old set	Normal(7) - Close(3.5)	0.97	-1.23	-0.02
		Normal(7) - Wide(14)	0.82	-0.41	0.004
		Close(3.5) - Wide(14)	1.58	-1.05	0.02
	Set 1	Normal(7) - Close(3.5)	0.74	-1.07	-0.01
		Normal(7) - Wide(14)	2.26	-2.58	0.03
		Close(3.5) - Wide(14)	3.11	-3.46	0.03
	Set 2	Normal(7) - Close(3.5)	3.86	-3.32	-0.002
		Normal(7) - Wide(14)	3.89	-3.24	0.01
		Close(3.5) - Wide(14)	4.51	-6.86	0.01
Kaligandaki	Survey 1	Normal(7) - Close(3.5)	0.93	-1.52	-0.003
		Normal(7) - Wide(14)	2.37	-1.87	-0.009
		Close(3.5) - Wide(14)	1.96	-1.28	-0.006
	Survey 2	Normal(7) - Close(3.5)	0.29	-0.43	-0.001
		Normal(7) - Wide(14)	2.64	-0.86	0.004
		Close(3.5) - Wide(14)	2.67	-0.78	0.005
Kulekhani	Survey 1	Normal(7) - Close(3.5)	4.56	-3.98	-0.0001
		Normal(7) - Wide(14)	3.76	-3.75	0.0030
		Close(3.5) - Wide(14)	2.42	-2.29	-0.0031
	Survey 2	Normal(7) - Close(3.5)	2.00	-1.74	-0.0001
		Normal(7) - Wide(14)	3.29	-3.10	-0.05
		Close(3.5) - Wide(14)	2.92	-2.78	-0.05

2.4.4 ASSESMENT OF VOLUMETRIC ACCURACY

Øvre Leirfoss and Selbusjøen

The summary of volume calculated from different data sets in Øvre Leirfoss and Selbusjøen are tabulated in Table 2-13. The average computed volume from three different equipments are 47,200 m^3 and 4.49 mill. m^3 in Øvre Leirfoss and Selbusjøen, respectively. The computed volume using data from Old set is lower by 0.9%, Set 1 is lower by 0.02% and Set 2 is higher by 0.59% than average volume in Øvre Leirfoss. Similarly, in Selbusjøen, maximum difference is only 0.58% between average volume and volume calculated for Old set survey.

Table 2-13: Summary of calculated volume of Øvre Leirfoss and Selbusjøen

		Øvre Leirfoss			**Selbusjøen**		
S. No.	**Description of surveys**	**Survey points per 10,000 m^2**	**Volume, (1,000m^3)**	**Deviation in volume with average volume , (%)**	**Survey points per 10,000 m^2**	**Volume, (1,000 m^3)**	**Deviation in volume with average volume (%)**
1	Old set	213	46.8	0.90	26	4,459	0.58
2	Set 1	126	47.2	0.02	118	4,475	0.22
3	Set 2	165	47.5	0.41	100	4,499	0.30
5	Combination of all survey	504	47.4	0.33	241	4,507	0.49

Further, the data are searched at distance of 10 m interval from data Set 1 and Set 2 surveyed in Selbusjøen. The summary and comparison of the results are given in Table 2-14.

Table 2-14: The comparison of results based on common data set in Selbusjøen

S. No.	**Description of surveys**	**Survey points**	**Area, (1,000 m^2)**	**Survey points per 10,000 m^2**	**Volume, (1,000 m^3)**	**Volume deviation (%)**
1	Set 1	2130	181	118	4,475	0.64
	Set 1 with 10 m	685	181	38	4,504	0.64
2	Set 2	1848	185	100	4,499	0.46
	Set 2 with 10 m	754	185	41	4,519	0.46

Table 2-14 shows that the calculated volume of Set 1 and Set 2 with survey point spacing of 10 m is 0.65% and 0.46% higher than 5 m spacing data set.

The volumetric differences between data sets with survey point spacing of 5 m and 10 m are computed depending on the elevation of Selbusjøen. A relative percent difference for the elevation-capacity curves comparing the survey data set with survey point spacing of 5 m and 10 m are shown in Figure 2-35 and tabulated in Table 2-15. The volume differences are 8.6% and 4.2% at elevation 120 masl to 0.6% and 0.5% at elevation 160 masl for Set 1 and Set 2, respectively.

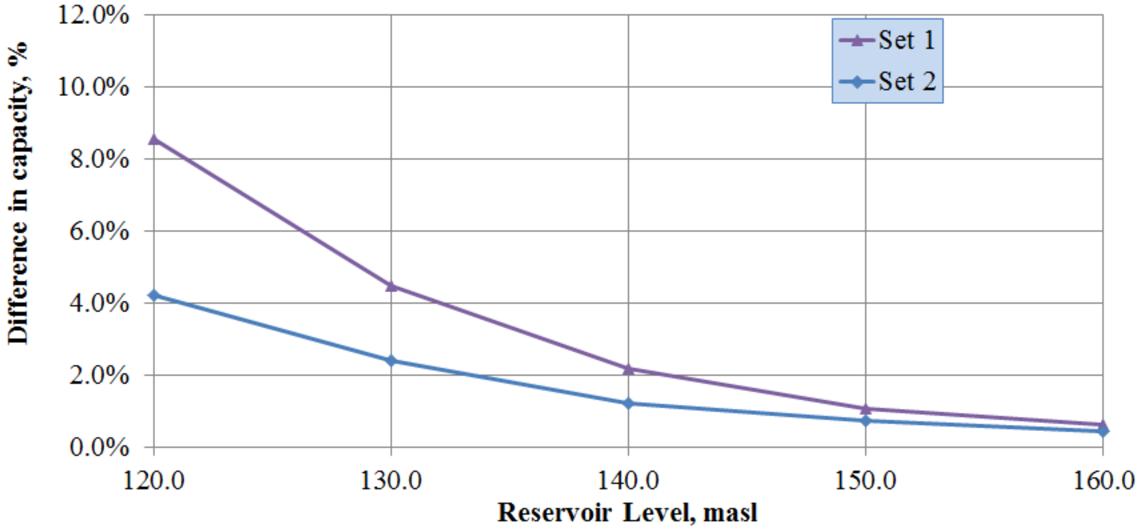


Figure 2-35: A relative percent difference of elevation-capacity curves for different survey point intervals in Selbusjøen.

Table 2-15: Reservoir capacity of Selbusjøen for different survey point spacing

Point spacing	**5 m**		**10 m**		**Difference between 5 m and 10 m**			
Equipment sets	Set 1	Set 2	Set 1	Set 2	Set 1		Set 2	
Number of points	2130	1848	685	754	1445		1094	
Survey points per 10,000 m^2	118	100	38	41	80		59	
Elevation (masl)	**Volume (mill. m^3)**		**Volume (mill. m^3)**		**Volume (mill. m^3)**	**%**	**Volume (mill. m^3)**	**%**
120	0.20	0.19	0.22	0.19	0.017	8.6	0.008	4.2
130	0.70	0.70	0.73	0.72	0.031	4.5	0.017	2.4
140	1.54	1.54	1.57	1.55	0.034	2.2	0.019	1.2
150	2.78	2.77	2.81	2.79	0.030	1.1	0.020	0.7
160	4.48	4.50	4.50	4.52	0.029	0.6	0.021	0.5

The accuracy of the equipment can also be determined by finding difference between the volume calculated from already verified equipment (Old set) data and volume calculated from new equipment (Set 1 and Set 2) data sets. The accuracy of the model is quantified with a linear regression. The linear regression plots for survey data set from Øvre Leirfoss and Selbusjøen are presented in Figure 2-36 and Figure 2-37, respectively. From each linear regression, R^2 value was determined. Both R^2 values were high (more than 0.99), indicating that the consistency of the equipment is very high in surveys carried out in Øvre Leirfoss and Selbusjøen.

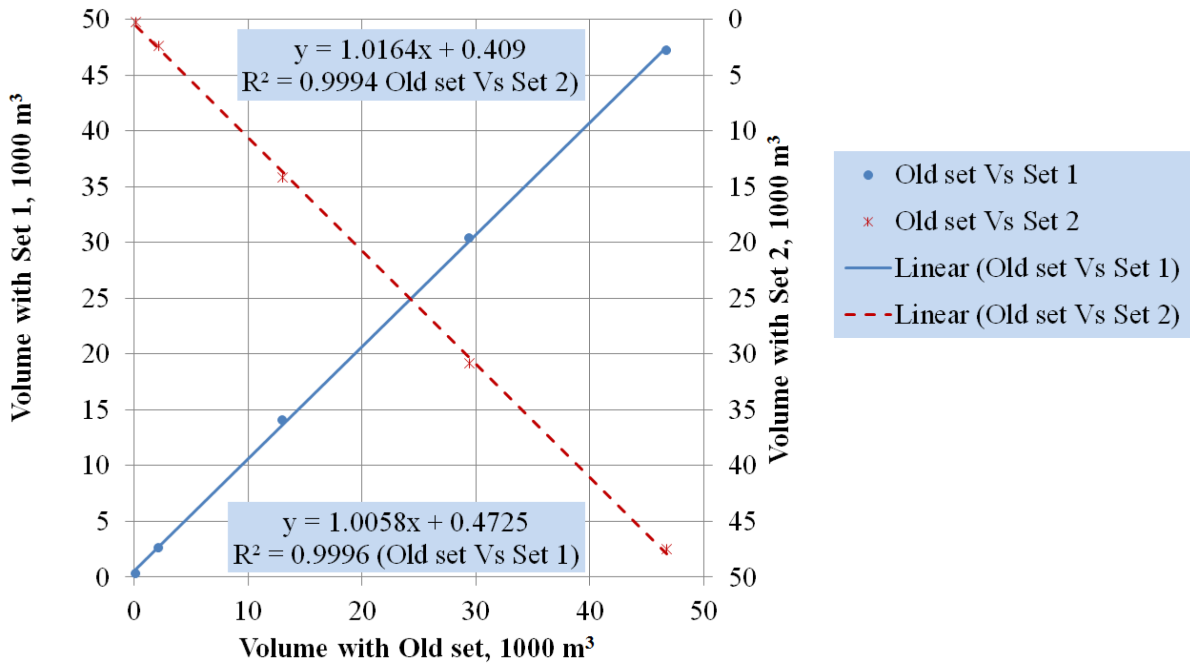


Figure 2-36: Linear regression plots for Øvre Leirfoss

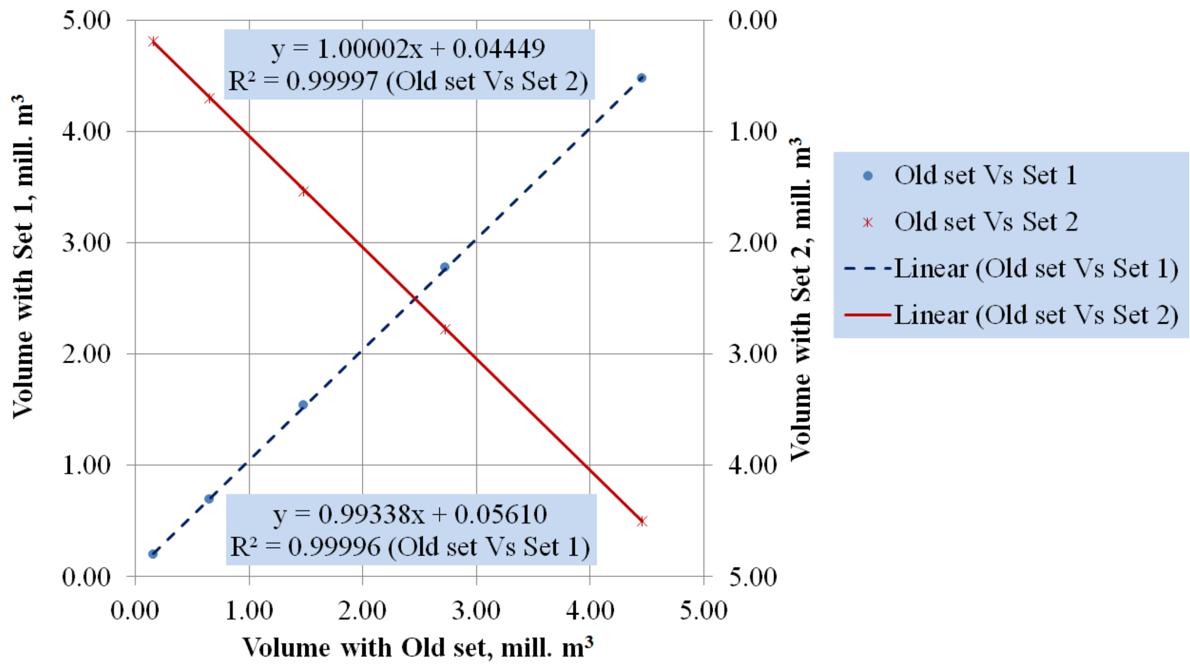


Figure 2-37: Linear regression plots for Selbusjøen

Kulekhani and Kaligandaki A reservoirs

The summary of the volume calculated from different surveys in Kulekhani reservoir are tabulated in Table 2-16. The average computed volume from Survey 1 and Survey 2 are 4.78 mill m^3 and 4.72 mill. m^3, respectively.

Table 2-16: Summary of calculated volume of middle part of Kulekhani reservoir with comparison

S. No.	Description of survey	Survey data set	Survey points per 10,000 m^2	Volume, (1,000 m^3)	Volume deviation (%)
1	Excluding centerline	Survey 1	67	4,526	2.20
		Survey 2	56	4,421	2.20
2	Excluding centerline and shore line	Survey 1	46	4,945	0.70
		Survey 2	38	4,910	0.70
3	Excluding shore line	Survey 1	72	4,982	0.95
		Survey 2	65	4,935	0.95
4	Combination of all	Survey 1	92	4,655	1.05
		Survey 2	82	4,607	1.05

Table 2-16 presents the summary of calculated volume of surveys in Kulekhani reservoir with comparison of each data set. It shows maximum difference of 2.20% for survey excluding centerline and 1.05% difference when all data are combined for each survey. Hence, the volumetric accuracy of the bathymetric survey can be increased by including centerline and shoreline surveys of the entire reservoir.

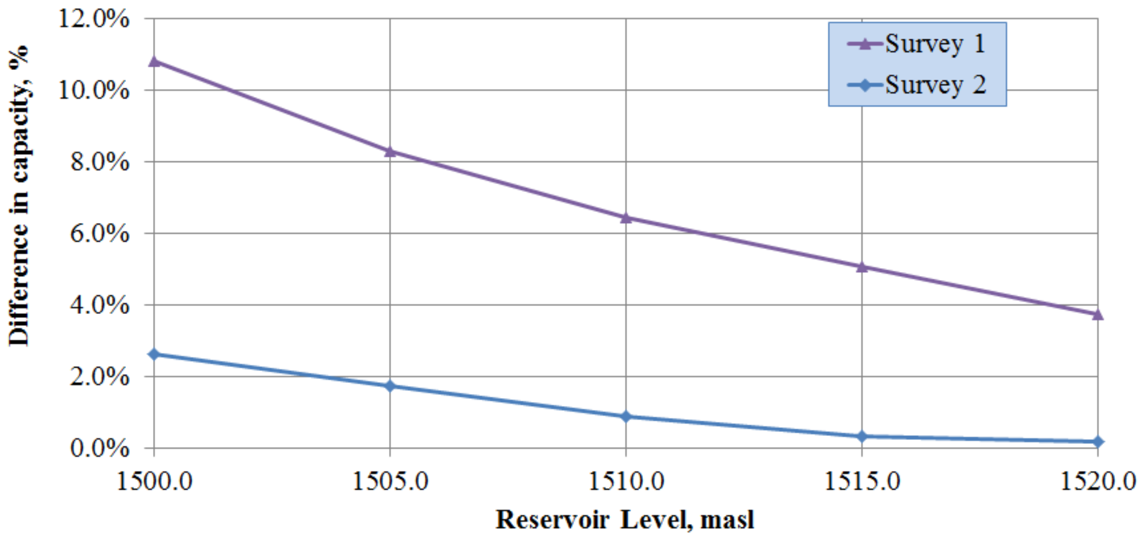


Figure 2-38: A relative percent difference of elevation-capacity curves for different survey point spacing in Kulekhani reservoir.

Further, the data are searched at distance of 10 m from Survey 1 and Survey 2 data set of Kulekhani. The volumetric difference between data set with survey point spacing of 5 m and 10 m is computed depending on the elevation. A relative percent difference for the elevation-capacity curves comparing this data set are shown in Figure 2-38 and tabulated in Table 2-17. The volume differences are 10.8% and 2.6% at elevation 1500 masl to 3.7% and 0.2% at elevation 1520 masl for Survey 1 and Survey 2, respectively.

Table 2-17: Reservoir capacity of Kulekhani reservoir for different survey point spacing

Point spacing	**5 m**		**10 m**		**Difference between 5 m and 10 m**			
Surveys	1	2	1	2	Survey 1		Survey 2	
Number of points	2,356	2,100	685	754	1,671		1,346	
Survey points per 10,000 m^2	93	82	42	42	51		40	
Elevation (masl)	**Volume (mill. m^3)**		**Volume (mill. m^3)**		**Volume (mill. m^3)**	**%**	**Volume (mill. m^3)**	**%**
1500	0.49	0.46	0.44	0.45	0.053	10.8	0.012	2.6
1505	1.25	1.19	1.14	1.17	0.103	8.3	0.020	1.7
1510	2.22	2.16	2.07	2.14	0.143	6.4	0.019	0.9
1515	3.36	3.31	3.19	3.29	0.171	5.1	0.011	0.3
1520	4.66	4.61	4.48	4.60	0.174	3.7	0.008	0.2

Table 2-18 presents the summary of calculated volume of survey data carried out in Kalgandaki A reservoir. It shows 2.8% volume is less for the Survey 1 than the volume calculated for the Survey 2.

Table 2-18: Calculated volume of Kaligandaki A reservoir for Survey 1 and Survey 2

S. No.	**Description of surveys**	**Survey points**	**Area, (1,000 m^2)**	**Survey points per 10,000 m^2**	**Volume, (1,000 m^3)**	**Volume deviation (%)**
1	Survey 1	1,344	135	99.30	937	2.73
2	Survey 2	663	134	49.45	963	2.81

Further, the data are searched at spacing of 10 m from Survey 1 and Survey 2 data set of Kaligandaki A reservoir. The summary and comparison of the results are given in Table 2-19.

Table 2-19: The comparison of results based on common data set with spacing at 5 m and 10 m in Kaligandaki A reservoir

S. No.	Description of surveys	Survey points	Area, (1,000 m^2)	Survey points per 10,000 m^2	Volume, (1,000 m^3)	Volume deviation (%)
1	Survey 1, 5 m	1,344	135	99	937	0.34
	Survey 1, 10 m	718	136	53	934	0.34
2	Survey 2, 5 m	663	134	49	963	0.12
	Survey 2, 10 m	419	134	31	962	0.12
3	Combination of all survey, 5 m	2,384	135	176	932	1.56
	Combination of all survey, 10 m	1137	135	84	947	1.56

Table 2-19 shows that calculated volume varies from 1.56% maximum for the combination of both to 0.12% minimum for Survey 2. Calculated volume with 5 m interval is lower than 10 m interval data set for Survey 1 and Survey 2, but it is reverse when all data is combined.

2.4.5 ACCURACY IMPROVEMENT

As discussed in Section 2.2.4, the software "Surfer 8" is used for data processing. Surfer is a grid-based graphic program. Elevations are interpolated from original data points to a regularly spaced grid. The interpolation is based on a global weighting scheme derived from the data variance. Nearest points are representative points for the interpolation. Figure 2-39 shows selection of points to estimate elevations of grid nodes by "Surfer 8".

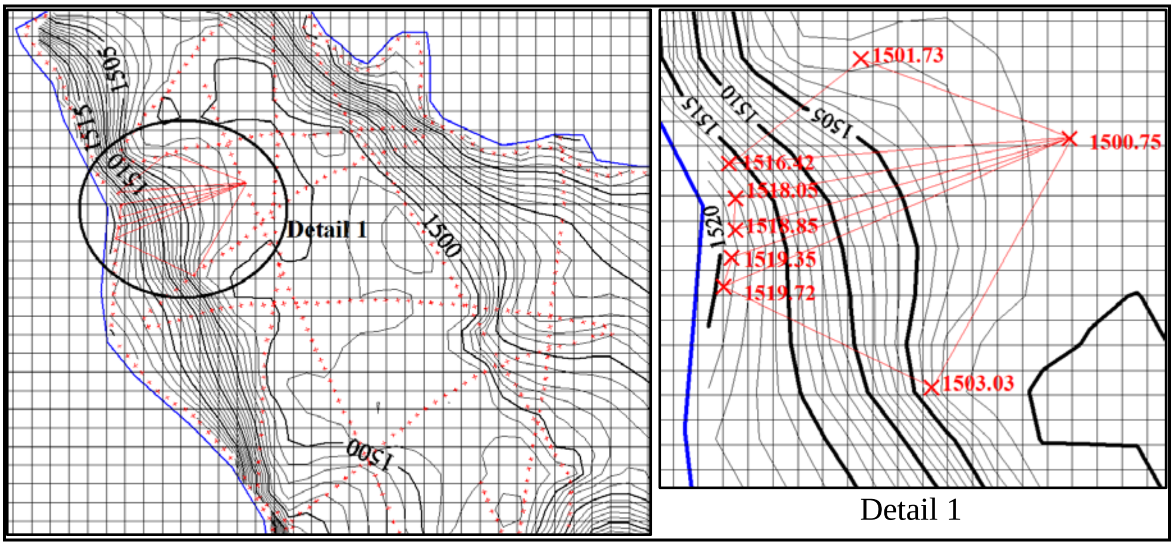


Figure 2-39: Selection of points to estimate the elevation (Z) by Surfer

Figure 2-40 shows bathymetric maps of Kulekhani reservoir excluding and including the longitudinal track lines based on Survey 2. A bulging of contours towards the deep area is presented when the survey points are spaced faraway. The bulging of the contours may be due to insufficient survey data between the representative points of the interpolation.

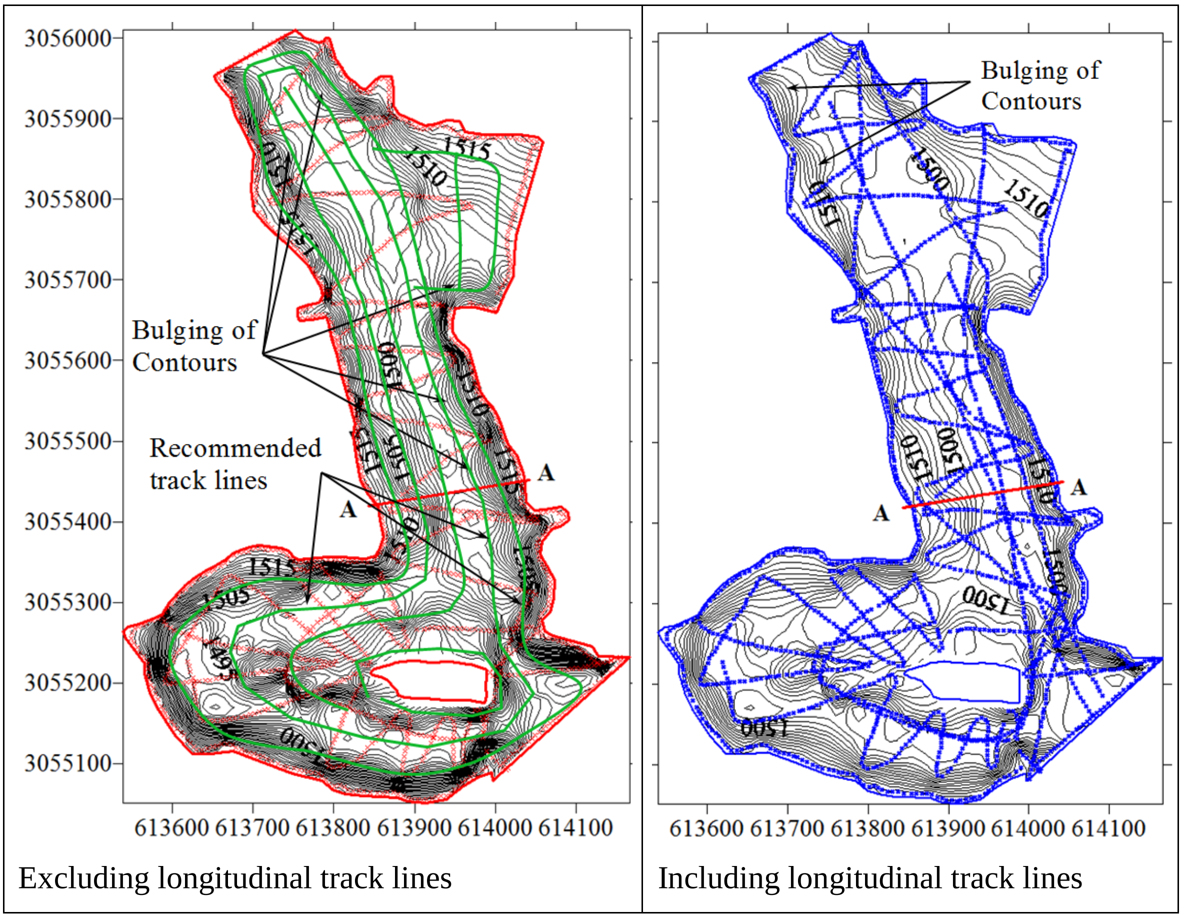


Figure 2-40: Bathymetric maps of Kulekhani reservoir excluding and including longitudinal track lines

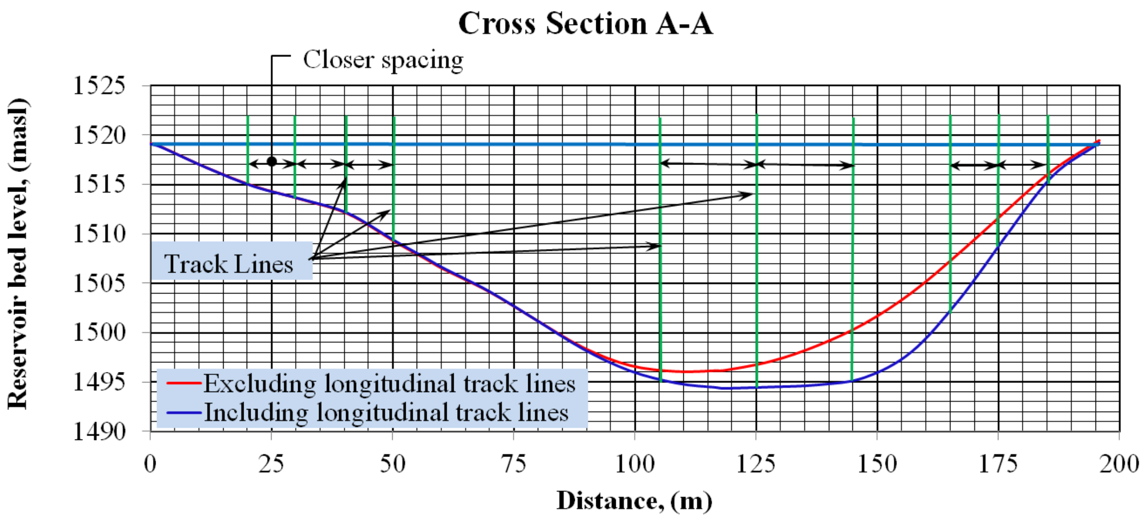


Figure 2-41: Cross section A-A in Kulekhani reservoir

Figure 2-41 shows the cross sections at A-A (see Figure 2-40 for location). The deviations in depth are higher (up to 5 m) at the left bank of the reservoir. Because, same survey points are used at the right bank but some additional points from the survey data of longitudinal survey are used to plot at the left bank of the reservoir. The computed volumes of Kulekhani reservoir excluding and including longitudinal track lines are 4.43

mill. m^3 and 4.61 mill. m^3, respectively. These deviations in depths and volumes may be due to the difference in track lines of the survey. To minimize bulging of contours and deviations in depth and volume, the accuracy of bathymetric survey can be improved adopting proper survey track lines and point density. For example:

- The survey track lines should be kept parallel to the shoreline
- Density of the survey points should be defined according to the depth gradient and irregularity of the terrain. The distance between two survey points along the track lines (a) and distance between track lines (b) should be smaller when the depth gradient and irregularity of terrain are high
- The distance between track lines should be closer in the shoreline area with high depth gradient than the middle part of the reservoir as shown in Figure 2-41

2.4.6 FREQUENCY OF SURVEYS

Each reservoir contains unique conditions that must be considered when determining the timing and frequency of the reservoir resurveys. The decision on the schedule and the frequency to conduct a reservoir survey must be made on a case-by-case basis. The frequency of surveys in reservoirs depends on several factors; the main ones are:

- The total capacity of the reservoir
- Average depth of the reservoir
- Specific sediment yield
- The precision of the survey technique
- The rate and pattern of storage loss
- The area of the reservoir

At reservoirs losing capacity very slowly, a survey interval on the order of 20 years or even longer may be adequate. On the other hand, at important sites that are losing capacity rapidly, or where the impact of sediment management is being evaluated, a survey interval as short as 2 or 3 years might be used (Morris and Fan, 1998).

The effect of the specific sediment yield and sediment deposit depth on the frequency of survey with respect to the accuracy of DGPS method is assessed in this study.

The capacity reduction rate of a reservoir is assessed with specific sediment yield of 5,000 to 12,500 m^3/year/km^2 at the interval of 2,500 m^3/year/km^2 and shown in Figure 2-42. Figure 2-42 shows that the reservoir capacity is reduced from 30% to 32% within one and three years with a specific sediment yield of 5,000 and 12,500 m^3/year/km^2, respectively.

If the sediment yield is distributed equally in the entire reservoir area, the average annual depth of sediment deposit (m/year) can be obtained from sediment yield (m^3/year) dividing by the reservoir area (m^2). The rate of sediment deposition depth of the reservoir is also assessed by varying the specific sediment yield (Figure 2-43). Figure 2-43 shows that in two years of reservoir operation, the sediment will be deposited by 0.58 m and 1.44 m for the specific sediment yield of 5,000 m^3/year/km^2 and 12,500 m^3/year/km^2, respectively.

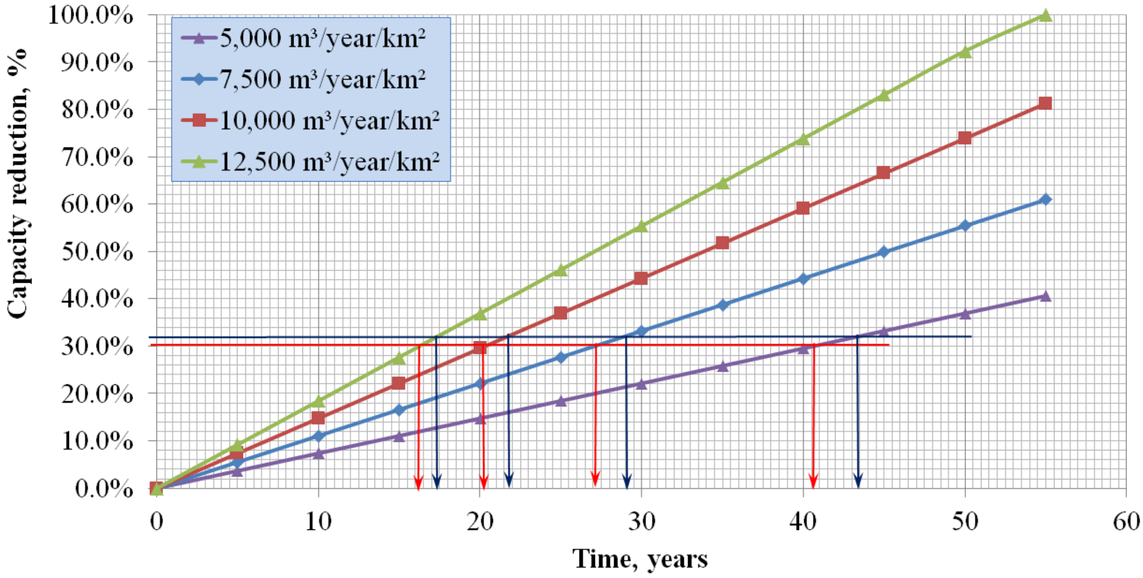


Figure 2-42: Capacity reduction rate with respect to various specific sediment yield

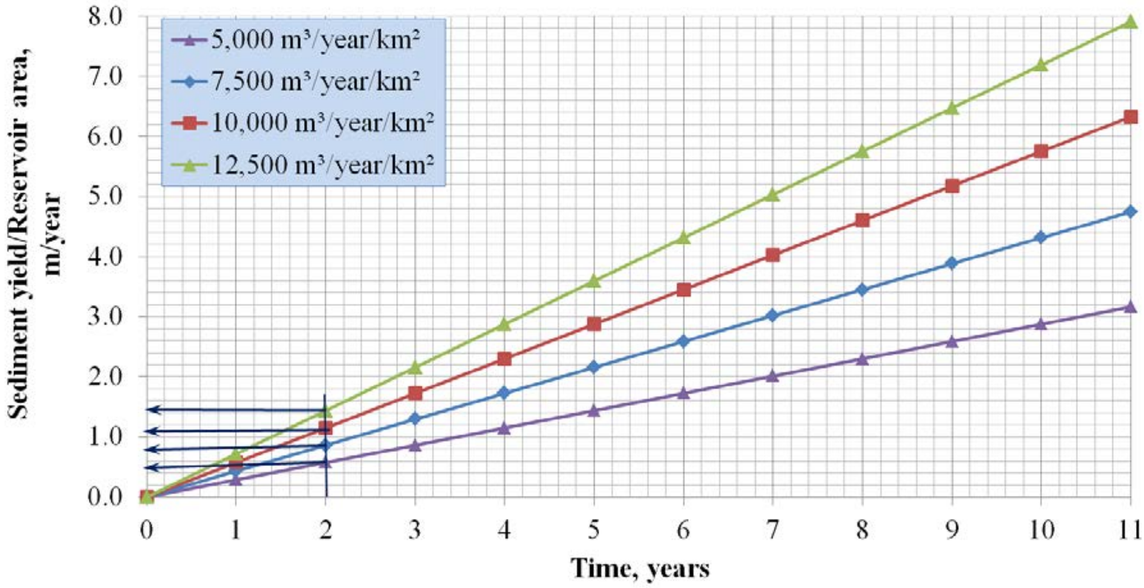


Figure 2-43: The rate of sediment deposition depth with respect to various specific sediment yield

The time required for a sediment deposition depth of 1.5 m in the reservoir is assessed by varying reservoir area and specific sediment yield. As discussed, the bathymetric survey with DGPS incorporates the error in depth up to 0.5 m (Section 2.4.3). Therefore, 1.5 m is considered as a minimum depth required between two consecutive bathymetric surveys. Figure 2-44 shows time required between two consecutive bathymetric surveys with DGPS method.

Figure 2-44 shows that more frequent surveys should be carried out for large catchment area (Ac) with small reservoir area (Ar). If the catchment area and reservoir area ratio (Ac/Ar) is greater than 240, the bathymetric survey can be carried out every year for a specific sediment yield exceeding 6,000 m^3/year/km^2. The frequency of the bathymetric survey can be determined with the help of Figure 2-44, which is based on specific sediment yield and catchment and reservoir area ratio (Ac/Ar). For example, the bathymetric survey of a reservoir with specific sediment yield of 6,000 m^3/year/km^2 and with catchment and reservoir area ratio (Ac/Ar) of 60 can be carried out in every four years.

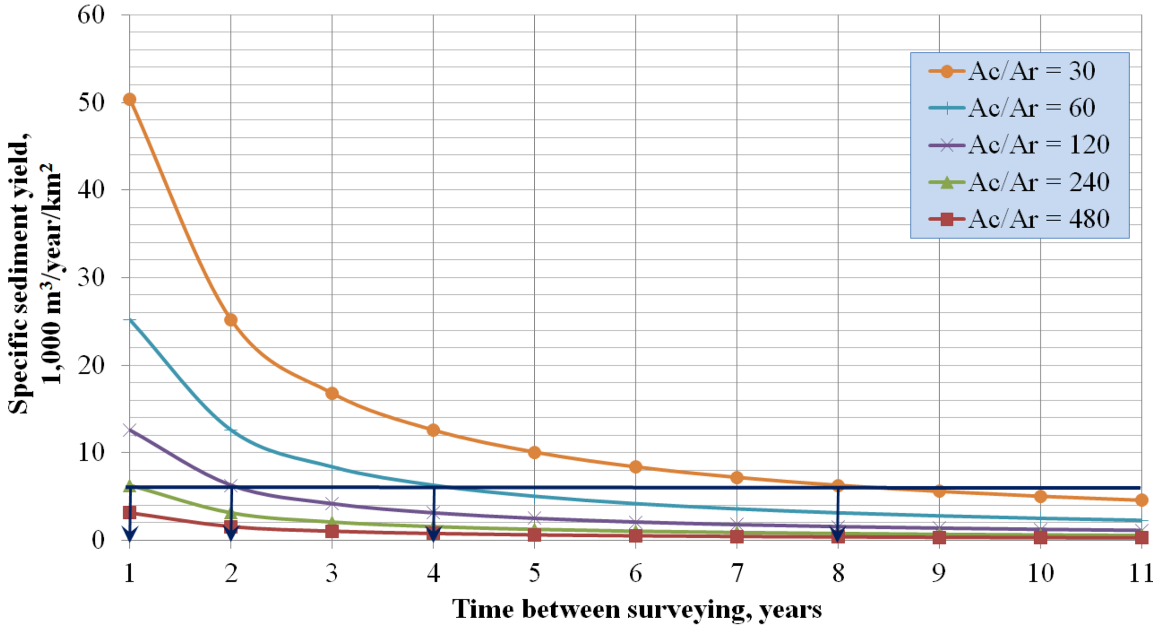


Figure 2-44: Frequency of bathymetric survey with respect to specific sediment yield and catchment area (Ac) and reservoir area (Ar) ratio (Ac/Ar)

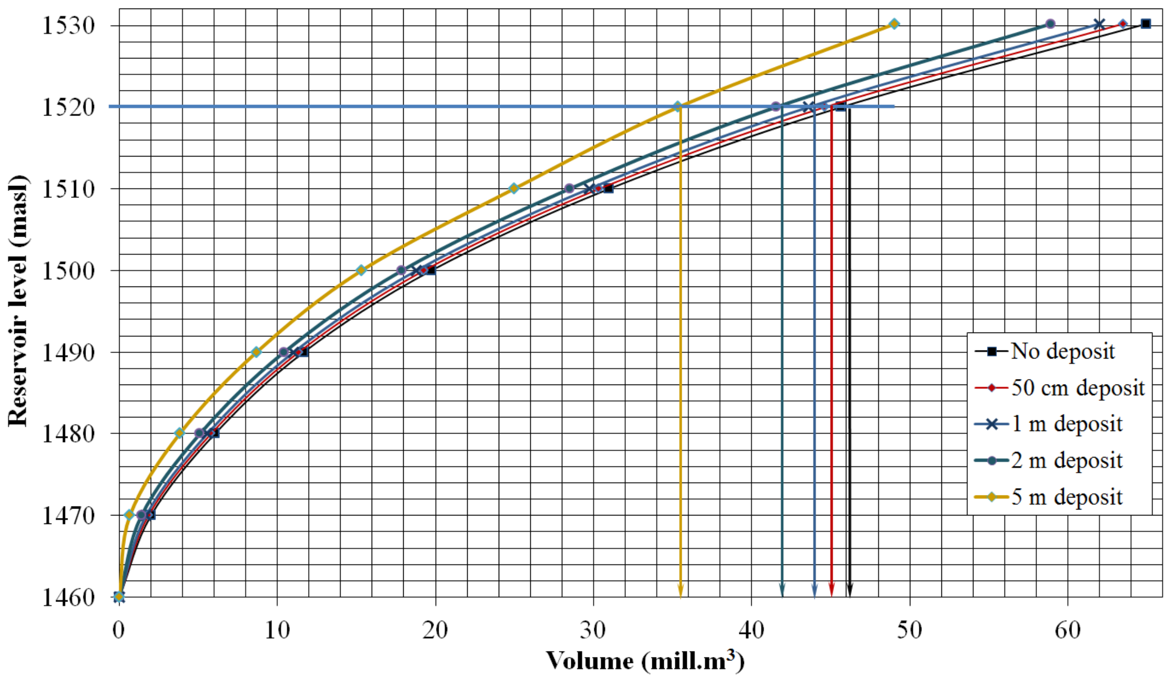


Figure 2-45: Capacity curves for various thickness of the sediment deposits

Further, the capacity reduction with respect to sediment deposition thickness is calculated using the Surfer programme for various thickness of sediment deposit. The capacity curves for various thickness of the sediment deposit are shown in Figure 2-45.

Figure 2-45 shows that the capacity of the reservoir is reduced by 1.03 mill. m^3, 2.06 mill. m^3, 4.12 mill. m^3 and 10.3 mill. m^3 for sediment deposit thickness of 50 cm, 1 m, 2 m and 5 m, respectively. The capacity reduction is directly proportional to the sediment deposit thickness.

Sedimentation rate is computed as the difference between volume measurements from two surveys. The minimum survey interval depends on the accuracy of the survey technique, the rate and thickness of the sediment deposition. The capacity reduction rate and sediment deposition depth should be assessed according to the reservoir topography and sediment yield before determining minimum survey interval. For instance, bathymetric survey with DGPS technique incorporates a volumetric error of about 2 percent of the total reservoir volume and the reservoir is losing capacity at two percent in three years for 5,000 m^3/km^2/year (Figure 2-42), a 3-year survey interval may be too short to produce reliable information unless most sediment inflow is focused into a small portion of the impoundment. Similarly, the survey incorporates the error in depth from 0.02 m to 0.5 m (Section 2.4.3) depending on the maximum reservoir depth, survey track lines and point density. The thickness of the sediment deposit is only 0.43 m in two years if the reservoir area is reduced by 50% (Figure 2-44). Hence a 2-year survey interval may be too short to produce reliable information.

The schedule and frequency of conducting reservoir surveys should be assessed depending on the accuracy of the used survey technique, the capacity reduction rate and the sediment deposition thickness with respect to sediment yield.

2.5 CONCLUSION AND RECOMMENDATION

The average error in depth measurement varied from 0.02 m in Kulekhani reservoir to 0.49 m in Selbusjøen. The study shows that the precision of the depth measurement decreases with increment of the water depth. However, the accuracy of depth measurement by bathymetric survey with DGPS method can be increased with increment of the survey point density with coverage of all area of the reservoir.

The results of the survey conducted in Øvre Leirfoss tailrace pond and outlet area of the Selbusjøen show the difference of 0.9% in volume surveyed with Old set and new sets (Set 1 and Set 2). The linear regression plots of the volume with Old set versus new sets show high R^2 values (more than 0.999). The difference of the calculated volume is due to the instrumental error, terrain irregularities and data density of the survey.

The bathymetric survey conducted in Kulekhani and Kaligandaki A reservoirs show that the survey conducted including shoreline, centerline and cross section survey gives less difference (less than 1%) in volume computation than the survey excluding centerline and shoreline (up to 5%). Therefore, the survey should include shoreline, centerline and cross section survey to minimize the error in volume computation as recommended in Section 2.4.5.

The main factors that influence the accuracy of survey data or volume computations from the bathymetric survey are terrain irregularity, data density and track lines of the survey. Terrain irregularity has major impact on the accuracy of volume computations. These effects can be minimized by increasing survey point data density. Other factors may affect accuracy of the survey data. These factors may include errors due to fluff, vegetation, velocity variations or sensitivity drift. These errors are difficult to quantify.

Along with the accuracy of survey method, the schedule and frequency of conducting reservoir surveys should depend on the estimated rate of reservoir sediment accumulation. The results of survey carried out in the reservoirs of this study shows that the sediment yield, the capacity reduction rate and sediment deposition thickness should be assessed with respect to the accuracy of the survey technique to determine schedule and frequency of bathymetric survey of the reservoir as recommended in Section 2.4.6.

3 BASELINE SURVEY FOR SEDIMENTATION MONITORING IN KALIGANDAKI AND MIDDLE MARSYANGDI RESERVOIRS

3.1 INTRODUCTION

In general, repeated reservoir surveys in large reservoirs are used to determine the total storage capacity of a reservoir, the capacity-elevation curves, the sediment deposition rate and the sediment yield from the watershed. However, in daily peaking reservoirs which are designed to regulate only during the peak hours of a day, the reservoir surveys are conducted to determine the sediment deposition pattern, amount of flushed sediment after flushing, the shift in the sediment deposition area and capacity-elevation curves.

The reservoir-operating rule of a peaking reservoir has a large influence on the sediment accumulation pattern within the reservoir. For instance, if the reservoir is operated at Highest Regulated Water Level (HRWL) during the monsoon season, sediment deposition occurs at a high pool level within the live storage. In contrast, if the reservoir is operated at Lowest Regulated Water Level (LRWL) during the monsoon season, sediment will deposit in the dead storage or be transported through the reservoir to the river downstream of the dam. Furthermore, the deposited sediment within the live storage can be scoured and re-deposited to the dead storage. Therefore, the daily peaking reservoirs should be operated at LRWL during flood time and monsoon to maintain the regulation capacity of the reservoir (Lysne et al, 2003).

The Himalayan rivers are characterized by high sediment loads, which make the reservoir to be filled by sediment deposition very fast. TAHAL consultancy in association with GEOCE, ARMS and CEMAT conducted sediment study in major river basins of Nepal for Nepal Irrigation Sector Project (NISP) in 2000 and 2001 (TAHAL *et al.,* 2002). The sediment data was collected from 15 river basins. The monthly sediment yield of the sediment data collected from stations of these river basins by the project shows that more than 95% of the annual sediment load is transported during the monsoon season from June to October. During the dry season from November to May, the flow in the river is smaller with less sediment load. Similarly, a sediment study conducted by Sangroula (2005) in Palung Khola and Chitlang Khola shows that more than 70% of the annual sediment load is transported during the floods that occur about 10 to 15 times in a year during the monsoon period. It means if the daily peaking reservoir is not operated properly during flood time, sediment can be deposited within the live storage of the reservoir and in the intake area. This high sediment load not only causes the sediment erosion on hydro-mechanical and civil parts of the hydropower plant, but it reduces the reservoir capacity. Some of the examples are given below.

Chorten (2010) presented significant energy generation loss during monsoon months (i.e. June to October) in Chhukha hydropower plant, Bhutan due to deficiency in headworks performance relating to the sedimentation in the intake area. Figure 3-1 shows the sediment deposition in front of the Chhukha hydropower plant, Bhutan.

Figure 3-1: Sediment deposition in intake area of Chhukha HP (Chorten, 2010)

According to Kayastha (2010), operation of Middle Marsyangdi reservoir at Highest Regulated Water Level (HRWL) during June-July 2009 has resulted in high sedimentation in the live storage of the reservoir. Sediment deposition in Middle Marsyangdi HEP reservoir after the monsoon 2009 is shown in Figure 3-2.

Figure 3-2: Sediment deposition in Middle Marsyangdi reservoir (Kayastha, 2010)

Figure 3-3 shows the sediment deposition in the reservoir and in the Forebay of the Kaligandaki A hydropower plant.

Figure 3-3: Sediment deposition in reservoir and forebay of Kaligandaki A reservoir during the flushing.

The above information indicates that sedimentation is one of the major challenges for sustainable hydropower development in the Himalayan region no matter whether it is Peaking Run of River (PRoR) or storage hydropower projects.

The value of the peaking hour energy is very high in Nepal where people are facing more than 16 hours load shedding in a day. Furthermore, off peak load demand is about 45% lower than the peak load demand and total peak load is only 7 hours a day, about 2.5 hours in the morning and about 4.5 hours in the evening. Therefore, it is important to monitor the sedimentation process of the peaking reservoirs in order to know more about the status of the reservoirs for proper handling of the sediment.

Kaligandaki A hydropower plant is the biggest hydropower plant in Nepal with 144 MW installed capacity. Middle Marsyangdi hydropower plant was commissioned in December 2008 and the live storage of the reservoir was partly filled with sediment deposit due to the reservoir operation at HWRL during June-July 2009. At present, Kaligandaki A and Middle Marsyangdi hydropower plants supply about one third of total hydropower capacity of Nepal. The location of these hydropower plants are shown in Figure 5-1. The sustainability of these reservoirs is a vital issue in the energy sector of Nepal. Therefore, bathymetric survey in these reservoirs was conducted in December 2010 in order to prepare base line map of the reservoir to monitor the sedimentation process in these reservoirs.

3.2 KALIGANDAKI A HYDROPOWER PLANT

The Kaligandaki A hydropower plant is approximately 180 km west of Kathmandu. The main component of the project is located at Syangja District in Gandaki Zone and partially encompasses other districts such as Palpa, Parbat, Gulmi, Kaski and Rupandehi. The diversion dam is about 500 meters downstream from the confluence of Kaligandaki river and Andhi Khola and in between Mirmi and Harmichaur Village. The main components of the project comprises of a concrete gravity diversion dam of about 100 meters in length and 44 m of height, open surface settling basin, tunnel of about 6 kilometer length and 7.4 m diameter and a surface powerhouse. The project uses water from Kaligandaki at the dam site and conveys it to the powerhouse at Manuwa, Beltari Village through the tunnel utilizing a net head of 115 m and rated discharge of 134 m^3/s. The project generated power since 16 August 2002.

As per the detail design, the reservoir at maximum level stores a total volume of 7.7 mill. m^3 and a daily storage of 3.1 mill. m^3 with an area of 650,000 m^2 (65 ha) at maximum operating level. Water volume between 518 m and 524 m is utilized to run two turbines for 6 hours during peak time in the dry season. The map of the reservoir area with survey track lines is shown in Figure 3-4. The length of the reservoir is about 6 km along the Kaligandaki river and about 2 km along the Andhi Khola river. The average width is about 50 m.

The daily peaking reservoir maintained its regulation capacity through the following measures:

- Head water level is maintained at LRWL of 518 masl throughout the monsoon season to prevent deposition in the live storage.
- Reservoir flushing as shown in Figure 3-3.

3.3 MIDDLE MARSYANGDI HYDROPOWER PLANT

Middle Marsyangdi hydroelectric project is located on Marsyangdi river in the mid hills of central Nepal, about 40 km upstream of the old Marsyangdi hydropower plant. The project site is about 170 km west of Kathmandu. Middle Marsyangdi Hydroelectric Project (MMHEP) is a run-of-river type scheme with daily poundage for five hours. The plant has been designed for an installed capacity of 70 MW in order to generate 398 GWh in average annually. The main components of the project consist of a 62 m high, 95 m long combined concrete and rockfill gravity dam with a gated concrete spillway, a 1.6 million m^3 capacity peaking reservoir, three numbers of underground settling basin caverns (15m x 100 m x 25.1 m each) with two chambers in each cavern, 5.4 m diameter 5,210 m long concrete lined power tunnel, 20 m diameter 45 m high surge tank, a 450 m long penstock, a surface powerhouse. The project generated the energy since 14 December 2008.

As per the design, the reservoir at maximum level stores a total volume of 5.6 million m^3 with a live storage of 1.5 million m^3. The water volume between elevations 621 masl and 626 masl is utilized to run two turbines for 6 hours during peak time in the dry season. The map of the reservoir area with survey track lines is shown in Figure 3-5. The length of the reservoir is about 2.8 km with maximum width of 230 m just upstream of Dam. The upstream part of the reservoir is about 70 m wide.

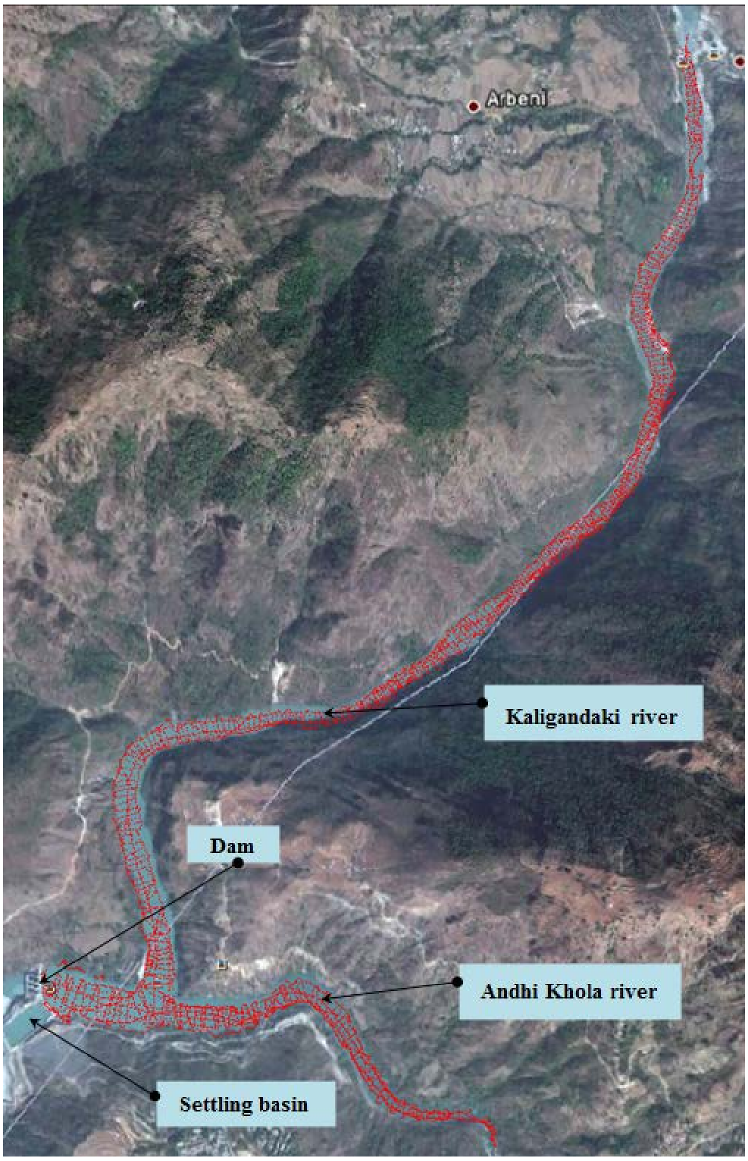


Figure 3-4: Kaligandaki A HP reservoir (from Google Earth 2012)

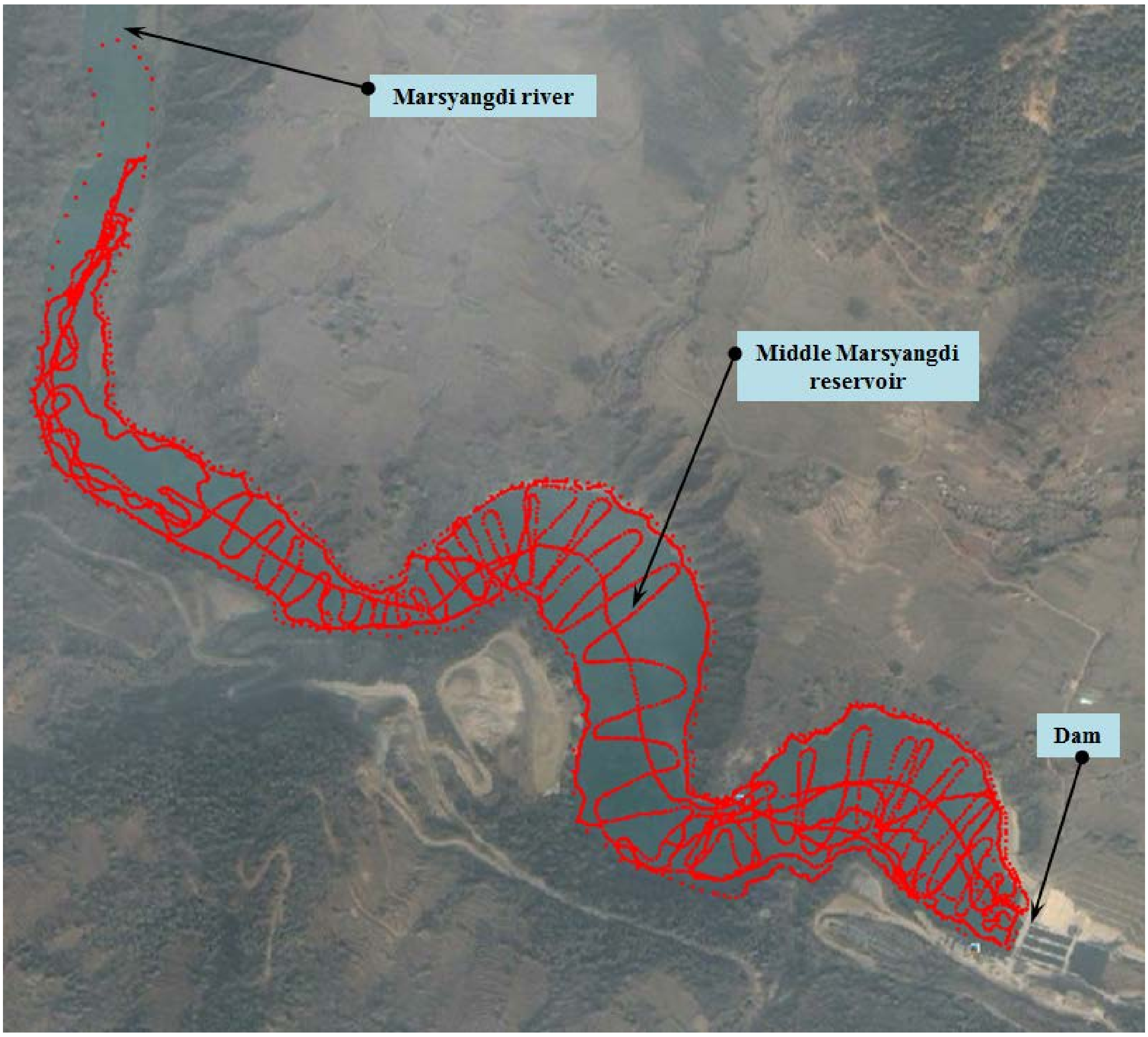


Figure 3-5: Middle Marsyangdi HP reservoir (from Google Earth 2011)

3.1 FIELD SURVEY

The bathymetric survey in Kaligandaki A and Middle Marsyangdi reservoirs were carried out using Differential Global Positioning System (DGPS). The same survey equipment and procedure were adopted as explained in Chapter 2. Data logging interval was set to 5 m and the reservoir was surveyed along both banks, the longitudinal as well as cross sections on both reservoirs. Figure 3-4 and Figure 3-5 present the survey track lines in Kaligandaki A and Middle Marsyangdi reservoirs, respectively.

3.1.1 Kaligandaki A Reservoir

The bathymetric survey of Kaligandaki A hydropower plant reservoir was carried out from 14th to 17th of December 2010 by the author. Professor Durga Sangroula and other members from Hydro Lab and NEA were also involved in this survey. Survey was carried out by DGPS system from a motor boat. Figure 3-6 shows the survey team with the motor boat.

Previously established benchmark “JG18” by NEA was adopted for the setup of the base station for the downstream part of the reservoir and Andhi Khola area survey, which was located at the hill slope of the confluence of Kaligandaki and Andhi Khola rivers. Coordinates of JG 18 are 3095685.195N, 459529.13E with elevation of 593.398 masl. Due to the high and steep hills (gorge) on both banks of Kali Gandaki river, the base station at JG18 was not sufficient for the survey of the upstream part of the Kali Gandaki A reservoir. Therefore, two other benchmarks (HL1 and HL2) were established.

HL1 was located about 200 meters downstream of the suspension bridge of Seti Beni on the left bank of the reservoir, below the road track. Similarly, HL2 is located on a boulder about 800 m downstream of the suspension bridge of Seti Beni on the left bank of the reservoir at the cultivated land. HL2 was adopted as a second base station for the reservoir survey for the remaining upstream portion of Kaligandaki A reservoir. Coordinates of HL 2 are 3098074.133N, 461290.889E with elevation of 534.348 masl. Base stations JG18 and HL2 are shown in Figure 3-7.

Figure 3-6: Survey team with the rover (Garmin Etrex) and Fish Finder

Figure 3-7: Base stations JG18 and HL2

Altogether 6,857 data points were recorded during the whole survey. The survey track lines in the Kaligandaki A reservoir are shown in Figure 3-4. Water level of the reservoir records during the survey varied from 520.92 to 521.94 meter above sea level (masl). The recorded water levels are used to compute the bottom elevations of the reservoir as described in Section 2.2.4.

3.1.2 MIDDLE MARSYANGDI RESERVOIR

The bathymetric survey of Middle Marsyangdi hydropower plant reservoir was also carried out by the author from 19th to 21st of December 2010. Staffs from Hydro Lab and NEA were also involved in this survey.

Previously established benchmark "PZ-2L" by NEA was used for the setup of base station for the survey, which was located at left bank of the Dam. Coordinates of PZ-2L are 3118928.132N, 541452.119E with elevation of 634.2 masl. As the reservoir length was only 3 kms and only small hills around the survey area, PZ-2L was sufficient for the whole reservoir survey area. The base station PZ-2L is shown in Figure 3-8.

Figure 3-8: Base stations PZ-2L

A motor boat (Figure 3-9) with capacity of 1,500 kg was used for the survey. The motor of the boat did not work on 19th and 20th December. Small manual boats were not available in this area, so the boat was rowed manually on that days. The boat was quite big to row manually, needed four persons to row it. It was repaired on 21st December and the final survey was carried out by the motor boat. Altogether 5,046 data points were recorded during the whole survey. The survey track lines in the Middle Marsyangdi reservoir are shown in Figure 3-5. Water level records during the survey were varied from

624.14 to 624.8 masl. The recorded water levels are used to compute the bottom elevations of the reservoir as described in Section 2.2.4.

Figure 3-9: Motor boat used during the survey

3.2 DATA PROCESSING

The survey data was processed as explained in chapter 2.

Attempts were made to track the shoreline boundary of the reservoir during the survey of the reservoir, but because of the reservoir low level as well as the bushes, big boulders and trees close to the shoreline, it was impossible to track the shoreline boundary completely. Hence, the shoreline boundary of the Kaligandaki A reservoir was generated based on the bathymetric survey carried out in 2000. These data was also used to generate contours in between 521.5 to 524.0 masl and to calculate area and capacity of the reservoir in 2000. Similarly, topographic map of the Middle Marsyangdi reservoir of the detail design phase is used to calculate the area and volume of the reservoir and to generate contours in between 524.5 to 526.0 masl.

3.3 RESULTS AND DISCUSSION

The bathymetric surveys of both reservoirs were analysed, maps prepared and storage capacities for both reservoir were calculated.

The bathymetric maps of the Kaligandaki A and Middle Marsyangdi reservoirs are shown in Figure 3-10 and Figure 3-12.

3.3.1 KALIGANDAKI A

The total storage capacity of the Kaligandaki A reservoir at elevation 524 masl is 7.7 and 3.77 mill. m^3 in 2000 and December 2010, respectively. The surface area at the same level is 0.65 km^2. The surface area-elevation and capacity-elevation curves of Kaligandaki A reservoir are shown in Figure 3-11.

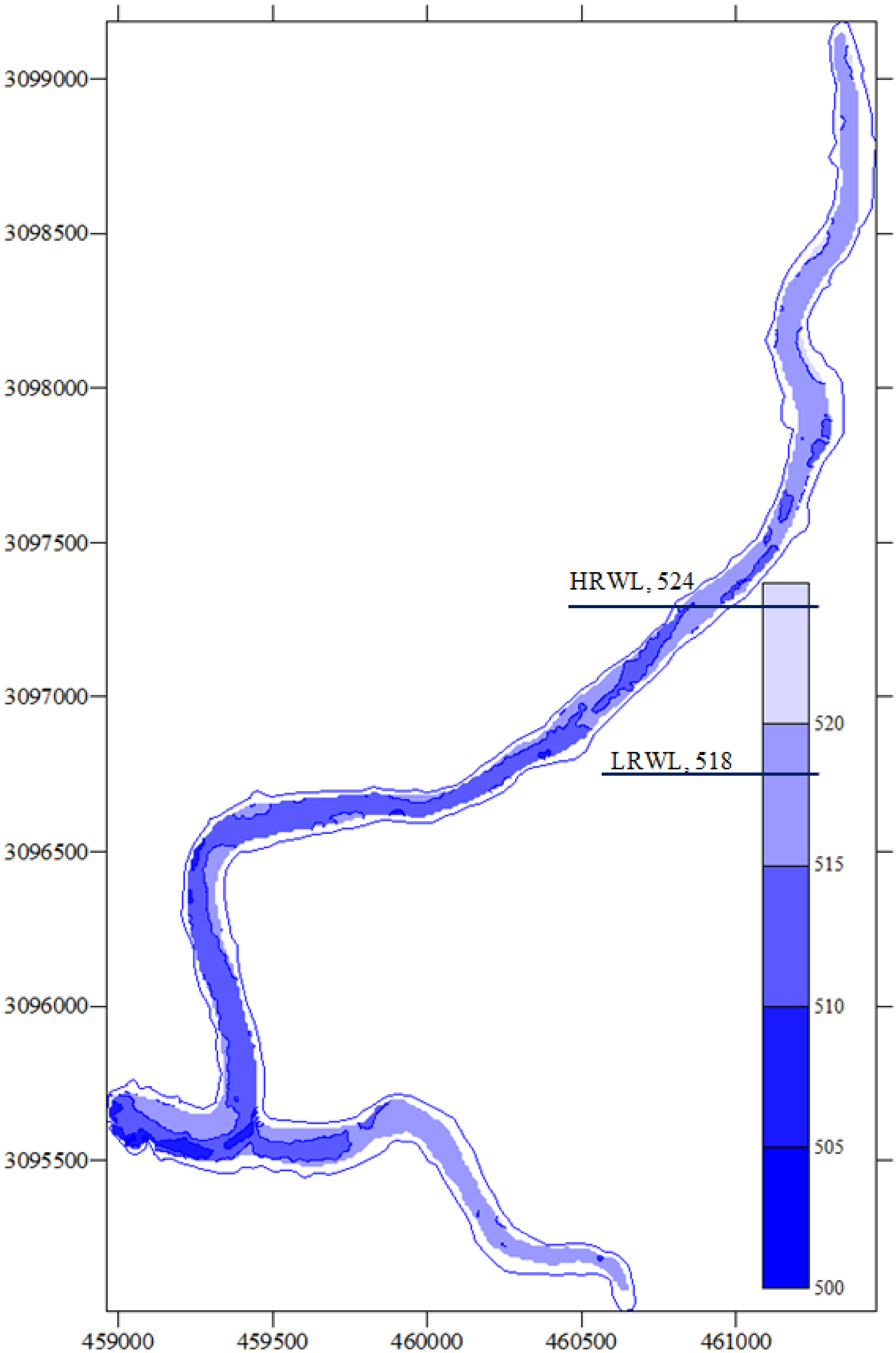


Figure 3-10: Bathymetric map of Kaligandaki A reservoir

Table 3-1 shows the summary of computed surface area and reservoir volume of Kaligandaki A reservoir for bathymetric survey 2000 (original) and 2010.

Table 3-1: Reservoir area and capacity of Kaligandaki A reservoir

Year	2000		2010	
Elevation (masl)	**Area (1,000 m^2)**	**Volume (mill. m^3)**	**Area (1,000 m^2)**	**Volume (mill. m^3)**
508	153.29	0.73	0.88	0.00
510	190.85	1.13	6.50	0.01
512	238.07	1.64	19.42	0.03
514	293.41	2.26	63.63	0.11
516	360.16	3.03	148.07	0.33
518	418.89	3.95	272.81	0.77
520	476.15	5.01	399.67	1.50
522	558.08	6.24	516.51	2.50
524	650.31	7.70	650.31	3.77

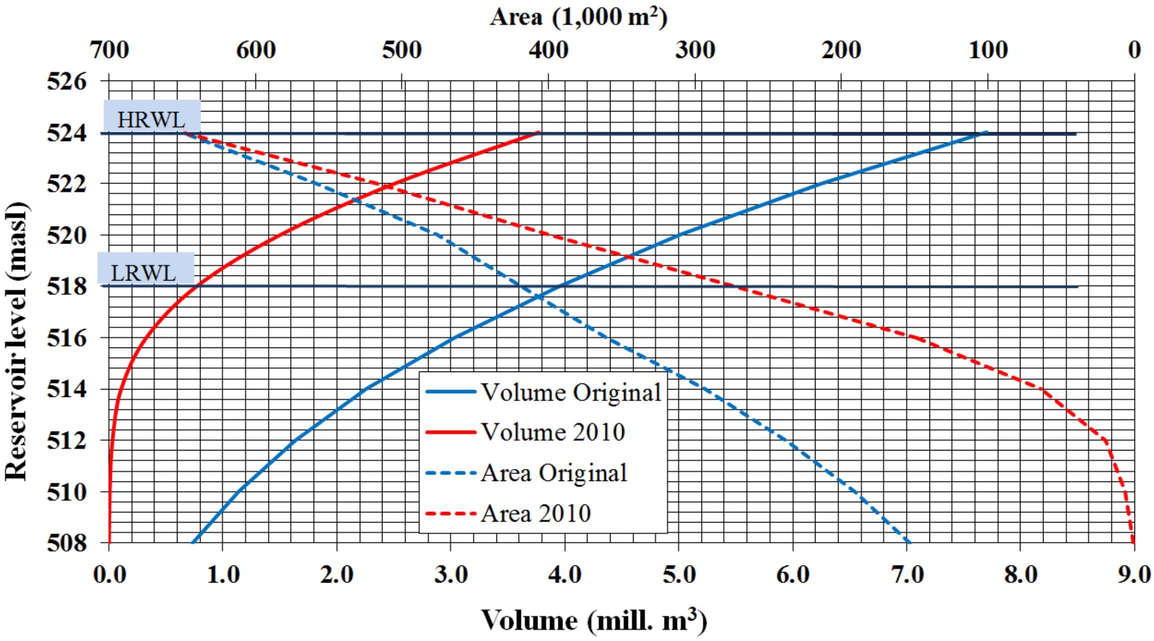


Figure 3-11: Area-elevation and capacity-elevation curves of Kaligandaki A reservoir

The Kaligandaki A reservoir has lost about 4 mill. m^3 in 10 years of operation. It is about 51% of the original capacity of the reservoir.

3.3.2 MIDDLE MARSYANGDI

Based on the bathymetric survey in December 2010, the total storage capacity of Middle Marsyangdi reservoir at 526.0 masl is 1.94 mill. m^3 and the surface area at the same level is about 0.40 km^2. The surface area-elevation and the capacity-elevation curves of Middle Marsyangdi reservoir are shown in Figure 3-13.

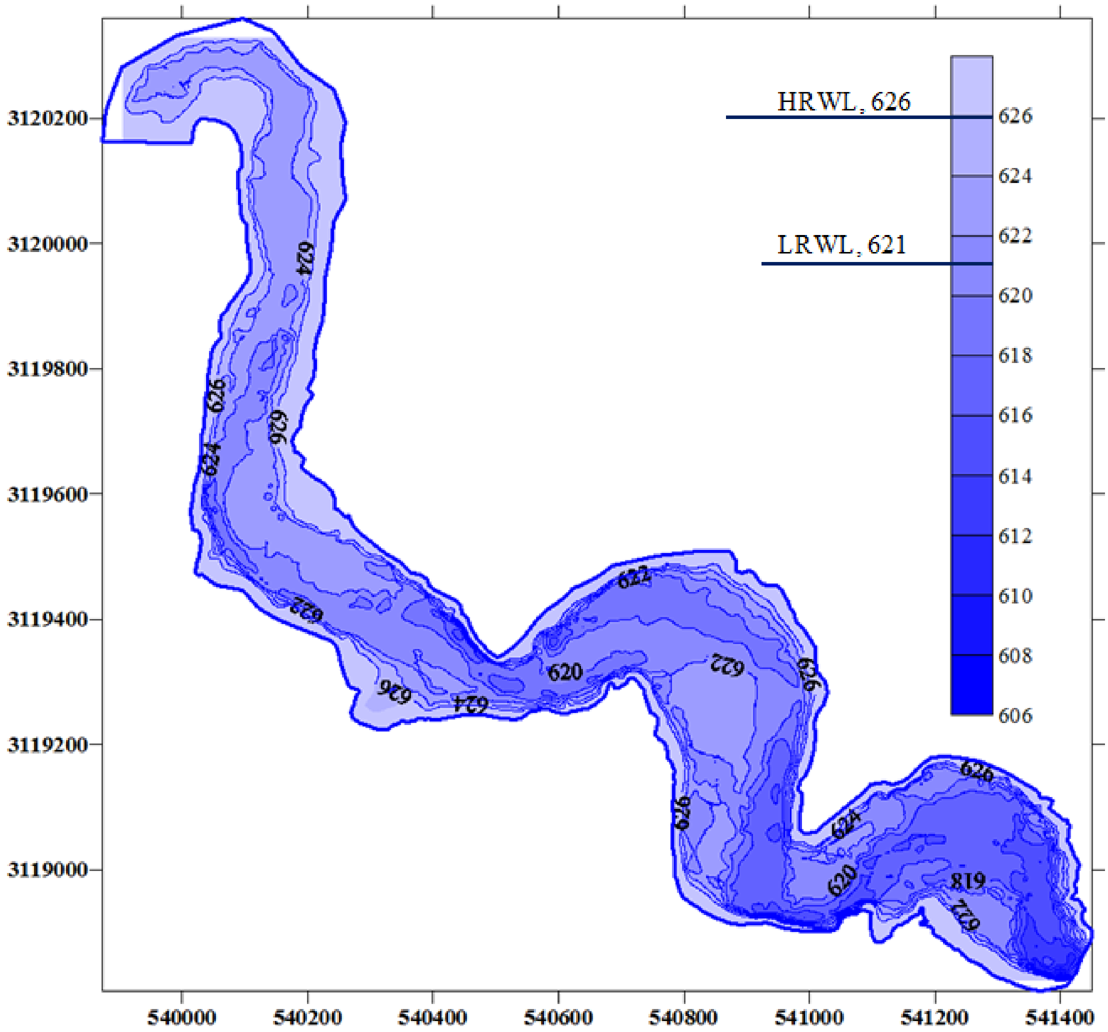


Figure 3-12: Bathymetric map of Middle Marsyangdi reservoir

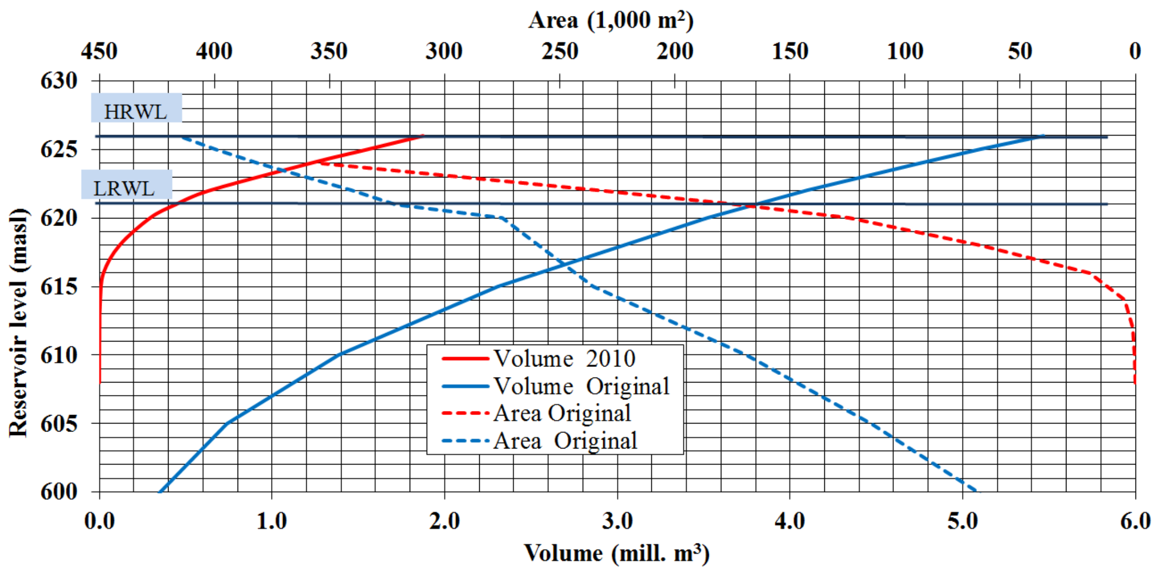


Figure 3-13: Area-elevation and capacity-elevation curves of Middle Marsyangdi reservoir

Table 3-2 shows the summary of computed surface area and reservoir volume of Middle Marsyangdi reservoir for topographic map 2006 (original) and bathymetric survey 2010.

Table 3-2: Reservoir area and capacity of Middle Marsyangdi reservoir

Original, 2006 (Generated from topographic map)			**2010 Bathymetric Survey**		
Elevation (masl)	**Area (1,000 m^2)**	**Volume (mill. m^3)**	**Elevation (masl)**	**Area (1,000 m^2)**	**Volume (mill. m^3)**
590	7	0.03			
595	38	0.10	610	0.24	0.00
600	68	o.35	612	1.03	0.00
605	114	0.74	614	4.78	0.01
610	168	1.39	616	19.61	0.02
615	235	2.31	618	67.05	0.12
620	274	3.52	620	124.23	0.29
621	321	3.80	621	173.29	0.44
622	340	4.10	622	233.68	0.63
624	380	4.75	624	253.21	1.22
625	399	5.10	625		
626	415	5.47	626		

Figure 3-14: Sediment deposition pattern upstream of the intake October 2009 (Kayastha, 2010)

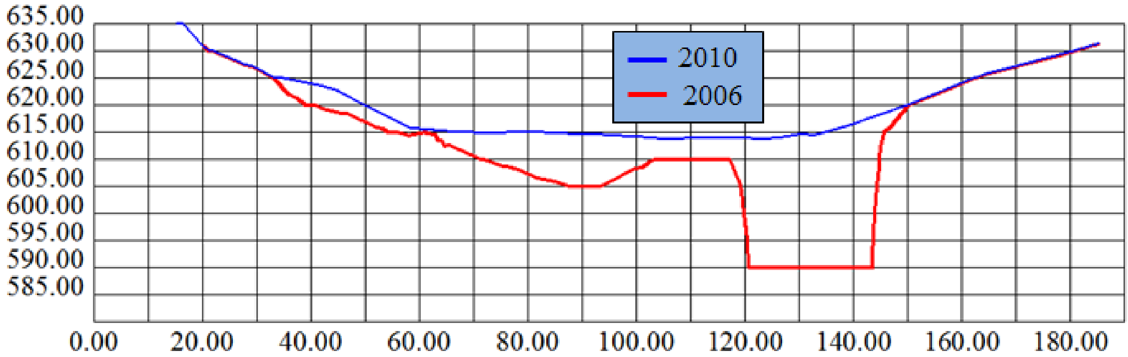


Figure 3-15: Cross section about 70 m upstream of the dam

The Middle Marsyangdi reservoir has lost about 3.52 mill. m^3 by 2010. It is about 65% of the original capacity of the reservoir. Sediment deposition pattern upstream of the intake area is shown in Figure 3-14 and cross section about 70 m upstream of the dam is shown in Figure 3-15. It should be noted that this deposition was due to the six weeks of operation of the reservoir at Highest Regulated Water Level (HRWL) during June-July 2009. The deposition pattern shows that the sediment deposition will be extended towards the intake area. Intake capacity will be significantly reduced when the sediment deposition will reach to the intake (Figure 3-14). Boulders are falling on the left bank of reservoir during the reservoir flushing when the water level was maintained at lower level as shown in Figure 3-14 (Kayastha, 2010). This boulder riprap was provided to protect the rock fill dam. If the dam was protected by a concrete slab, the flushing could be done without any risk.

3.4 Conclusion and Recommendation

Base maps of the Kaligandaki A and Middle Marsyangdi reservoirs were developed on the basis of bathymetric survey carried out in December 2010.

The results of the survey conducted in the reservoirs show that the current storage capacities of the Kaligandaki A and Middle Marsyangdi reservoirs are 3.77 mill. m^3 and 1.94 mill. m^3 which represent 51% and 65% total losses in the Kaligandaki A and Middle Marsyangdi reservoirs, respectively. However, the remaining storage capacities of the Kaligandaki A and Middle Marsyangdi reservoirs within the live storage are 3.0 mill. m^3 and 1.43 mill. m^3, respectively. Total losses within the live storage are 6.7% and 14.1% of the live storage capacity of the Kaligandaki A and Middle Marsyangdi reservoirs, respectively. The higher loss in the live storage area of the Middle Marsyangdi reservoir is due to the reservoir operation at HWRL during June-July 2009.

Kaligandaki A power plant can generate with its full capacity (144 MW) for 6.23 hours continuously with the remaining capacity of 3.0 mill. m^3 excluding the inflow during generation. Similarly Middle Marsyangdi power plant can generate with its full capacity (70 MW) for 4.97 hours continuously with the remaining capacity of 1.5 mill. m^3. This is more than the peak load demand time (4.5 hours in evening) in Nepal.

The main sediment related problem in the Kaligandaki A hydropower plant is high quantity of sediment entry to the settling basins and high sediment concentration in the flow after the settling basin. Measures to minimise the sediment entry to the settling basins and to increase the trap efficiency of the settling basins should be worked out in the Kaligandaki A hydropower plant.

Bathymetric survey of the entire reservoir should be conducted at regular intervals of two/three years and after significant flood events. The intake area should be surveyed every year after the monsoon to assess the deposition pattern and extension of deposit towards the intake area.

Measures to control of the sediment extension towards the intake area should be worked out in Middle Marsyangdi reservoir. The extension of sediment deposit towards the intake and falling of boulder from riprap during the reservoir flushing at minimum reservoir level can be prevented by building up a waterway along the left bank by shifting the sediment deposit from the intake area to left bank.

The reservoir flushing is recommended during the annual peak flood, not at a relative low flow situation at the end of the monsoon as recommended in the operation manual of the plant.

4 SEDIMENTATION MONITORING IN THE KULEKHANI RESERVOIR

4.1 KULEKHANI HYDROPOWER PLANTS

Kulekhani I hydropower plant is situated in Lower Mahabharat Range of Makwanpur District in Nepal. The dam is situated at 21 km southwest of Kathmandu. This power plant is so far the only reservoir type power plant in Nepal. Its installed capacity is 60 MW in total, consisting of 2 Units of 30 MW each. Its estimated average annual energy production is 165 GWh as primary energy & 46 GWh as secondary energy. The power plant was constructed under the financial assistance of the World Bank, the Kuwait Fund, the OPEC Fund, the UNDP & the Overseas Economic Co-operation Fund (OECF) of Japan.

The Kulekhani II power plant with installed capacity of 32 MW was the second stage development of Kulekhani power plant, that utilises released water from Kulekhani I power plant and a small quantity of water from Mandu river. The Kulekhani II power plant was commissioned in December 1986.

Kulekhani III hydropower project (14 MW) is under construction. It is a cascade project that mainly utilizes the tail water from Kulekhani II hydropower project. Discharge from Khani Khola will also be added at the tailrace of Kulekhani II.

4.1.1 KULEKHANI WATERSHED

The Kulekhani watershed area lies at the north-eastern part of Makwanpur district in the Central Development Region of Nepal. The catchment area is approximately 126 km^2. The area is composed of rugged terrain and is surrounding by numerous mountains and valleys. The watershed area is drained by the Palung Khola and many tributary streams and rivulets. Figure 4-1 shows the catchment area of the Kulekhani reservoir.

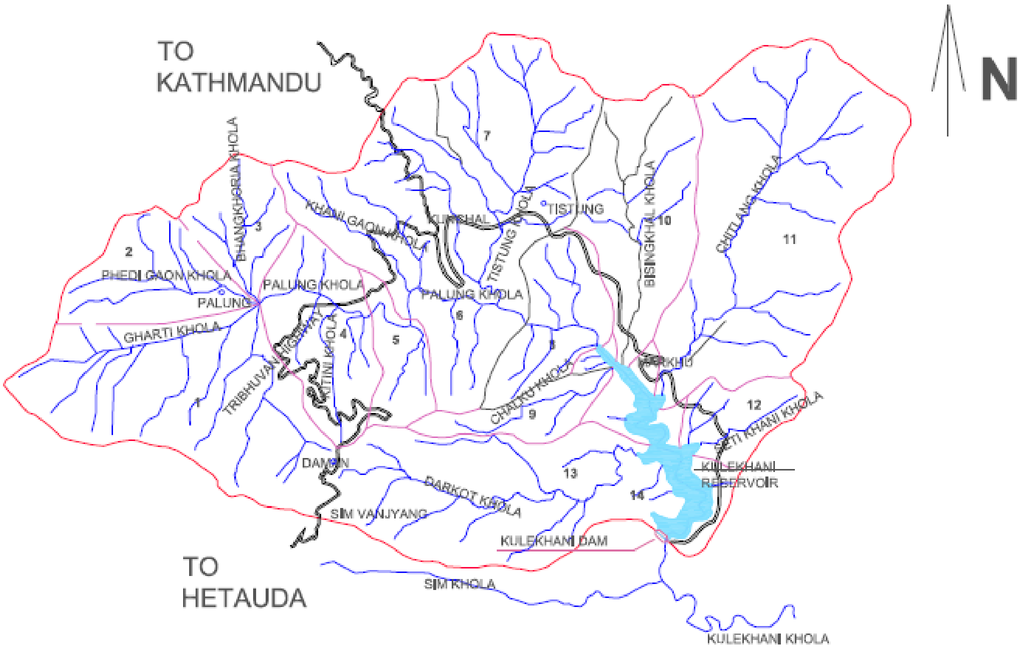


Figure 4-1: Catchment area of Kulekhani reservoir (Source: Sangroula, 2005)

Topography and Land Use

The elevation of catchment area varies from 1,534 masl at the dam site to 2,621 masl at the peak of Simbhanjyang of the Mahabharat Range, which is located at the southern part of the watershed (Nippon Koei, 1994).

Wide and relatively flat land spreads throughout the middle part of the watershed mainly consisting Palung, Tistung and Chitlang valleys. These areas are well cultivated and densely populated. The river gradient of the tributaries are gentle in the flat valley, upstream of the Kulekhani reservoir.

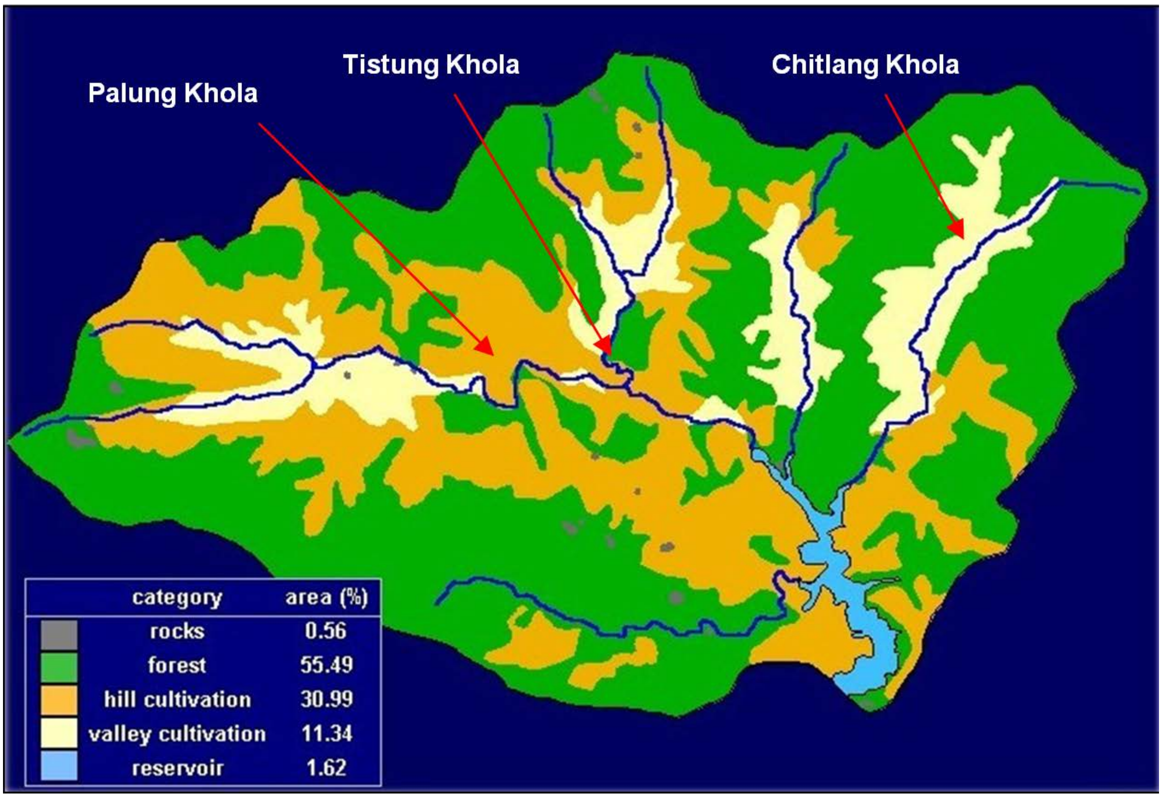


Figure 4-2: Land use map of Kulekhani watershed (Source: Sangroula, 2005)

Figure 4-2 shows a land use map of the Kulekhani watershed. The main land use categories are forest, hill slope cultivation, valley cultivation, rock exposer and reservoir. According to the land use map, the forest area occupies 55% and hill cultivation 31% of the watershed (Sangroula, 2005).

Geology

Figure 4-3 shows the geological map of the watershed of Kulekhani reservoir. The watershed lies on the Kathmandu complex of the Lesser Himalaya. The Kathmandu complex is divided into Bhimphedi and Phulchoki groups separated by an unconformity. The Kulekhani formation is well-bedded alternation of the biotic schist and micaceous quartzite of dark and light as well as green, grey colours. Rock slides observed around Phedigaon were located on the schist of the Kulekhani formation (Nippon Koei, 1994).

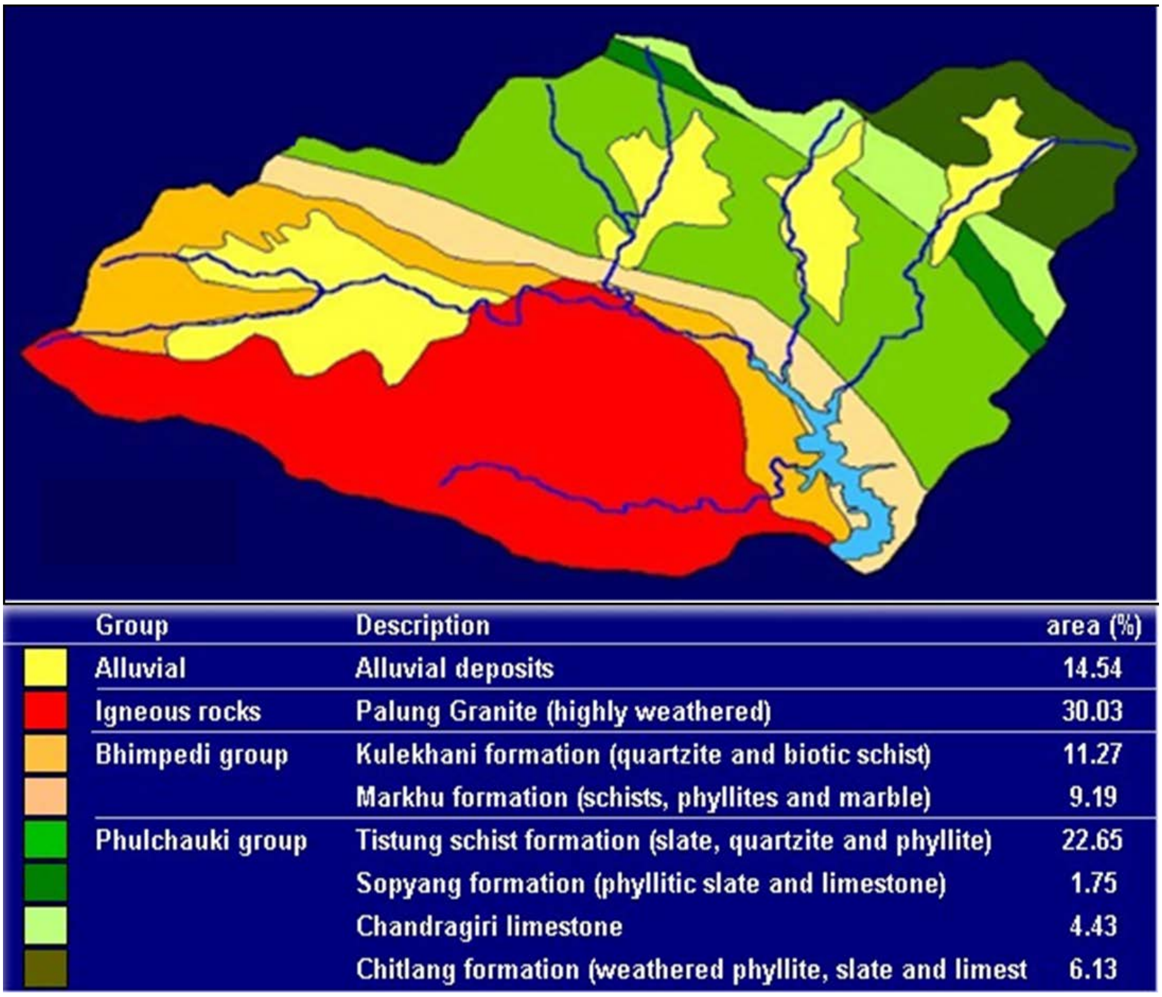


Figure 4-3: Geological map of Kulekhani watershed (Source: Sangroula, 2005)

Hydrology

The Kulekhani river system originates from the middle mountains. The river is a tributary of the Bagmati river. It is divided into eight divisions based on the drainage system. Each system is known by the name of the major river that catches all the incoming water drained towards it. They are: Palung Khola, Sankhmool Khola, Tistung Khola, Bisingkhel Khola, Chitlang Khola, Simbhanjyang Khola and Tasar Khola. The drainage area, which accumulates water for power generation, comprises Kulekhani, Chakhel and Sim catchments. Chakhel and Sim rivers joins Khulekhani river about 1 km downstream from the main dam. Table 4-1 gives the hydrological features of the rivers used for power generation.

Table 4-1: Hydrological features of rivers used for power generation (Nippon Koei, 1994)

Sub catchments	**Catchment area (km^2)**	**Annual total runoff (mill. m^3)**	**Mean (m^3/s)**
Khulekhani	126	122.9	3.9
Chakhel	23	22.4	0.71
Sim	7	11.8	0.37
Total	***156***	***157.1***	***4.98***

4.1.2 DAM AND RESERVOIR

The Khulekhani dam is a rockfill dam. The dam is 114 m in height and has a crest of 397 m long and 10 m wide. The crest of the dam is located at 1,534 masl and has a freeboard of 4 m above the high water level. The upstream slope is 1:2 between the crest elevation and 1,519 masl and 1:2.35 below 1,519 masl while the downstream slope is 1:1.8. The foundation area of impervious core embankment was excavated up to bedrock and a grout curtain was provided to control excessive seepage from the reservoir (Nippon Koei, 1994). Plan and cross section of the Kulekhani Dam is shown in Figure 4-4.

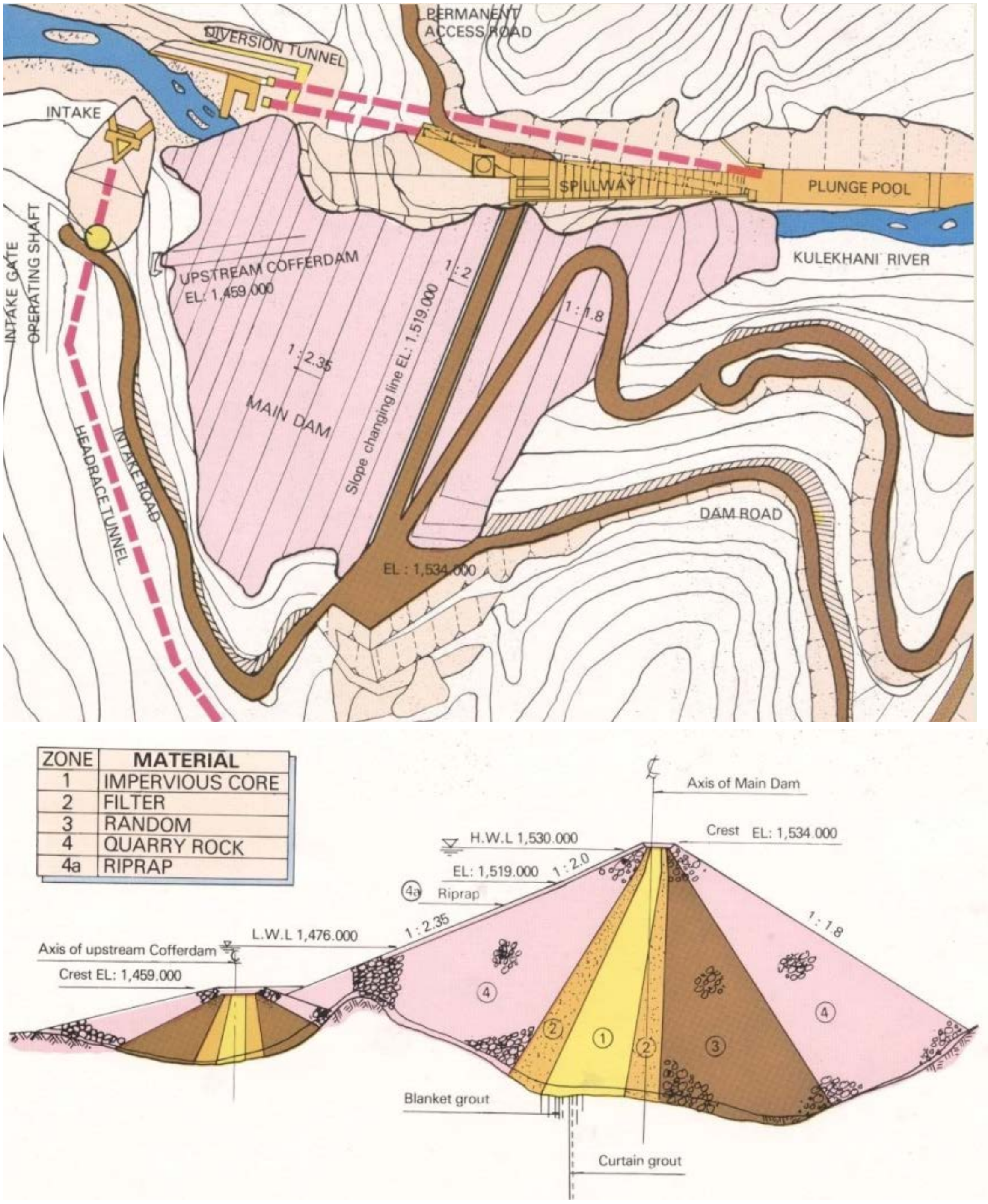


Figure 4-4: Plan and Cross section of Kulekhani Dam (Source: Sangroula, 2005)

The full supply level of the reservoir is 1530.2 masl. The original total volume of the reservoir was 85.3 mill. m^3 of which 73.3 mill. m^3 was live and 12.0 mill. m^3 was dead volume with reference to the level of previous intake invert level of 1471 masl. After the completion of the sloping intake, the invert level of the intake has been raised to an elevation of 1480 masl, due to which the dead storage of the reservoir has increased by 5.05 mill. m^3. Consequently, the live storage has decreased by the same volume (NEA, 2007).

The following salient features show the project component as constructed (Nippon Koei, 1994).

Reservoir

High water level	:	1,530 masl
Low water level	:	1,476 masl
Drawdown	:	54 m
Surface area	:	2.2 km^2
Gross storage capacity	:	85.3 mill. m^3 as per design
Effective storage capacity	:	73.3 mill. m^3 as per design

Dam

Type	:	Zoned rockfill dam
Crest elevation	:	1,534 masl
Dam height	:	114 m
Crest length	:	397 m
Upstream slope	:	1:2:0 above 1,519 masl
	:	1:2.35 below 1,519 masl
Downstream slope	:	1:1.8

4.2 Reservoir Sedimentation Monitoring

4.2.1 Reservoir Sedimentation Monitoring by NEA

Nepal Electricity Authority (NEA) carried out first sedimentation survey in Khulekhani reservoir in 1989. It was followed by a similar study in 1993 by the department of soil conservation, who established additional benchmarks to carry out the survey. On 19th and 20th July 1993, exceptional torrential heavy rain and consequent flood hit the Kulekhani projects area as well as other parts of Nepal. The flood was extremely large and many landslides occurred in the Kulekhani watershed. In December 1993 and September 1994, Department of Soil Conservation carried out more surveys in the reservoir area. During April and May 1995 NEA/Nippon Koei Company made similar survey of the area. Since 1993, NEA is conducting sediment monitoring of the reservoir every year by the use of range line method of survey.

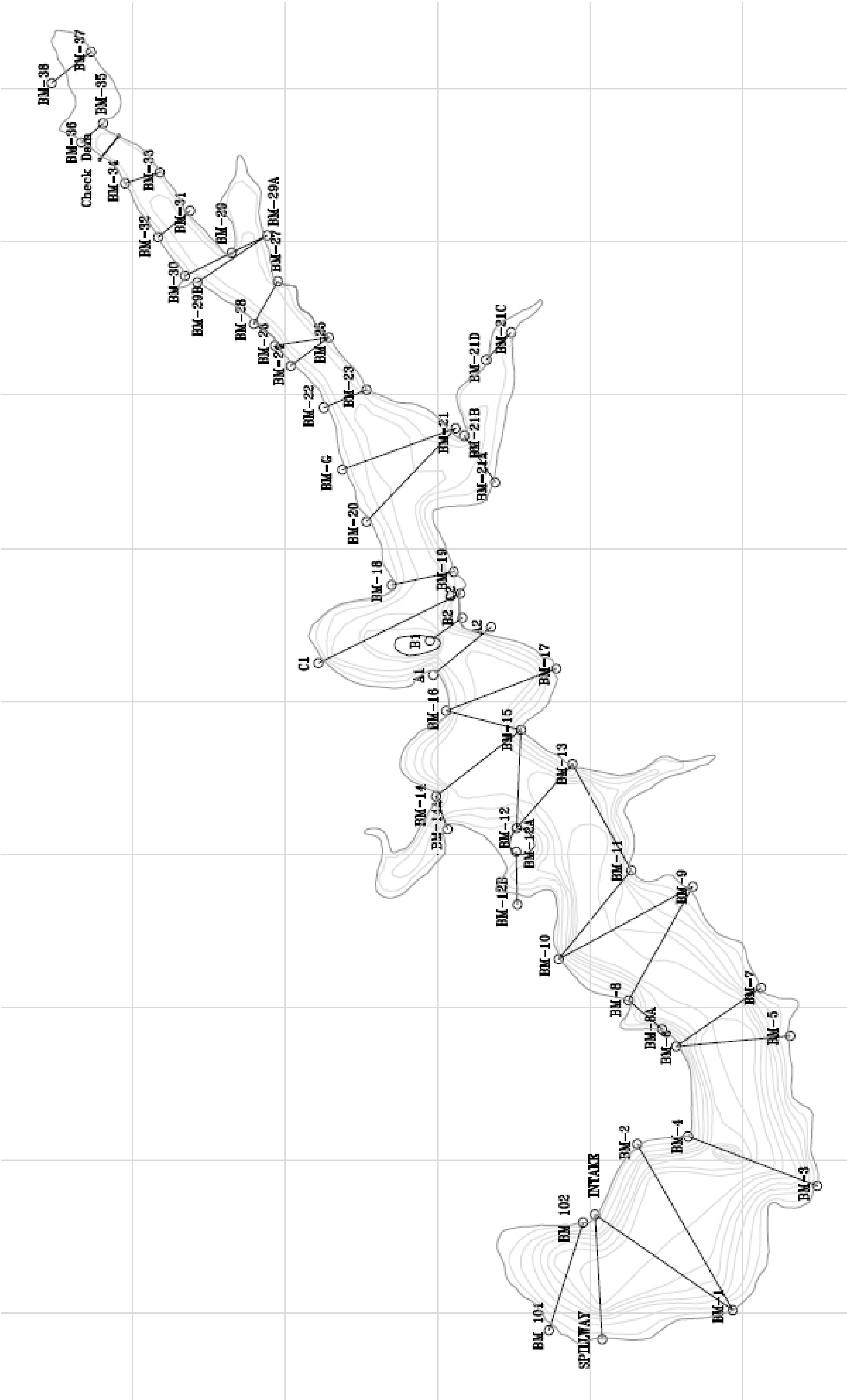


Figure 4-5: Bathymetric map with position of range lines (NEA, 2007)

The NEA Surveys were carried out from a boat using Range Line method. A rope marked at every 20 m interval was stretched between the two set benchmarks along which the measurement was carried out. The shore-to-shore distance was measured by rope and electronic total station (ETS). The distance between BMs and BM to shore were measured by ETS. The depth was measured by sonic sounding equipment at every 20 m on the rope.

Figure 4-5 shows the positions of range lines prepared by Nepal Electricity Authority (NEA).

It was determined that about 19 mill. m^3 of sediment accumulated in the reservoir during three monsoon periods (1993-1995) due to the flood in 1993 (Figure 4-6). Kulekhani reservoir sedimentation study shows that the remaining live storage capacity of the reservoir is 54.6 mill m^3 and total capacity of 60 mill m^3 (up to September 2010) (NEA, 2011). The average annual loss rate is about 1% of the original capacity of the reservoir.

The reduction of water storage of the reservoir is shown in Figure 4-6. It is observed that the rate of sediment accumulation in the reservoir is decreasing since 1995.

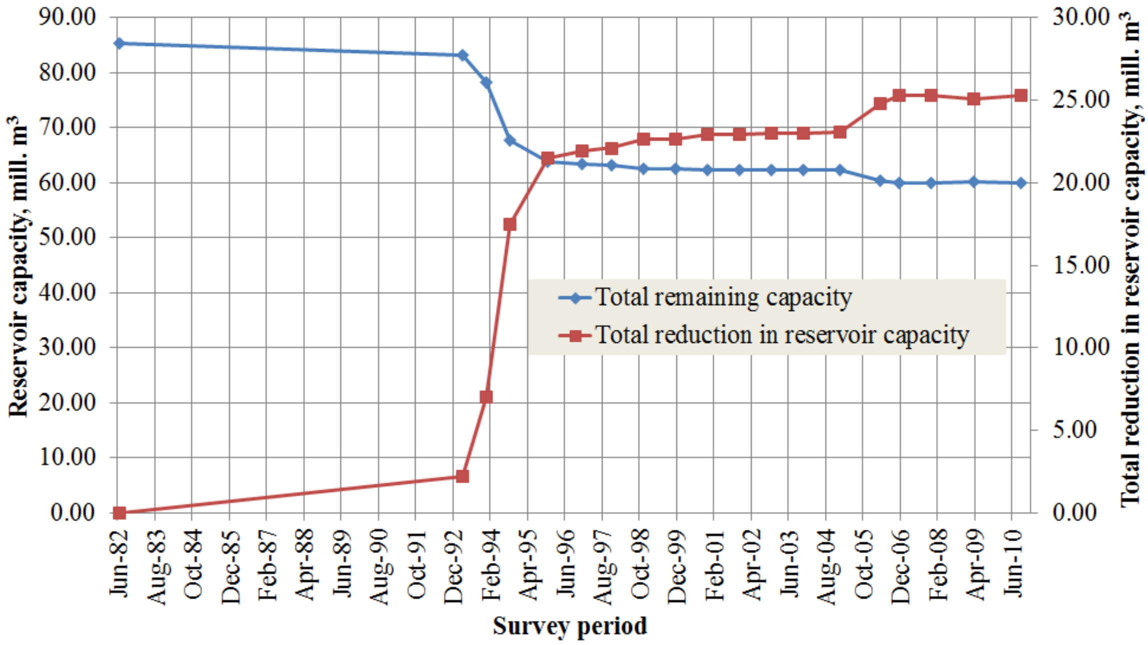


Figure 4-6: Reservoir capacity loss and deposition variation curves (Modified from NEA, 2001 and NEA, 2011)

Sediment deposition rates in the reservoir are assessed with respect to annual volume reduction rate in percentage and annual sediment deposition depth in meters. The volume reduction rate is computed with sediment deposition volume by dividing the remaining volume of the corresponding years. The sediment deposition depth is calculated from annual sediment deposition volume dividing by the reservoir area. The assumption is that all sediment deposition is distributed equally to entire area of the reservoir. The reservoir capacity losses in terms of percentage and sediment deposition depth are shown in Figure 4-7 and Figure 4-8, respectively.

According to the study of accuracy of DGPS bathymetric survey, the average volumetric error and average error in depth measurement were 1.1% and 0.3 m, respectively. However, Figure 4-7 and Figure 4-8 show that the sediment deposition is less than the average errors in survey with DGPS method after 1995 except in 2006.

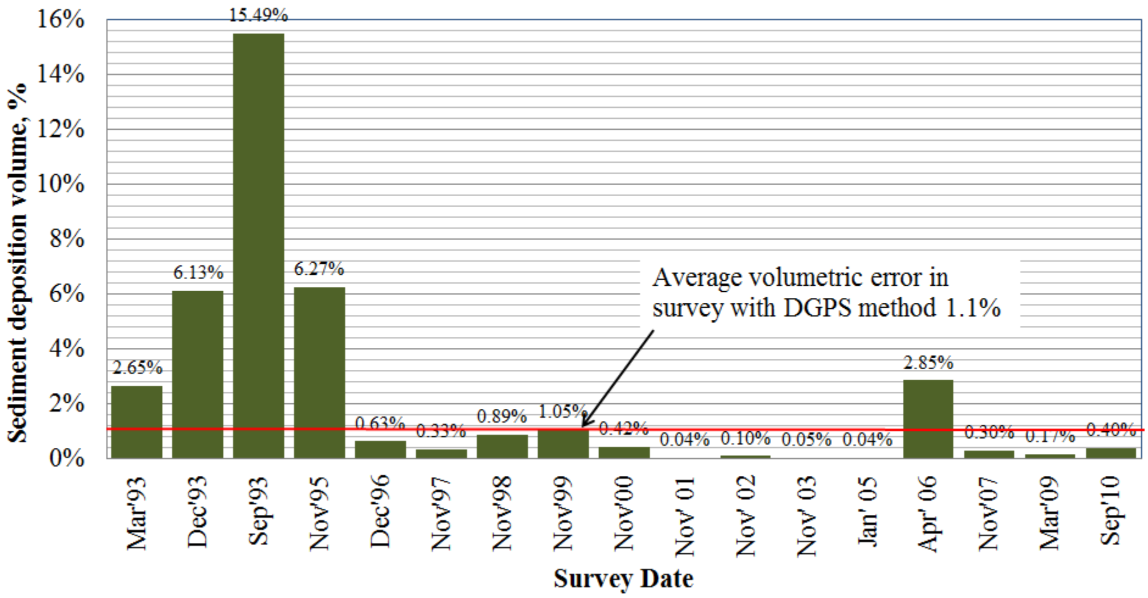


Figure 4-7: Reservoir capacity loss in terms of percentage

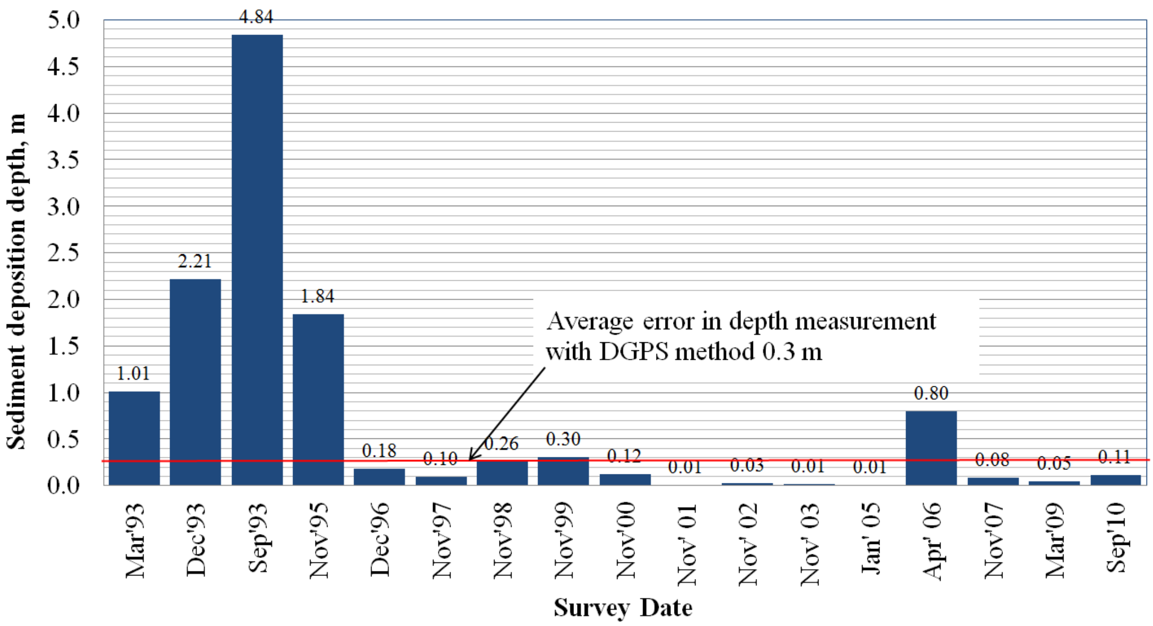


Figure 4-8: Reservoir capacity loss in terms of sediment deposition depth

4.2.2 CHALLENGES IN SURVEYING IN KULEKHANI RESERVOIR

The major challenge to conduct the bathymetric survey in the Kulekhani reservoir is to find a suitable boat for surveying. It was difficult to find motor boat, therefore, a local boat was used for the survey. The boat was about 5 m long and 1 m wide at the center (Figure 4-9). Such boats are used to transport fish. The main problem with the boat was the leakage and the working environment was also inconvenient. Another problem was high wind velocity after 11:00 AM and it was difficult to row the boat.

Figure 4-9: Wooden boat with equipments installed

Figure 4-10: Fish cages and floating house in the reservoir

Kulekhani reservoir is also used as small fish farms by local farmers. They were farming fish in 5x5x5 m cages until middle of 2010. Bamboos are being used for demarcation of the cages and to fix the nets on the surface as shown in Figure 4-10. Cages are mounted with ropes and fixed with trees by the shoreline for their stability. Since the transducer is located below the water surface, it was difficult to travel in these areas.

Because of the above mentioned challenges, it was quite difficult to follow same track lines as recommended by Sangroula (2005).

4.2.3 RESERVOIR SURVEY BY DGPS METHOD

Reservoir surveys were carried out using the contour method of surveying using automated hydrographic surveying equipment in 2003, 2004, 2009 and 2010. First two surveys in 2003 and 2004 were carried out by Sangroula (2005) and last surveys in 2009 and 2010 by the author. The same survey equipment and procedures were adopted as explained in Chapter 2. The survey track lines for survey 2003/2004 and 2009/2010 are shown in Figure 4-11 and Figure 4-12, respectively.

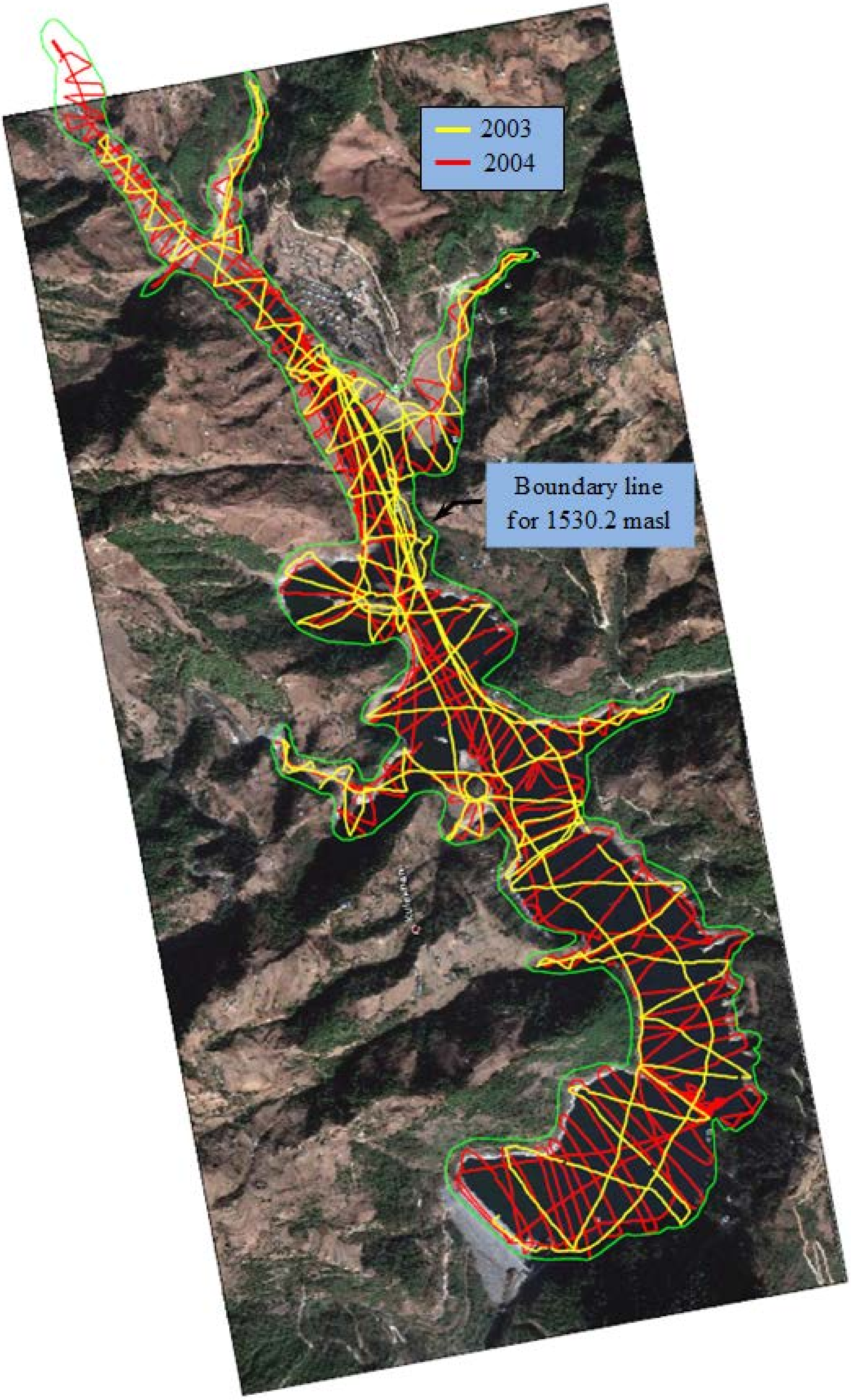


Figure 4-11: Survey track lines of 2003 and 2004 surveys

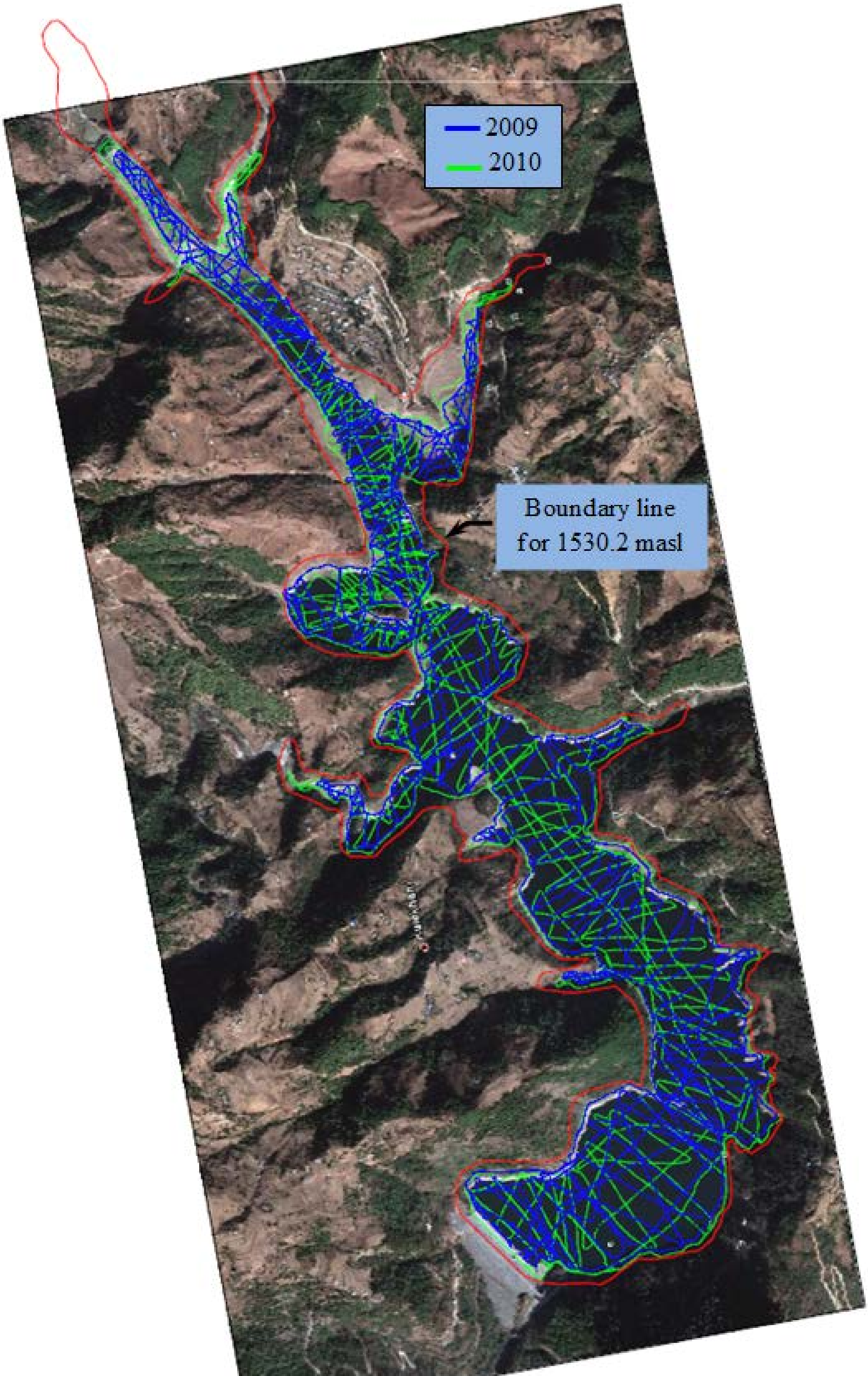


Figure 4-12: Survey track lines of 2009 and 2010 survey

Bathymetric map based on 2009 and 2010 survey

The procedure of data procession and map generation based on the bathymetric survey data of the Kulekhani reservoir is the same as explained in Chapter 2. The bathymetric maps of the reservoir were developed with a total number of 20,900 and 13,400 data points for 2009 and 2010, respectively.

The bathymetric maps of Kulekhani reservoir based on 2009 and 2010 are shown in Figure 4-13 and Figure 4-14, respectively.

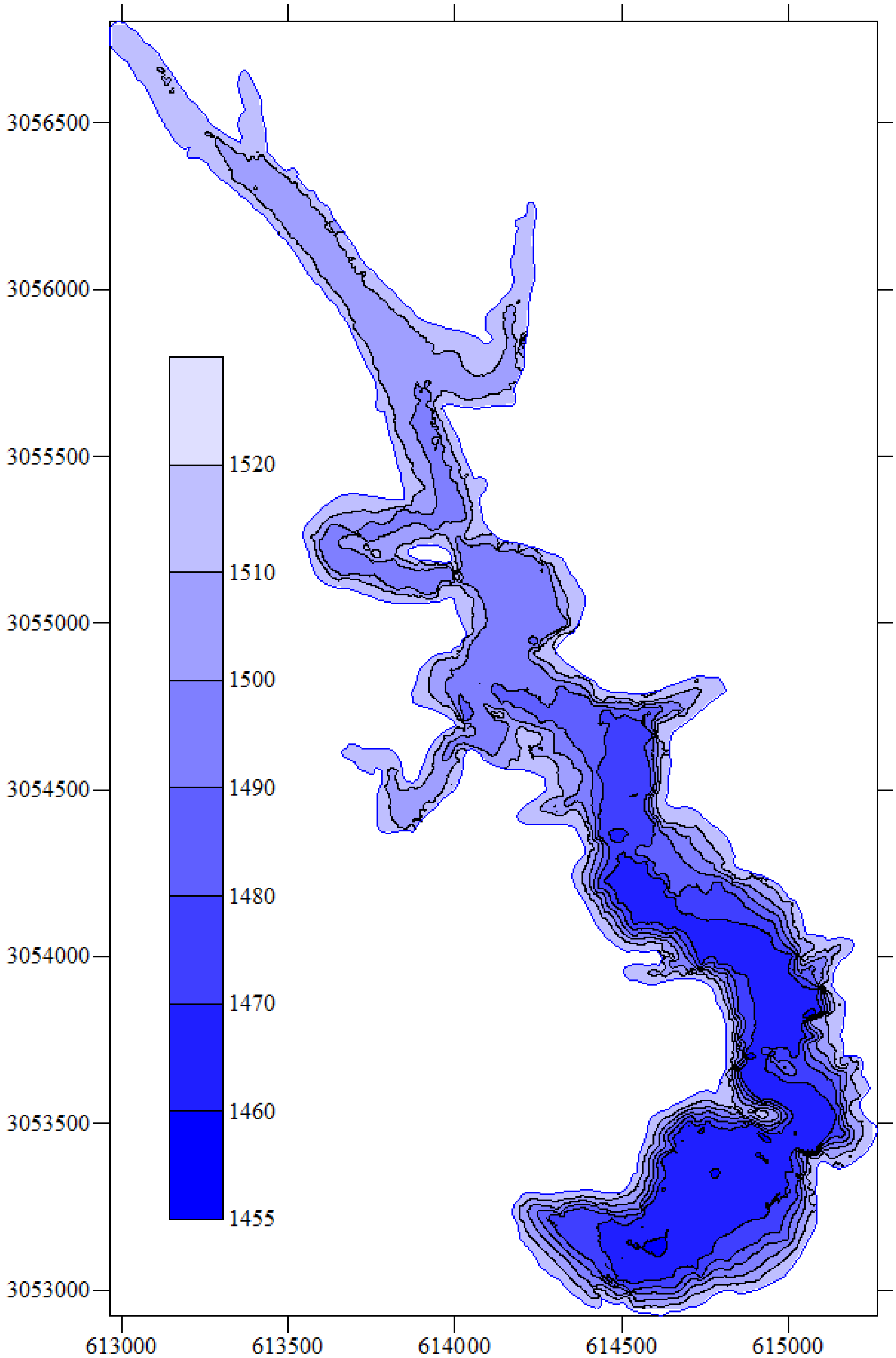


Figure 4-13: Bathymetric map based on the 2009 survey

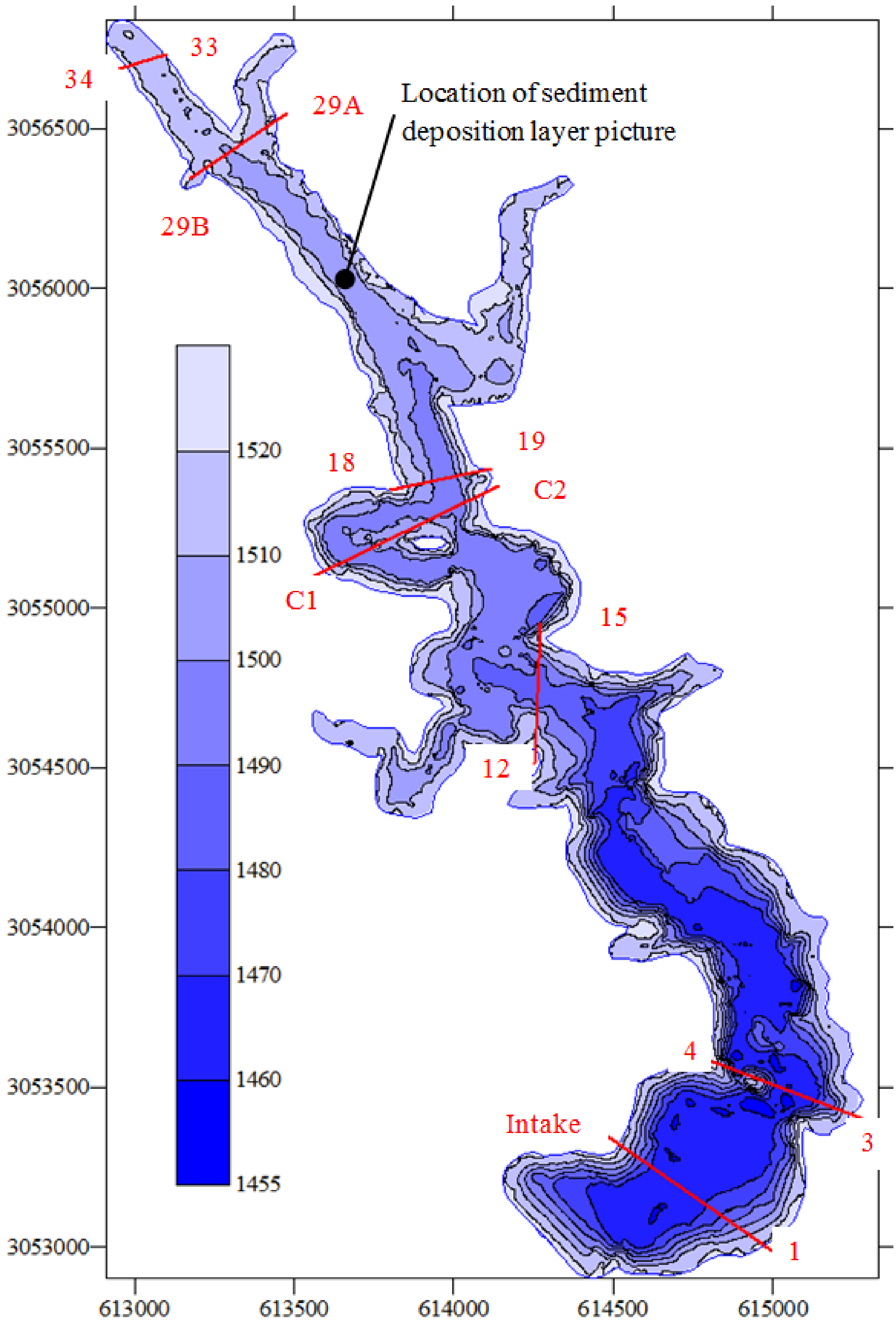


Figure 4-14: Bathymetric map based on the 2010 survey

The bathymetric maps show that there is no significant change within the reservoir from 2009 to 2010. However, some deposition occurred in Chitlang Khola, about 100 meters upstream of the confluence and some scoring occurred in the middle of the reservoir (about 2,400 m upstream of the dam).

The deviation in depths between survey data of 2009 and 2004, 2010 and 2004 and 2010 and 2009 are computed using Surfer Programme as described in Chapter 2. The deviation plots between surveys of 2009 and 2004 and 2010 and 2009 are shown in Figure 4-15 and Figure 4-16, respectively.

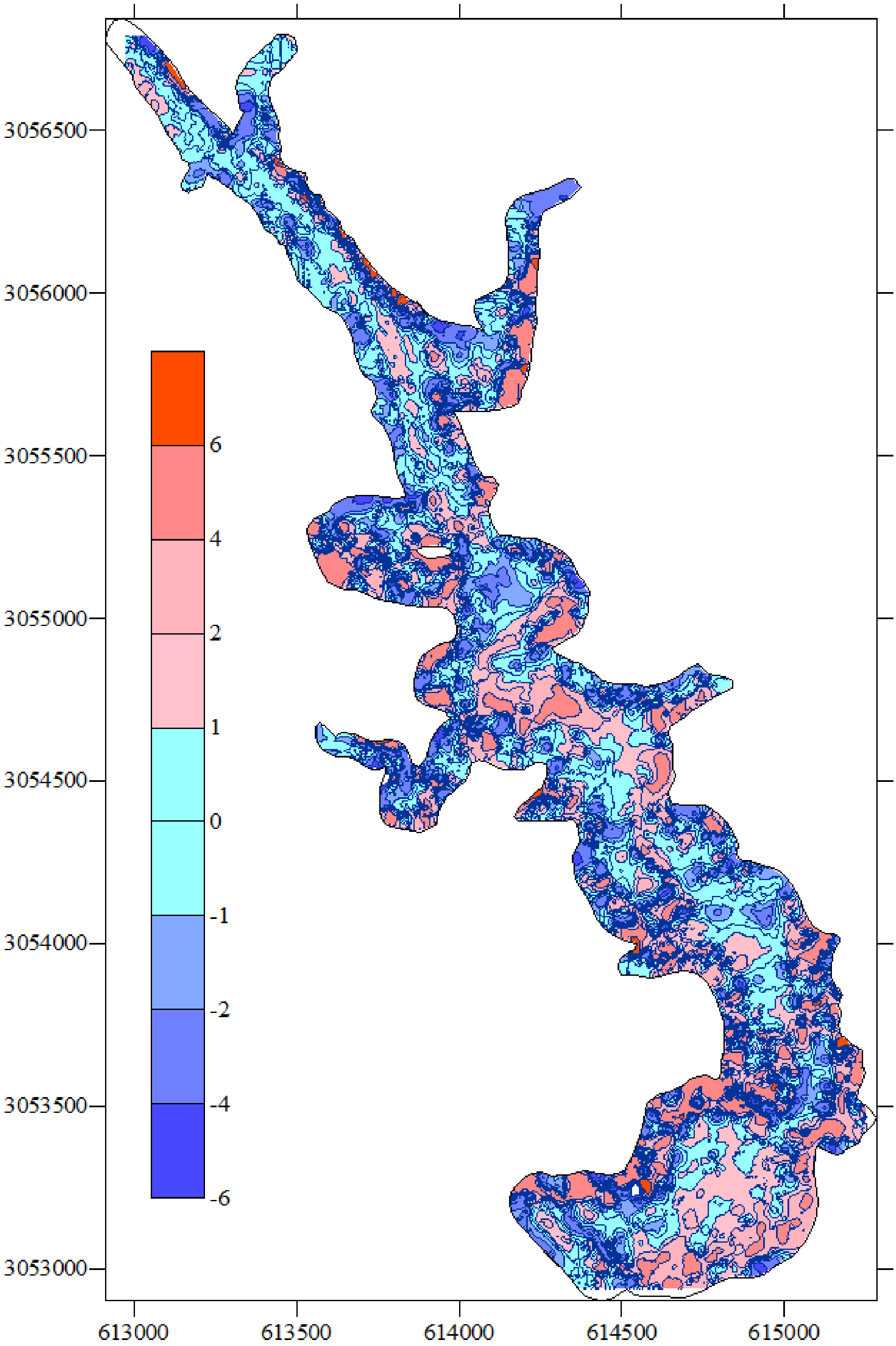


Figure 4-15: Deviation in depth between survey 2009 and 2004

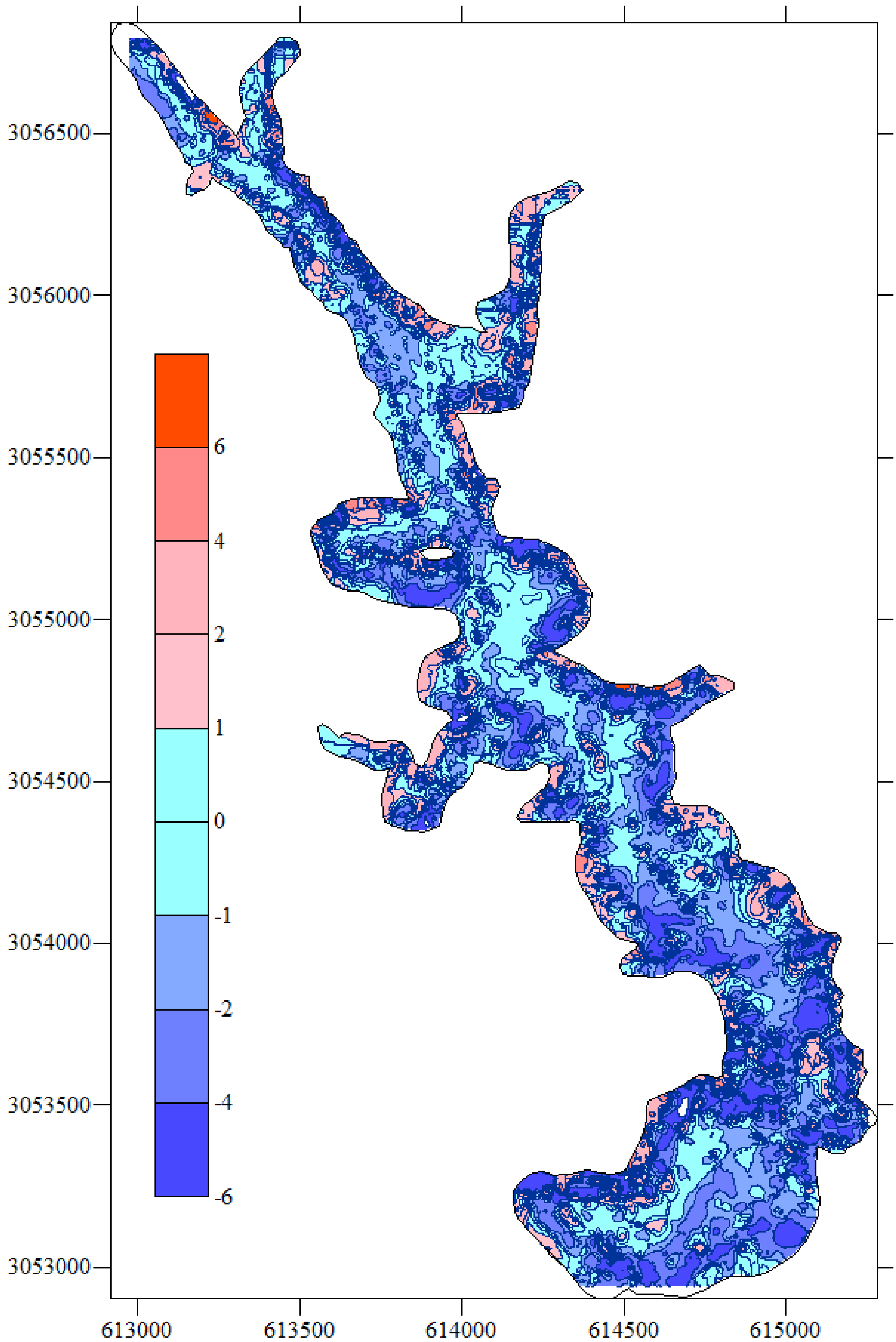


Figure 4-16: Deviation in depth between survey 2010 and 2009

Figure 4-15 shows that the deviations in depths are scattered irregularly in the entire reservoir. The scouring and deposition of sediment are distributed irregularly in the reservoir. Figure 4-16 shows that the deviation is negative in the major part of the reservoir. This may happen by two possible ways i.e., either the sediment deposit is removed from the reservoir or sediment deposition volume is reduced due to consolidation. However, this is illogical to believe that the sediment deposit could neither be removed nor could be reduced in such amount within a year due to consolidation. The deviation may have been caused due to the following reasons:

- Short intervals of the reservoir surveys; surveys should be carried out as recommended in Chapter 2.
- The errors in the survey due to differences in survey point density, track lines (Figure 4-11 and Figure 4-12). As recommended in chapter 2, the bathymetric survey should include survey along the shoreline, centerline and across the reservoir to minimize the errors of the survey. However, the survey along centerline was not carried out in 2004 and 2009 due to the presence of fish cages as shown in Figure 4-10.
- Method of deviation in computation in the Surfer program
- The errors in the survey due to differences in the water surface level and the number of data points recorded during the survey (see Table 4-2)

Table 4-2: Water level and number of data points

Survey year	**Water level**	**Number of data points**
2003	1528.95 to 1529.05	About 7,000
2004	1530.30 to 1530.45	11,420
2009	1516.61 to 1516.86	20,900
2010	1521.15 to 1521.33	13,408

Higher deviations are observed along the shoreline where the depth gradients are greater than in the middle part of the reservoir. This is due to lack of sufficient points of the survey parallel to the shoreline (Figure 4-11 and Figure 4-12) and the method of interpolation of the Surfer program, which uses a bilinear interpolation method to calculate Z values at points that do not coincide with grid nodes.

4.2.4 VERIFICATION OF SURVEY DATA THROUGH CROSS SECTIONS

Some cross sections at different locations of the reservoir are compared to verify the accuracy of the survey data. The cross sections are compared for the surveys 2009, 2010 and the survey carried out by NEA in September 2010. The cross sections for survey data 2009 and 2010 are developed using Autocad Civil 3D software and the cross section for the survey carried out by NEA is plotted with the cross section data itself. The cross sections are presented in Figure 4-17. The locations of the cross sections are shown in Figure 4-14. These cross sections match each other but there are some inconsistencies in the depth at some locations. This is because the survey track lines did not follow the exact cross section lines during the survey in the field.

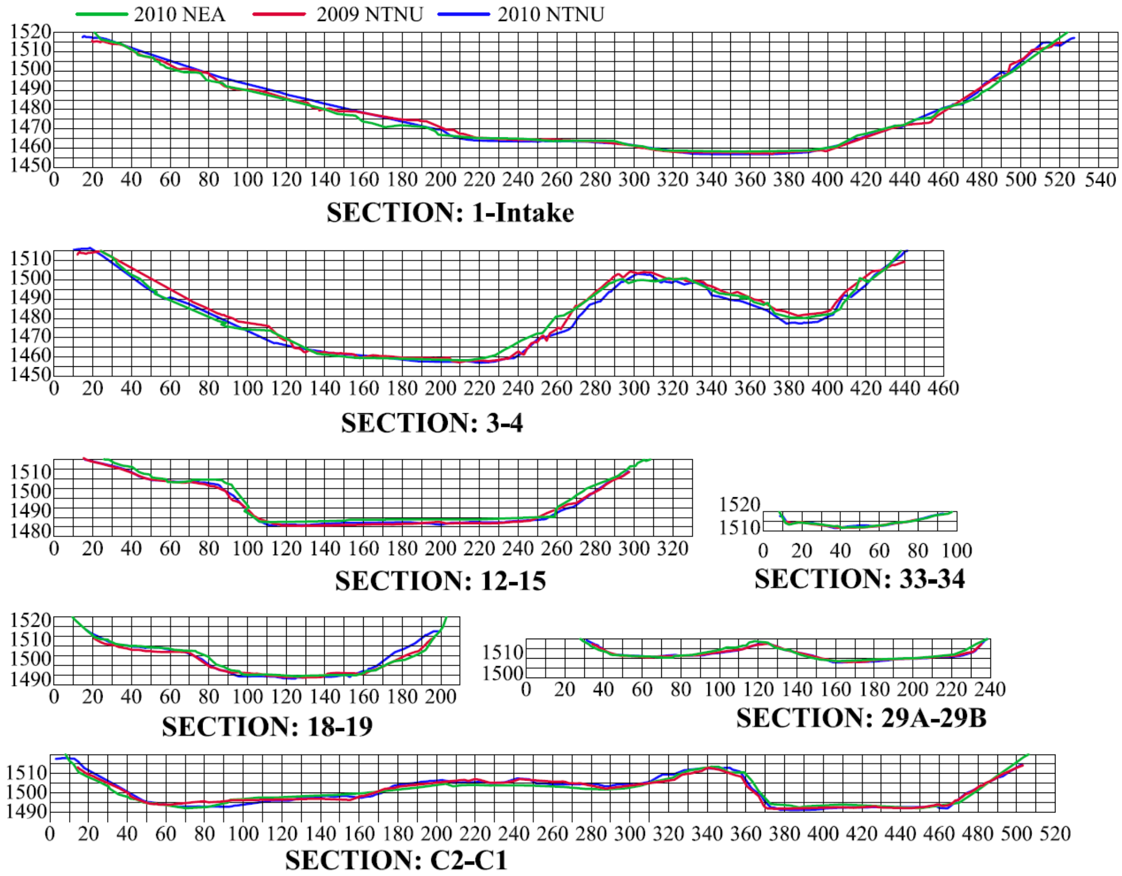


Figure 4-17: Cross sections at different location in the Kulekhani reservoir.

4.2.5 Reservoir Capacity Depletion

The reservoir surface area and the storage capacity of the reservoir are computed with the help of Surfer software. The area and the volume of the reservoir based on 2003, 2004, 2009 and 2010 surveys are tabulated in Table 4-3.

Table 4-3: Surface area and reservoir volume

Survey year	Reference water level (masl)	Volume (m^3)	Surface area (m^2)	Reference
2003	1530.2	60,744,040	2,164,737	Dr. Sangroula, 2005
2004	1530.2	63,599,265	2,166,307	
2009	1515.0	37,282,791	1,658,072	Author
2010	1520.0	45,659,923	1,814,143	

Surface area-elevation and capacity-elevation curves based on the surveys are presented in Figure 4-18 and tabulated in Table 4-4.

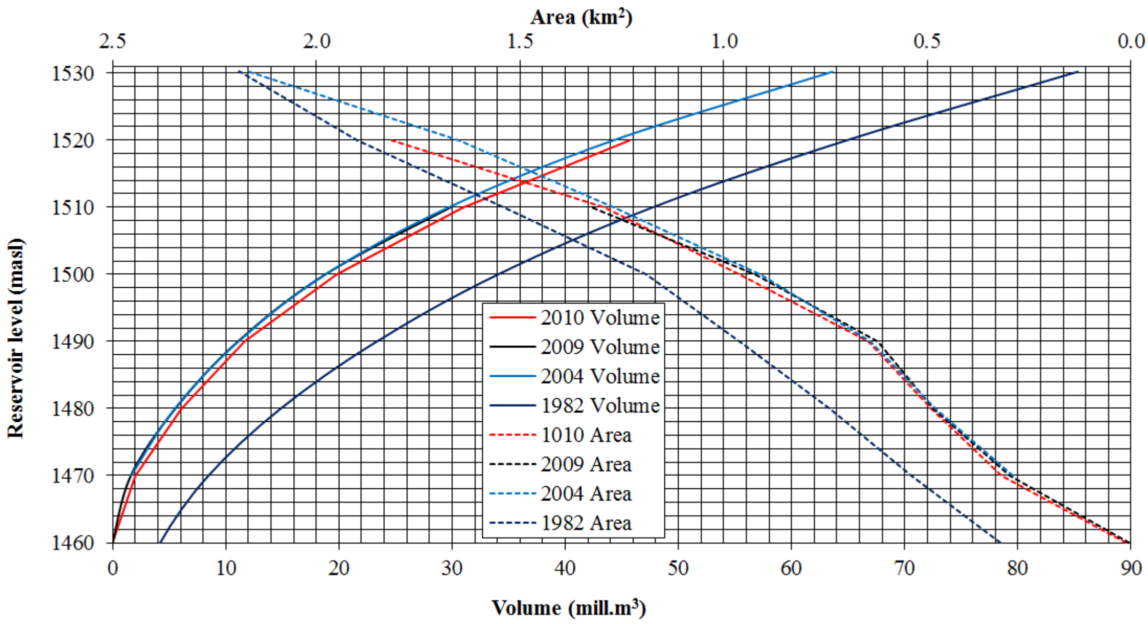


Figure 4-18: Area-elevation and capacity-elevation curves

Table 4-4: Surface area and reservoir volume

Original 1982			**2004**		**2009**		**2010**	
Elevation (masl)	**Area (km^2)**	**Volume (mill. m^3)**	**Area (km^2)**	**Volume (mill. m^3)**	**Area (km^2)**	**Volume (mill. m^3)**	**Area (km^2)**	**Volume (mill. m^3)**
1427.0	0.00	0.00						
1430.0	0.02	0.04						
1440.0	0.08	0.52						
1450.0	0.18	1.77						
1460.0	0.32	4.2			0.01	0.005	0.01	0.01
1470.0	0.54	8.46	0.29	1.64	0.30	1.60	0.32	2.00
1480.0	0.74	14.85	0.48	5.49	0.48	5.54	0.49	6.04
1490.0	0.96	23.34	0.64	11.11	0.62	11.06	0.65	11.67
1500.0	1.19	34.06	0.91	18.82	0.92	18.79	0.96	19.74
1510.0	1.54	47.73	1.27	29.70	1.32	29.86	1.29	30.99
1520.0	1.9	64.91	1.65	44.30			1.81	45.66
1530.2	2.19	85.28	2.17	63.60				

There is not any major change in the topography of the reservoir area between 1517.0 masl and 1530.2 masl (HRWL) after 2004 November, hence the volume above 1515.0 masl can be considered unchanged from 2004. Therefore, the volume of the reservoir based on 2009 and 2010 survey are 63.76 and 64.89 mill m^3, respectively. Kulekhani reservoir has lost 20.4 mill. m^3 in total and 14 mill. m^3 in the live storage in 27 years of operation. The average annual loss rate is about 1% of the original capacity of the reservoir.

4.2.6 SEDIMENTATION IN THE KULEKHANI RESERVOIR

Figure 4-19 shows the trend of longitudinal profile of the Kulekhani reservoir. The longitudinal profiles of the reservoir are created along the thalweg. The longitudinal data for 1971, March 1993, December 1993 and September 1994 are adopted from Nippon Koei (1994). The longitudinal profile of November 1999 is based on the bathymetric survey carried out by NEA in 1999. The profiles of 2004, 2009 and 2010 are based on the bathymetric surveys using DGPS method.

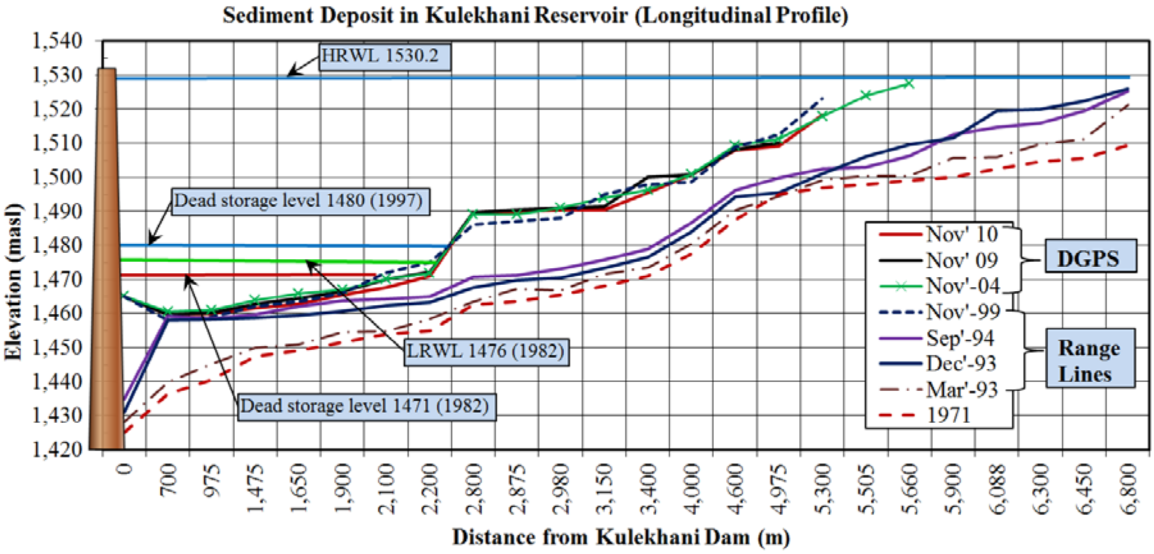


Figure 4-19: Trend of sediment deposition in the Kulekhani reservoir

The profiles of March 1993 (just prior to the July 1993 flood) and December 1993 (after the flood) indicate that the entire length of the reservoir had undergone deposition with the thickest deposit located 6 to 7 km upstream the dam. The largest volume of the sediment however was in the first 1.5 km. In September 1994, much of the sand had moved into the lower reaches of the reservoir while the bed in the upper reaches had degraded and become armoured (Nippon Koei, 1994). From September 1994 to November 1999, 5.2 mill. m^3 sediment deposition occurred with the maximum thickness of 21.5 m. The respective survey indicates that the rate of sediment accumulation in the reservoir is decreasing since 1999. There were not many changes in the profiles after 1999. The profiles of 1999 and 2004 show that the scouring occurred at 2,100 m to 2,500 m, 3,000 m to 3,500 m and upstream from 4,600 m upstream of the dam and moved towards downstream part of the reservoir. Similarly, in 2010 there was some scouring at 3.5 km upstream of the dam.

Figure 4-20 shows sediment deposition strata in the Kulekhani reservoir about 500 meters downstream from the main check dam (Figure 4-14). The picture indicates that about 90 centimeters thick sediment deposition occurred in this area due to the 1993 flood. The deposition of 1993 composed of gravels, pebbles and sand. The thicknesses of the sediment deposition were thin and consist of silt and sand in other years. It indicates that bigger sized sediment were deposited upstream of the main check dam and the rate of sediment accumulation into the reservoir is decreasing since 1995.

Figure 4-20: Sediment deposition layers in the Kulekhani reservoir (For location, see Figure 4-14)

4.3 CONCLUSIONS AND RECOMMENDATION

The volume of the reservoir based on 2010 survey is 64.9 mill m^3. The Kulekahi reservoir has lost 20.4 mill. m^3 in 27 years of operation. The average annual loss rate is about 1% of the original capacity of the reservoir.

More than 18.0 mill. m^3 of sedimentation occurred in the reservoir due to the disastrous floods of 1993. After the flood of 1993, the slopes of the drainage basin has been stabilised to some extent. The sediment inflow to the reservoir is less after 1995.

It is found out that the total volumes of the reservoir based on the 2009 and 2010 surveys are more than the total volume from 2003 and 2004 surveys and NEA data, which is not that realistic. The main reason of such difference in volume computation may be the error in survey due to the differences in density of survey points, survey tracks and reservoir levels during the survey as explained in Chapter 2. Difference in total volume estimated by NEA and NTNU (Sangroula and Shrestha) may be because of the difference in surveying and computation methods.

The annual average thickness of sediment deposition in the Kulekhani reservoir is 0.16 m from 1996. Similarly, the annual average sediment deposition volume is 0.56% of total reservoir capacity. Which are lower than the average error of the bathymetric survey with the DGPS method. The bathymetric survey with DGPS technique incorporates a volumetric error from 1 to 5 percent of the total reservoir volume and the error in depth from 0.02 m to 0.5 m (see chapter 2) depending on the reservoir depth, survey track lines and point density. Therefore, the bathymetric survey in the Kulekhani reservoir should be conducted at an interval of at least five years.

The bathymetric surveys show that sedimentation level around the intake area is increasing and approaching to the intake level. Certain measures should be considered to prevent further deposition in the intake area of the reservoir. For instance, hydraulic dredging can be used to shift the sediment deposits from the intake area to dead volume area to the left bank of the reservoir close to the dam and spillway area.

5 OPTIMUM LONG-TERM USE OF THE KULEKHANI RESERVOIR

5.1 BACKGROUND

Nepal's theoretical hydropower potential is estimated at 83,000 MW. At present 45,000 MW have been identified economically feasible. However, to the date, Nepal has developed less than two percent of its vast hydropower potential. The total installed capacity is 700 MW, of which 644 MW (93%) is generated by hydropower plants in the integrated system. The locations of major hydro power plants of Nepal are shown in Figure 5-1.

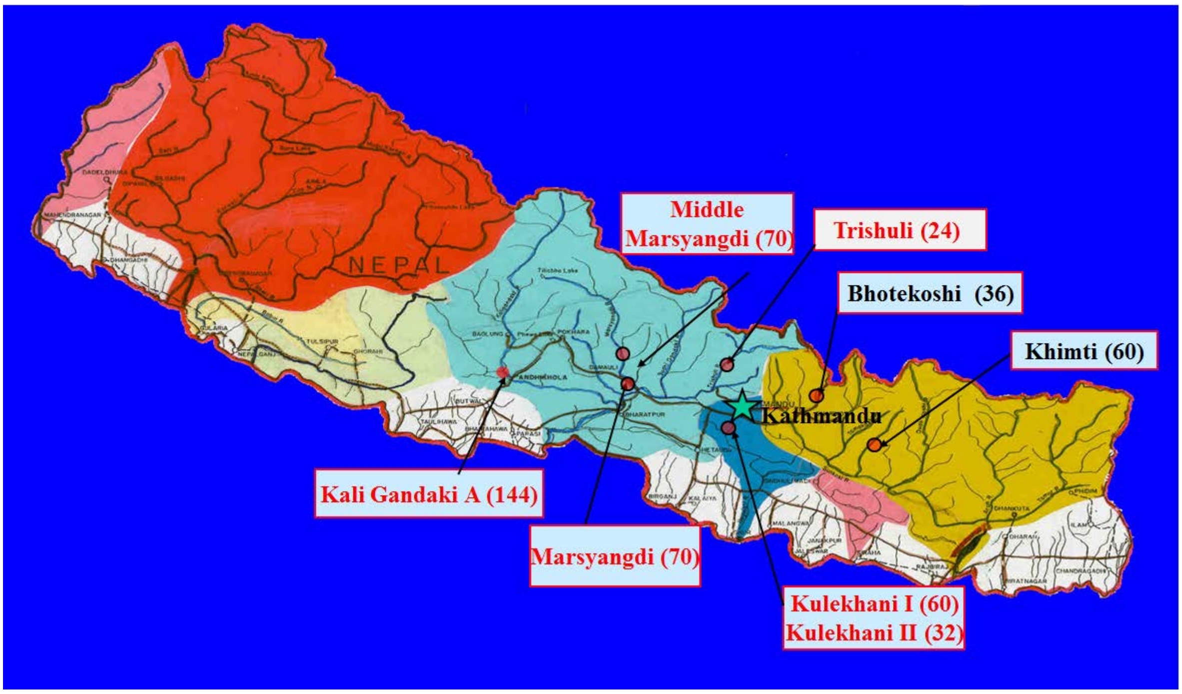


Figure 5-1: Location map of major hydropower plants of Nepal

The peak power and energy demand is growing by about 10% annually, creating electricity shortage in Nepal. Demand has been exceeding supply every year. To meet the significant gap between demand and supply, Nepal Electricity Authority (NEA) had increased power cuts in the country up to 16 hours a day in the dry season. The greatest challenge of NEA is bridging the gap between supply and demand of electricity. The energy situation in Nepal is presented in Appendix B.

All existing hydropower plants in Nepal are run-of-river (RoR) type except Kulekhani power plant and some are RoR plants with daily regulating capacity. Therefore, the peaking capacity as well as energy output of the power system drops in the dry season from December to May due to the shortage of available runoff. The load shedding hours in the dry season depends mainly on the availability of water in the Kulekhani reservoir.

The historical generation data from the major power plants shows that RoR plants generated power at lower capacity (up to 50% of the total capacity) during the wet season, but at the same time Kulekhani power plants also generated power up to 50% of its total capacity. If RoR plants generated at maximum capacity to meet the demand, Kulekhani power plant would not have to operate during the wet season.

If energy production from major hydropower projects is analysed and a new reservoir operation regime (reservoir guide curve) of the Kulekhani reservoir is developed, the gap between supply and demand could be reduced.

5.2 OBJECTIVE AND SCOPE OF THE STUDY

The objective of this study is to develop an alternative operation regime for the Kulekhani reservoir, which will optimize the use of the limited resources in Kulekhani within the context of the Nepalese Power Grid and assess the availability of water to remove deposits from the reservoir.

A model is developed for the optimum operation of the Kulekhani reservoir and some key PRoR power plants. The model is based on historic data records. The model focuses on water availability, generation and storage capacities and the power demand. The transmission part of the system is not considered a bottleneck in this context in order to focus on generation and storage capacities.

The main limitations of this study are:

- Generation from IPPs, NEA RoR, thermal plants and import from India are assumed as unregulated RoR generation.
- Only daily generation data is available for all power plants except Kaligandaki A power plant. The daily generation data is broken down to hourly generation data according to the available daily system load demand of the corresponding month.
- Daily inflow to Kulekhani reservoir is calculated according to the generation of the power plant and the observed reservoir level.
- The model is developed for the year 2006/2007. It can be used for other years according to available data with minor modifications.

5.3 METHODOLOGY

The daily generation and system load data including the Kulekhani reservoir was collected from System Operation Department of Nepal Electricity Authority (NEA) for the years 2005 to 2007. Hourly generation data of Kaligandaki power plant was collected from Kaligandaki power plant in Syanja. The generation data is presented in Appendix C.

Figure 5-8 presents the steps involved in the Excel model for the operation of the Kulekhani reservoir and some key PRoR power plants.

Hourly system load demands and generation from different plants are available for only one day in each month but daily peak loads and generation data are available for the fiscal year 2006/07. Hourly system load demands for the remaining days are calculated from the daily peak load according to the available hourly system load data for the corresponding months. Similarly, the daily generation from RoR plants of NEA, IPP, thermal plants and import from India are distributed to hourly generation according to the available hourly generation data of corresponding months. This total RoR hourly generation will be compared with the hourly system load demand. If the system load demand is more than the RoR generation, the PRoR plants will be operated to meet the demand.

Hourly generation capacity of Marsyangdi hydropower plant is calculated according to the generation capacity of Kaligandaki A hydropower plant on the pro rata basis of generation and capacity. Generation from the Kaligandaki A hydropower plant is optimised according to the peak load demand and available water volume in the reservoir to meet the hourly system load demand. If the system load demand is more than the sum of RoR and PRoR generation, the reservoir plants will be operated to meet the demand.

Daily inflow to the Kulekhani reservoir is calculated according to daily generation and water level of the Kulekhani reservoir. Kulekhani power plants are operated only when there is deficit in the system load demand. Water level of the Kulekhani reservoir is re-calculated according to the volume of water used for generation and total inflow to the reservoir.

5.4 ASSESSMENT OF ENERGY PRODUCTION FROM MAJOR HYDRO POWER PLANTS

According to NEA (2011), the annual peak power demand of INPS reached 946 MW in 2010/2011. Likewise, annual energy demand was estimated at 4,833 GWh, out of which only 3,850 GWh (79.67%) could be served from available sources and the rest 982 GWh (20.33 %) had to be curtailed as load shedding to keep the system in operational balance.

The power generating facilities in Nepal consists of hydro, diesel and solar power plants but it is basically a hydropower-oriented system. According to NEA (2011), the total installed capacity in Nepal was 705 MW in 2010/2011, and 652 MW (92.5%) was generated by hydropower in the integrated system. Among them, the major hydropower plants (Kaligandaki, Kulekhani, Marsyangdi and Middle Marsyangdi) generated 375 MW (53%).

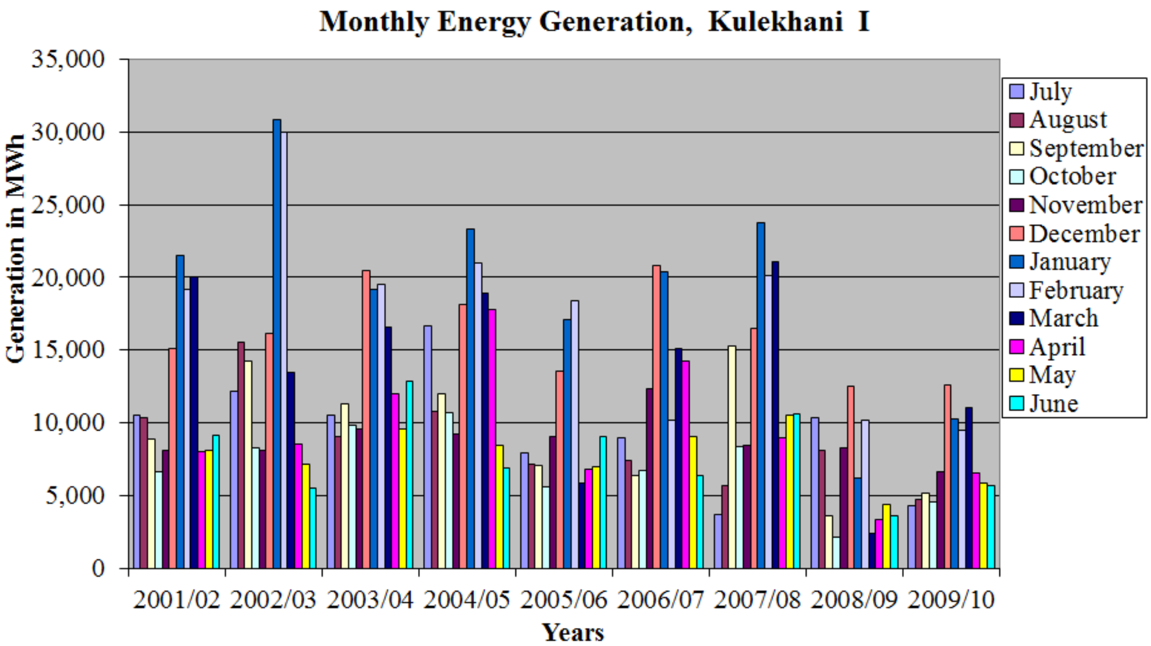


Figure 5-2: Monthly energy generation from Kulekhani I (Modified from NEA Generation, 2010)

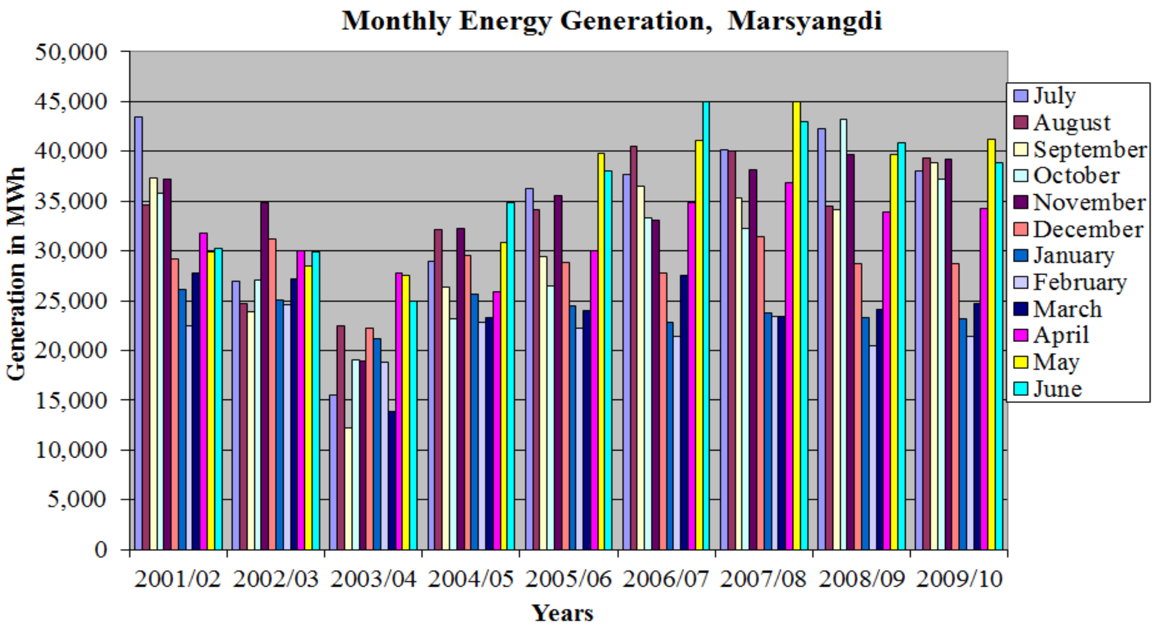


Figure 5-3: Monthly energy generation from Marsyangdi (Modified from NEA Generation, 2010)

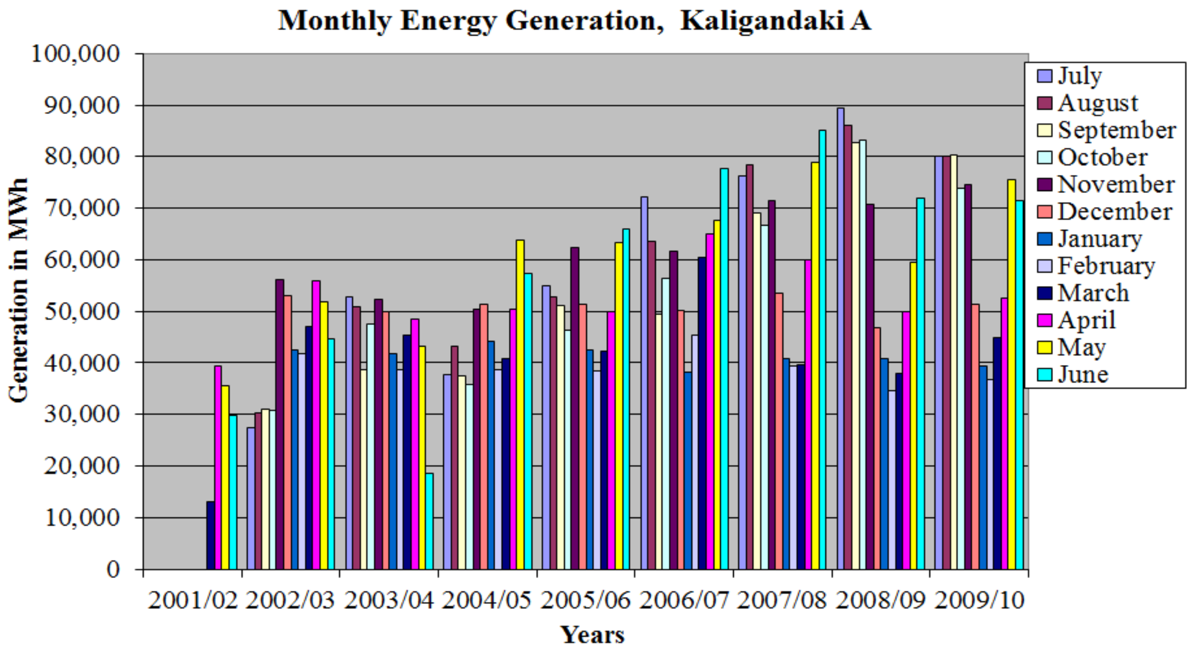


Figure 5-4: Monthly energy generation from Kaligandaki A (Modified NEA Generation 2010)

Figure 5-2 to Figure 5-4 presents the energy generation from major power plants of Nepal (Kulekhani I, Marsyangdi and Kaligandaki A) from 2001 to 2010. They show that Run of river plants (Marsyangdi and Kaligandaki A) generated with lower capacity (up to 50% of the total capacity) during the wet season from year 2001 to 2004, but at the same time reservoir power plants (Kulekhani) also generated up to 50% of its total capacity. If RoR plants generated at maximum capacity to meet the demand, the generation from Kulekhani power plant would not be needed during the wet season.

5.5 ASSESSMENT OF KULEKHANI RESERVOIR OPERATION

Figure 5-5 to Figure 5-7 present daily water level in Kulekhani reservoir and daily generation from power plant from year 2002 to 2010.

Figure 5-5 to Figure 5-7 show that the Kulekhani reservoir were filled to full capacity at the mid-September and continue to mid of December. The reservoir was not filled to its full capacity from 2007/08 to July 2011. The reservoir was operated during monsoon period (June, July and August) to generate energy from 200 MWh to 600 MWh, but at the same time the major run of river plants were operated with lower capacity as shown in Figure 5-3 and Figure 5-4.

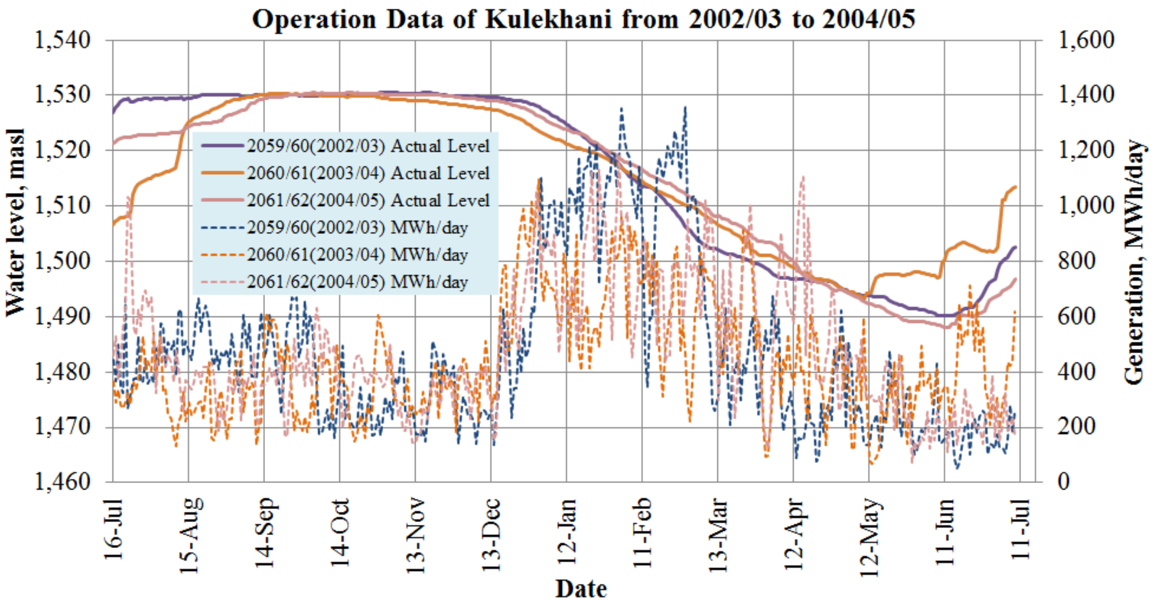


Figure 5-5: Daily water level and daily generation from Kulekhani from 2002 to 2004

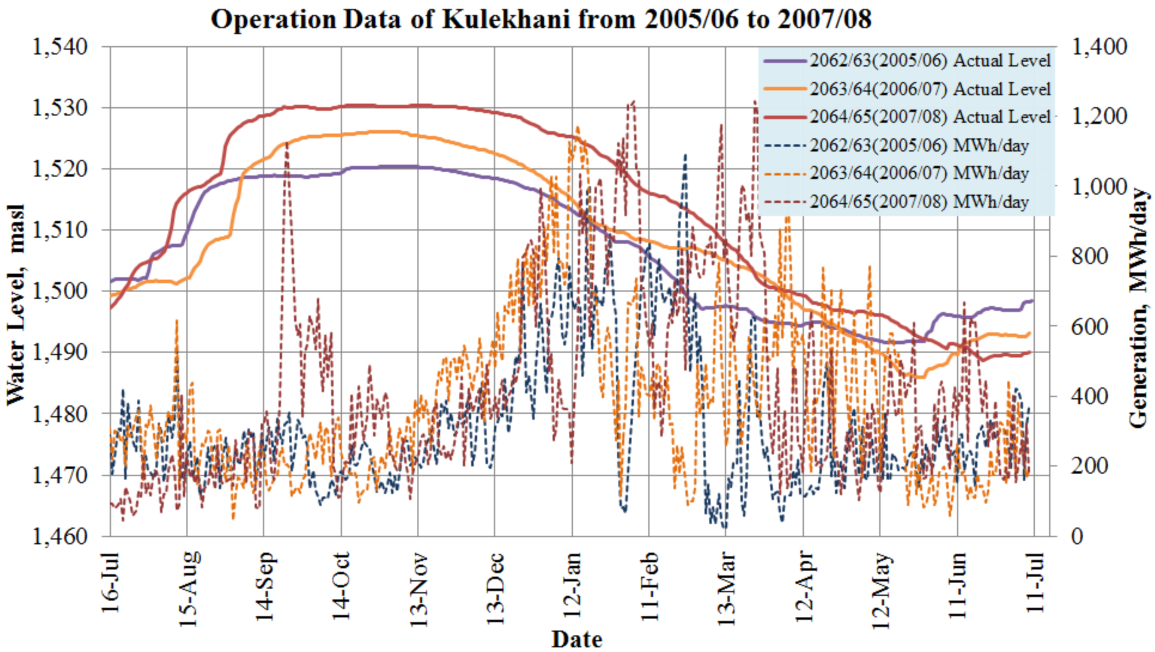


Figure 5-6: Daily water level and daily generation from Kulekhani from 2005 to 2008

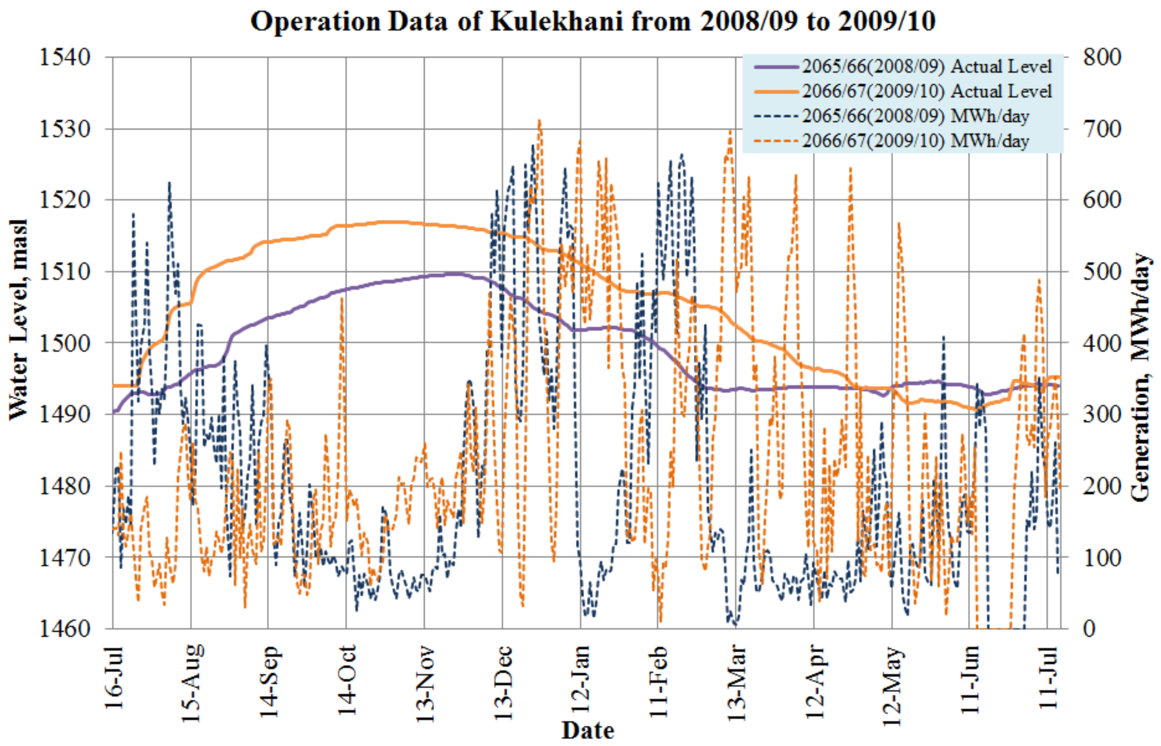


Figure 5-7: Daily water level and daily generation from Kulekhani from 2008 to 2010

Table 5-1: Mean monthly generation and water volume used for generation

Months		Shrawan	Bhadra	Aswin	Kartik	Mangshir	Push	Magh	Falgun	Chaitra	Baisakh	Jestha	Ashadh
		July/Aug	Aug/Sept	Sept/Oct	Oct/Nov	Nov/Dec	Dec/Jan	Jan/Feb	Feb/Mar	Mar/Apr	Apr/May	May/June	June/July
Gen	MWh	12,363	13,366	12,414	8,906	10,083	17,855	22,985	21,269	16,326	10,471	8,992	9,425
Volume	mill. m^3	9.82	10.62	9.86	7.07	8.01	14.18	18.26	16.89	12.97	8.32	7.14	7.49

The mean monthly electricity generation and water volume used for generation from July 2002 to July 2010 are shown in Table 5-1. If Kulekhani reservoir is not operated during monsoon (June, July and August) there will be about 27.9 mill m^3 water available for sediment handling every year. The excess water can be used for periodic removal of deposited sediment.

5.6 Alternative Model for Kulekhani Reservoir Operation

Figure 5-8 presents the steps involved in the excel model for the operation of the Kulekhani reservoir and some key PRoR power plants. The model consist of six excel files. They are Daily Gen, Hourly S Load, KGA Gen, RoR Model, PRoR Model, Storage Model and Summary. 'Daily Gen. and 'KGA Gen' files are available data files which are used as input data for the Model. 'Hourly S Load' file consists of Hourly System Load demand data and calculation of hourly system load demand and generation. 'RoR Model', 'PRoR Model' and 'Storage Model' are the Model files. 'Summary' file presents the summary of the Models. Visual Basic applications used in the model is presented in Appendix D.

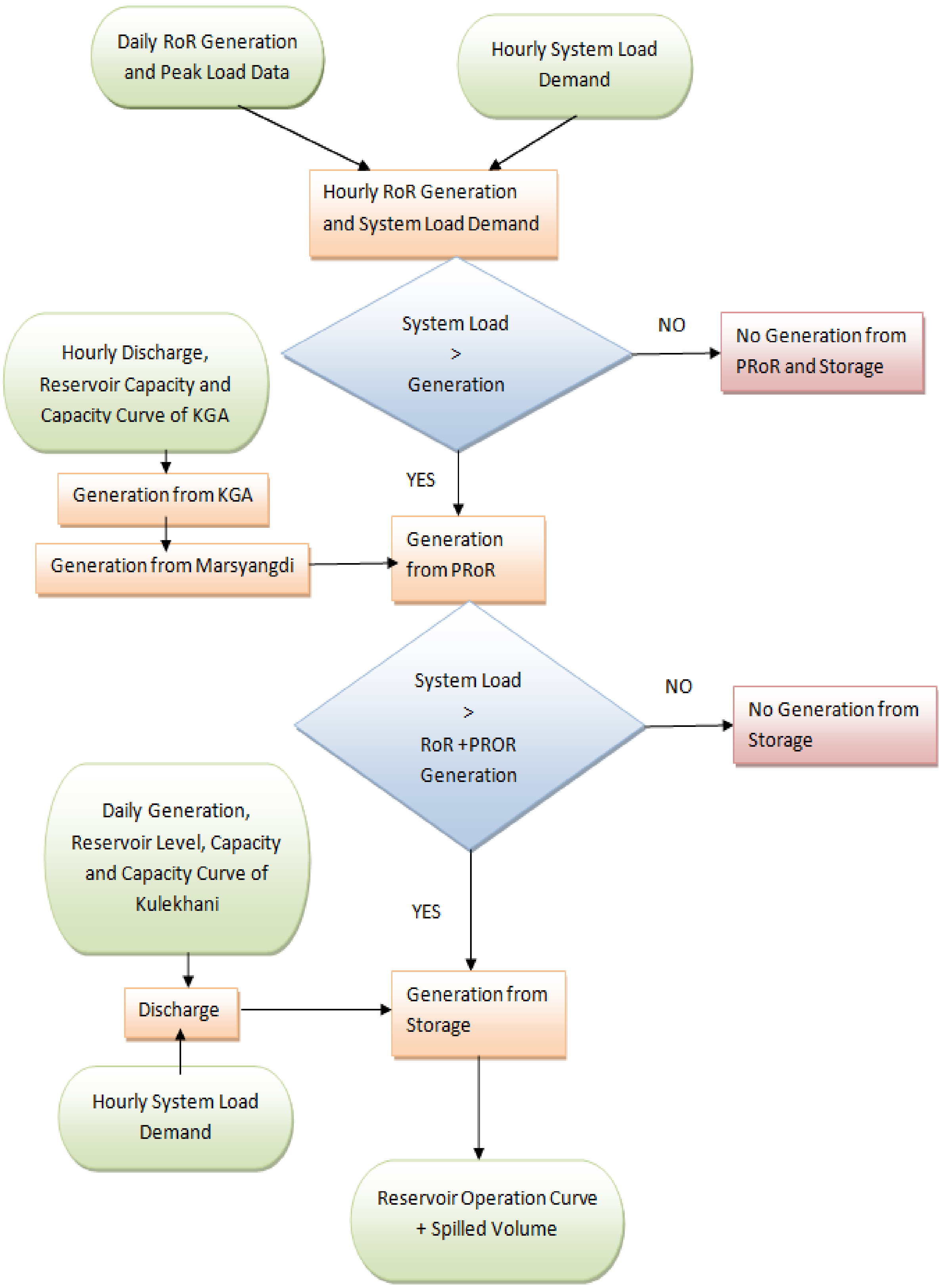


Figure 5-8: Steps involved in the excel model

5.6.1 SYSTEM LOAD DEMAND

Hourly system load demands are available for only one day in each month but daily peak loads are available for the whole fiscal year 2006/07. The dates of available hourly system load are presented in Table 5-2 below. The system load curves for December and February are shown in Figure 5-9 and Figure 5-10, respectively. The daily peak load demand is shown in Figure 5-11. The highest peak demand in the fiscal year 2006/07 was recorded on Poush 06, 2063 BS (December 21, 2006) at 18:20 (System Operation Department, 2007).

Table 5-2: Dates with available hourly system load data in 2063/2064 (2006/2007)

Year	**2063**									**2064**		
Months	Srawan	Bhadra	Aswin	Kartik	Mangshir	Push	Magh	Falgun	Chaitra	Baisakh	Jestha	Asadh
Day	23	6	29	29	21	6	7	15	7	31	1	5
Year	**2006**						**2007**					
Months	August	August	October	November	December	December	January	February	March	May	May	June
Day	8	22	15	15	7	21	21	15	21	14	15	19

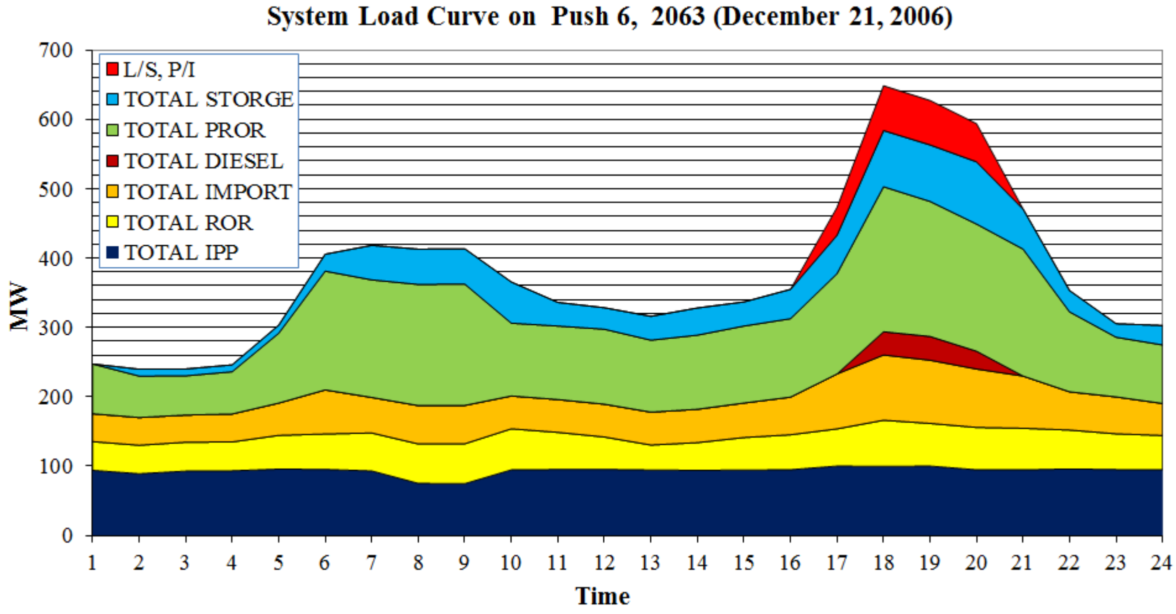


Figure 5-9: System load curve on December 21, 2006

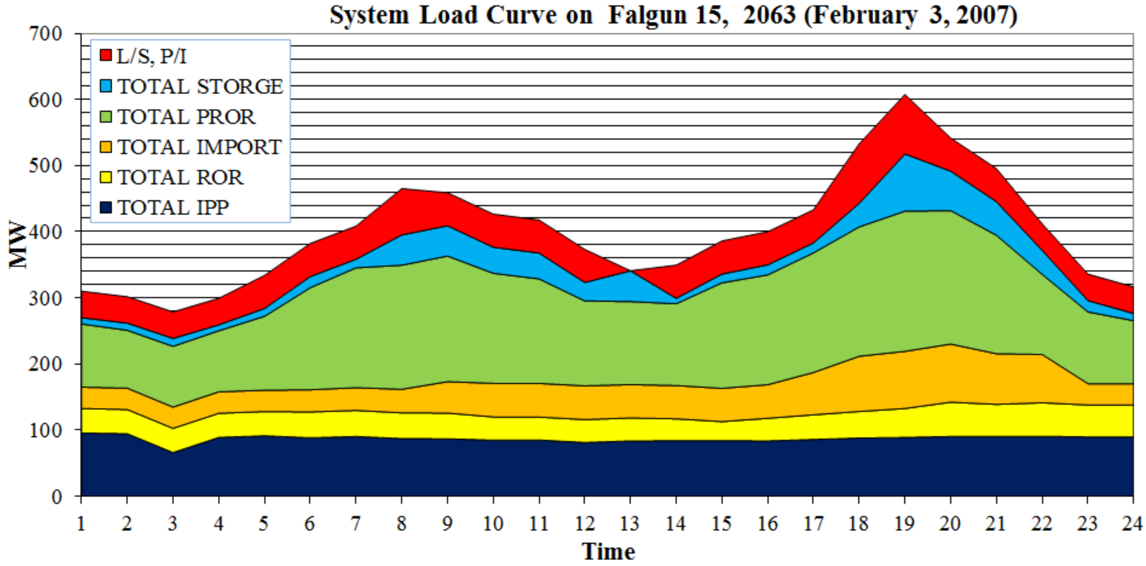


Figure 5-10: System load curve on February 3, 2007

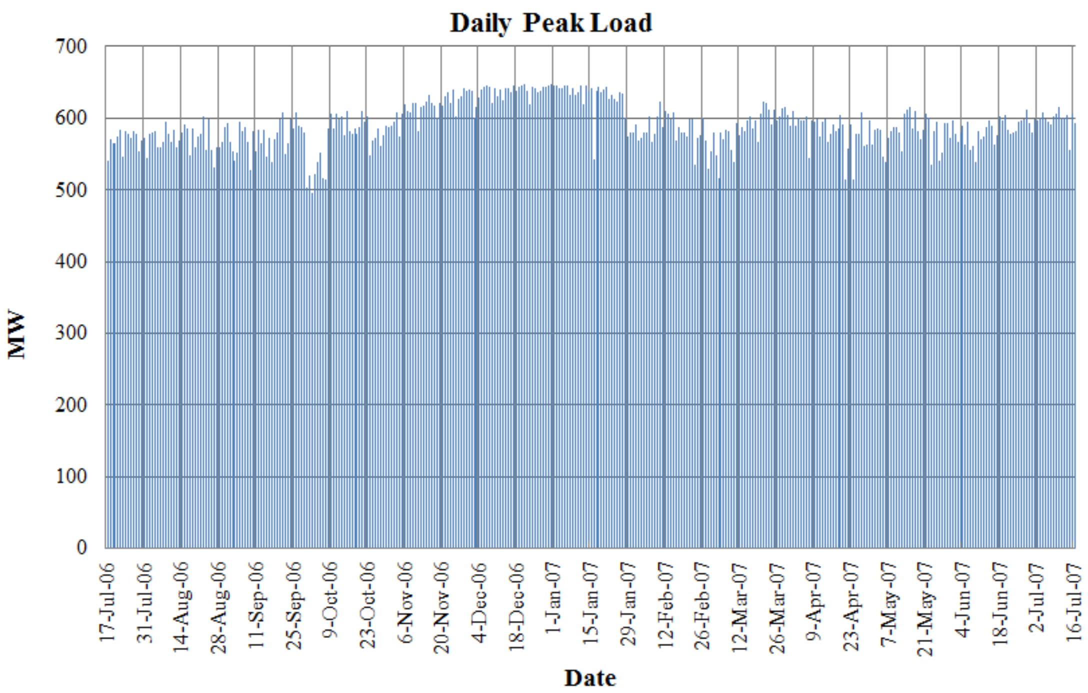


Figure 5-11: Daily peak load

Hourly system load demands for other days of the month are calculated according to the daily peak load and the available hourly system load data of the corresponding months. Hourly system load is calculated as:

$$HSL_t = \frac{DPL \times HSL_{t\ available}}{DPL_{available}} \qquad \text{Equation 5-1}$$

Where,

HSL_t : Hourly system load at time t (1 to 24)

DPL : Daily peak load

$HSL_{t\ available}$: Available hourly system load at time t (1 to 24)

$DPL_{available}$: Daily peak load of corresponding day

Final hourly system load is calculated as:

$$HSL_{t\ final} = HSL_t \times \frac{ML_{NEA}}{ML_{Cal}} \qquad \text{Equation 5-2}$$

Where,

ML_{NEA}: Total monthly system load from NEA

ML_{cal} : Total monthly system load

All the hourly system load demand calculation are done in 'Hourly S Load excel file' for each month in the separate sheets.

5.6.2 Generation from RoR Plants

The total generation from IPPs, NEA RoR, thermal plants and import from India are assumed as RoR Generation. The daily generation from RoR plants of NEA, IPP, thermal plants and import from India are shown in Figure 5-12. Hourly generation data from RoR plants are available for only one day in each month therefore, the daily generation data are distributed to hourly generation according to the available hourly generation data of corresponding months. Daily generation is distributed to hourly generation as:

$$HG_{tp} = \frac{DGp \times HG_{t\ p\ available}}{DGp_{\ available}} \qquad \text{Equation 5-3}$$

Where,

HG_{tp} : Distributed hourly generation at time t from plant p (RoR, IPP etc.)

DG_p : Daily generation from plant p (RoR, IPP etc.)

$HG_{t\ p\ available}$: Available hourly generation at time t from plant p (RoR, IPP etc.)

$DG_{p\ available}$: Daily generation of corresponding day from plant p (RoR, IPP etc.)

All the hourly RoR generation calculation are done in 'Hourly s load excel file' for each month in the separate sheets and summarised in 'RoR model' excel file.

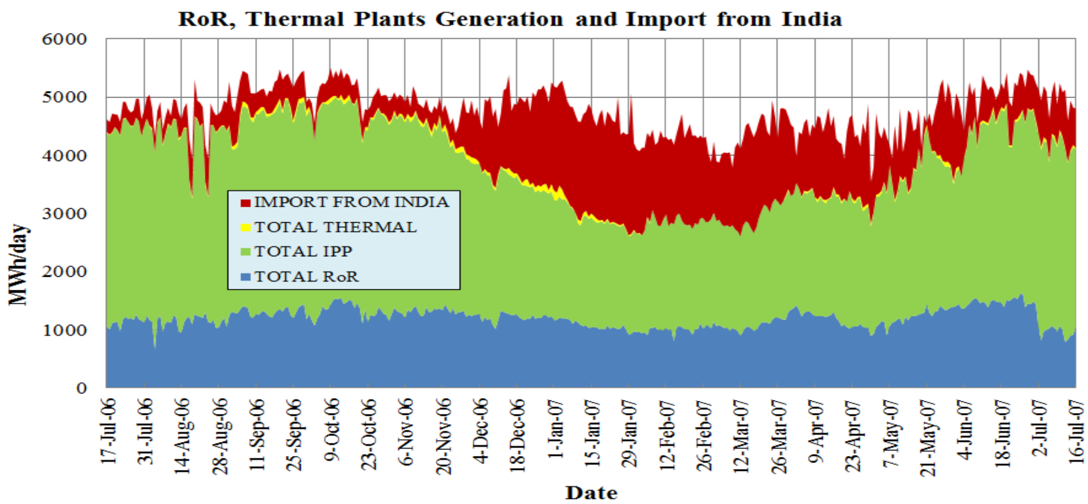


Figure 5-12: RoR NEA, IPPs, thermal daily generation and import from India

5.6.3 GENERATION FROM PROR PLANTS

Figure 5-13 presents the steps involved in development for the excel model of the operation of the PRoR (Kaligandaki A) hydropower plants. Total hourly generation from RoR of NEA, IPPs, thermal plants and import from India is compared with the hourly system load demand. If the system load demand is more than the RoR generation, the PRoR plants will be operated to meet the demand. All the PRoR calculation are done in 'PRoR model' excel file. The steps involved in development of the PRoR model are described below.

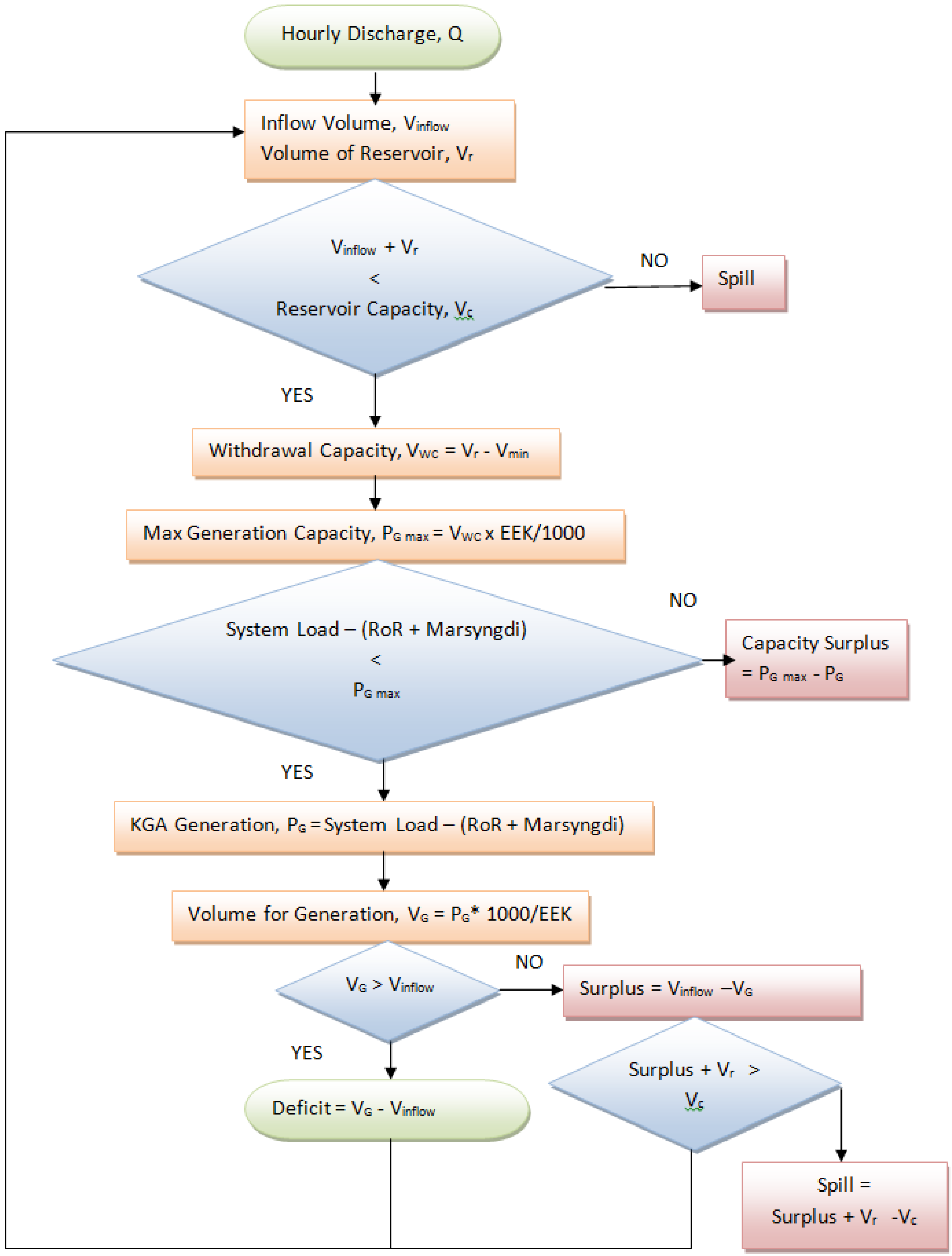


Figure 5-13: Steps involved for the operation of PRoR plants

Generation Capacity of Marsyangdi Hydropower Plant

The hourly discharge data was collected from the headworks site of the Kaligandaki A power plant. The hydrograph is shown in Figure 5-14. The hourly discharge is used to calculate RoR generation capacity of the power plant using equation 5-4. The calculated generation capacity is used to calculate the generation capacity of the Marsyangdi power plant using Equation 5-5.

$P_K=9.81x\eta xQxH$ Equation 5-4

$P_M=P_K\times\frac{P_{MC}}{P_{KC}}$ Equation 5-5

Where,

P_K : Generation capacity of Kaligandaki A, KW

η: Efficiency of power plant

Q : Discharge, m^3/s

H : Net head, m

P_M : Generation capacity of Marsyangdi, KW

P_{MC} : Installed capacity of Marsyangdi, KW

P_{KC} : Installed capacity of Kaligandaki A, KW

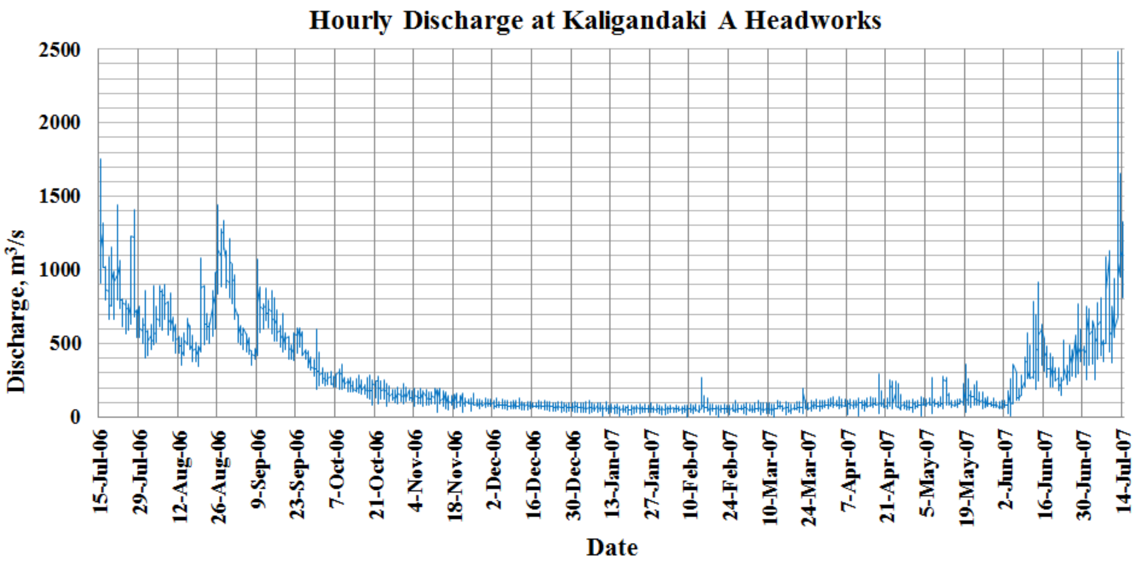


Figure 5-14: Hourly discharge at Kaligandaki A headworks

Kaligandaki A Reservoir Operation

The water level of the Kaligandaki A reservoir is calculated according to the inflow and generation volume based on the reservoir capacity curve. The reservoir capacity curve is shown in Figure 5-15. The calculation process of the generation from Kaligandaki A and the reservoir levels is described step by step below:

1. Reservoir volume is calculated according to the inflow volume and one hour earlier reservoir volume using the following equation

$V_{r(t)} = V_{r(t-1)} + V_{inflow}$ Equation 5-6

Where,

$V_{r(t)}$: Volume of reservoir, m^3

$V_{r(t-1)}$: Volume of reservoir one hour earlier, m^3

$V_{inflow(t)}$: Inflow volume to reservoir = Q x 3600

Q : Discharge m^3/s

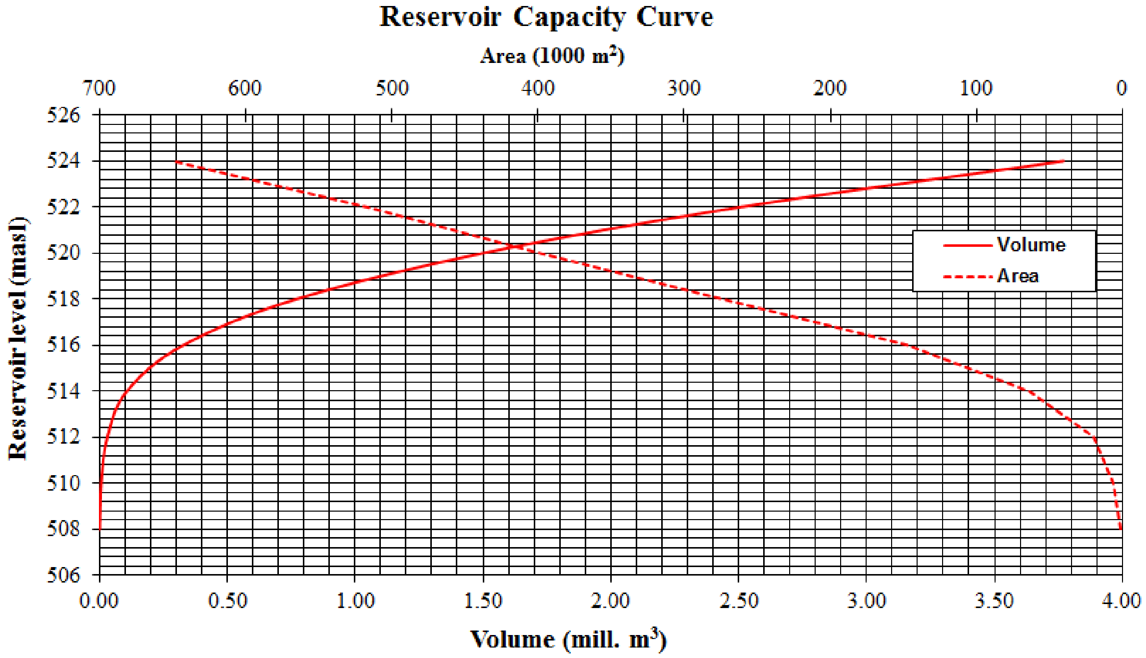


Figure 5-15: Reservoir capacity curve of Kaligandaki A based on the bathymetric survey 2010 (see Chapter 3.3.1)

2. The calculated reservoir volume is compared with the maximum reservoir capacity including the volume for the generation.
3. Maximum withdrawal capacity will be checked with the following equation:
   $V_{WC(t)} = V_{r(t)} - V_{min}$
   Where,
   $V_{WC(t)}$: Maximum withdrawal capacity from the reservoir, m^3
   $V_{r(t)}$: Volume of reservoir, m^3
   V_{min} : Dead volume of reservoir corresponding to minimum reservoir level, m^3
4. Maximum generation capacity is calculated according to the maximum withdrawal capacity with the following equations:

$P_{G\,max} = V_{WC(t)} \times EEK/1000$ Equation 5-7

$EEK = \eta \times \rho \times g \times H_n / 3600$ [kWh/m³] Equation 5-8

Where,

$P_{G\,max}$: Maximum generation capacity, MW

EEK : Energy equivalent, kWh/m³

η : Total efficiency

ρ : Density of water (= 1000 kg/m³)

g : Gravity constant (= 9.81 m/s²)

H_n : Net head, m

5. The calculated maximum generation capacity is compared with the system demand load and generation from RoR and Marsyangdi hydropower plant and the required generation from Kaligandaki A and capacity surplus is calculated according to System Load Demand (SLD) and total generation using the following equations:
   $P_{G(t)} = SLD_{(t)}$ - (RoR + Marsyangdi generation)$_{(t)}$
   Where,
   P_G : Required generation from Kaligandaki A, MW
   If the Maximum generation capacity is higher than the required generation from Kaligandaki A the capacity surplus will be the difference between them only if there is spilling from reservoir.
6. Then required water volume for generation is calculated as:
   $V_G = P_G \times 1000/EEK$
   If the volume of generation is greater than the inflow volume (V_{inflow}) the deficit volume (V_G-V_{inflow}) will be withdrawn from the reservoir. If inflow is higher than the generation volume, surplus volume (V_{inflow}-V_G) will be added to the reservoir. If the total volume in the reservoir including the surplus volume is greater than the reservoir capacity (V_c), the spilled volume is calculated as:
   Spilled volume $=V_r + (V_{inflow} - V_G) - V_c$, m^3 or
   Reservoir volume after the generation ($V_{r(t)}$) = Vr – (V_G - V_{inflow}), m^3
   According to the new reservoir volume, the reservoir level is determined according to the reservoir capacity curve (Figure 5-15).

These steps are repeated for each hour for calculation of generation from Kaligandaki A.

Generation Optimisation

The main objective of the generation optimisation is to avoid the spilling, maximise the generation and minimise the total load shedding hours and the gap between demand and supply during the peak hours. The optimisation is done using the solver programme. For optimisation, the first condition is to keep the reservoir level at the maximum level 524 masl at the beginning of peak hours to generate maximum during the peak hours and the second condition is to minimise the total load shedding hours in 24 hours. For the solver programme, the total load shedding hour is set as the objective cell to minimum, the load factor is set to the variable parameters which can be varied from 0 to 1 and the reservoir level is set as constraints to 524 masl at 16:00. Visual Basic programme is used for solving the optimisation for whole year. The programme optimises the generation and reservoir operation for each day (24 hours).

5.6.4 KULEKHANI RESERVOIR OPERATION

Daily reservoir level and generation from Kulekhani hydropower plant is shown in Figure 5-16. These levels and generation data are used to calculate the daily inflow to the Kulekhani reservoir according to the inflow and generation volume based on the reservoir capacity curve. Figure 5-17 shows the reservoir capacity curve. The daily discharge is assumed constant for the whole day and used to calculate the hourly inflow, generation and reservoir level. Total hourly generation from RoR and PRoR will be compared with the hourly system load demand. If the system load demand is more than total generation from the RoR and PRoR, the storage plants will be operated to meet the demand. All the storage calculation are done in 'Storage model' excel file. The steps involved in the development of the Storage model are calculation of daily discharge, hourly generation and the Kulekhani reservoir operation according to the hourly system load demand. Figure 5-18 presents the steps involved to calculate daily discharge from the daily generation and the water level of the Kulekhani reservoir.

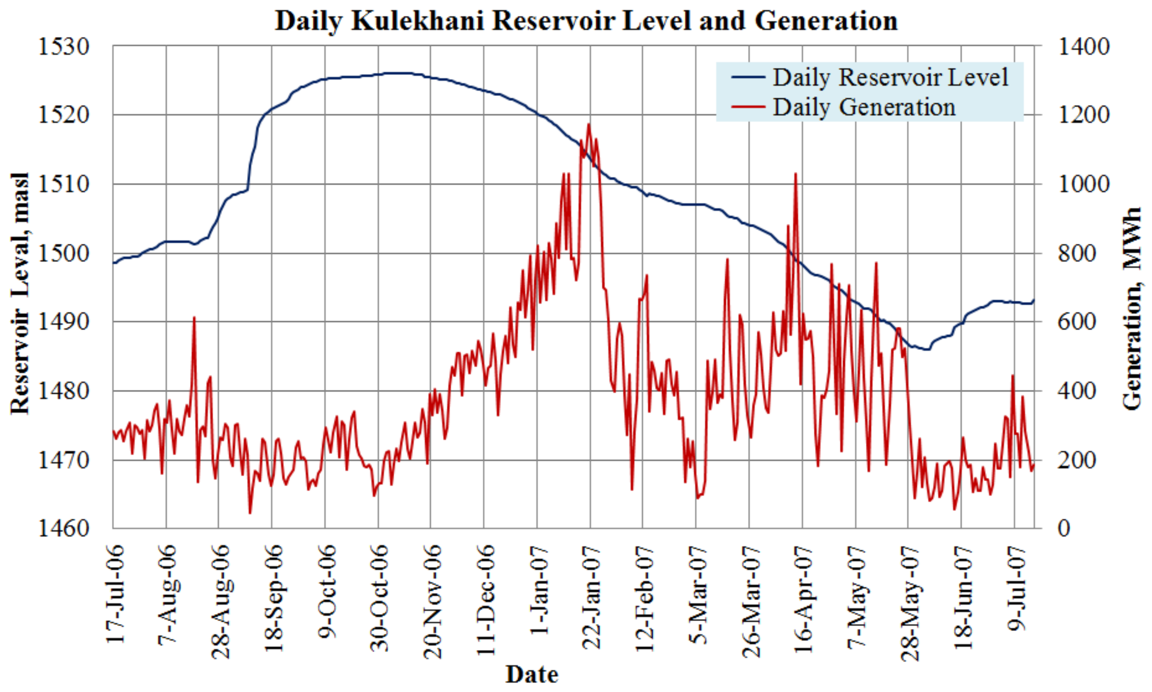


Figure 5-16: Daily reservoir level and generation from Kulekhani I

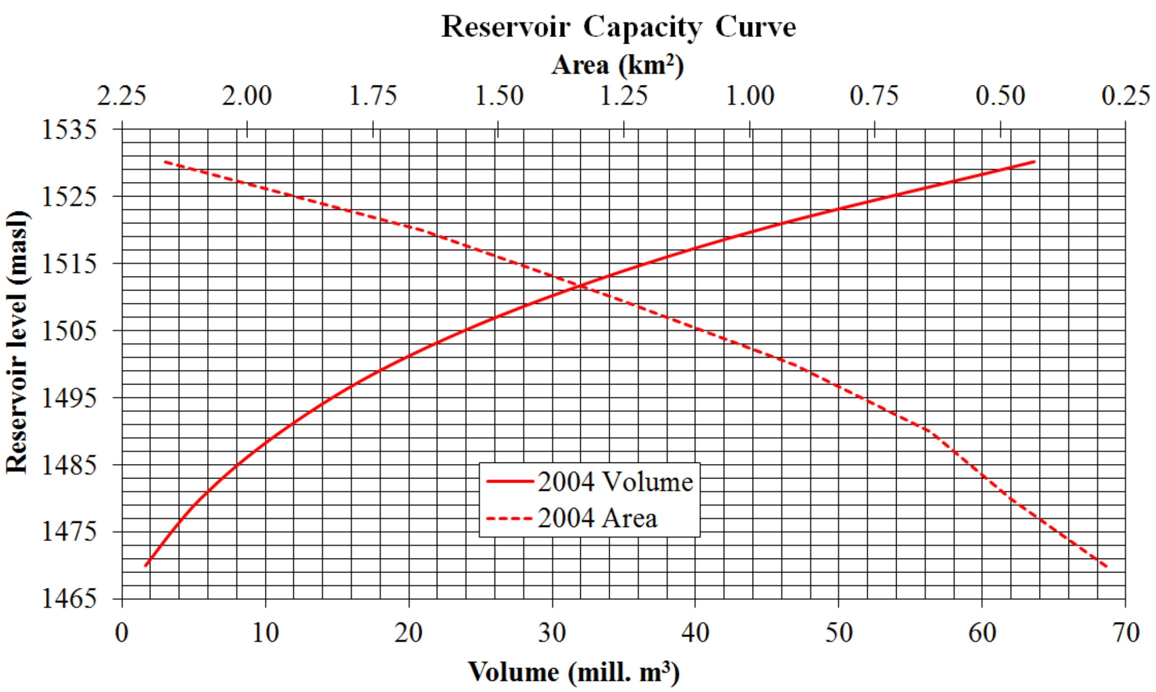


Figure 5-17: Reservoir capacity curve of Kulekhani reservoir based on 2004 bathymetric survey (see Chapter 4.2.3)

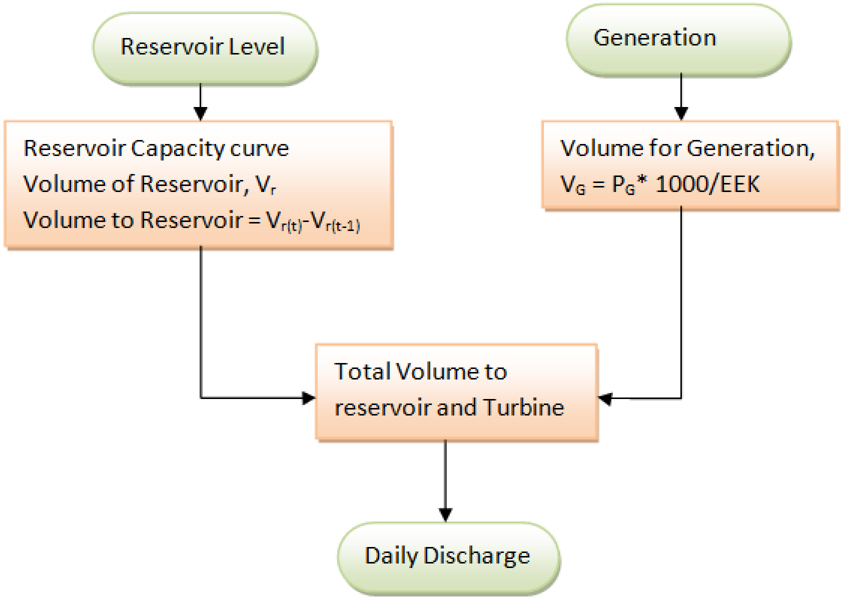


Figure 5-18: Steps to calculate daily discharge to the Kulekhani reservoir

Daily Inflow to Kulekhani Reservoir

The Daily inflow to the Kulekhani reservoir is calculated according to the daily reservoir level and the generation from Kulekhani I. The calculation process is described step by step below:

1. Reservoir volumes are obtained from the reservoir capacity curve (Figure 5-17) according to the reservoir level and volume to the reservoir at time t is calculated as:
   $\triangle V_{r(t)} = V_{rt} - V_{r(t-1)}$
   Where
   $\triangle V_{r(t)}$: Volume to reservoir at time t, m^3
   V_{rt} : Volume of reservoir at time t, m^3
   $V_{r(t-1)}$: Volume of reservoir at time t-1, m^3
2. Then water volume used for generation is calculated as:
   $V_{G(t)} = P_{G(t)}x1000/EEK$
   Where,
   $V_{G(t)}$: Water volume used for generation, m^3
   $P_{G(t)}$: Generation, MW
   EEK: Energy equivalent, kWh/m^3
3. Then total inflow to the reservoir is calculated as:
   $Q_t = \frac{V_{G(t)+}\Delta V_{r(t)}}{24\times60\times60}$

The calculated inflow is presented in Figure 5-19.

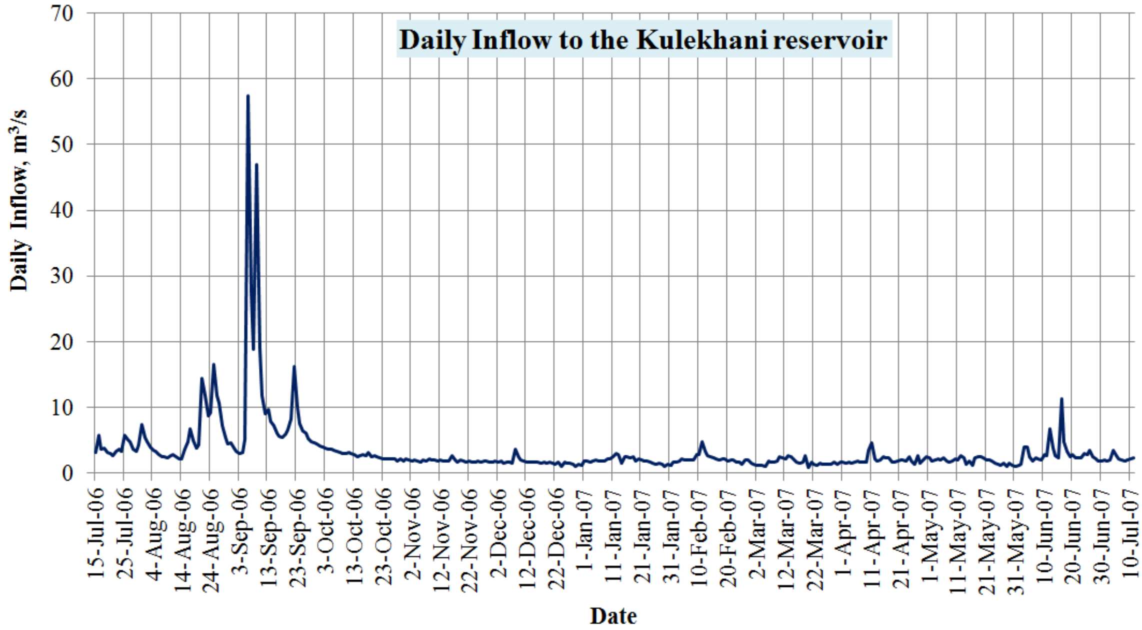


Figure 5-19: Daily inflow to Kulekhani reservoir

Reservoir Operation

The water level of the Kulekhani reservoir is calculated according to the inflow and generation volume based on the reservoir capacity curve. The reservoir capacity curve is shown in Figure 5-17. Figure 5-20 present the steps involved in development for the excel model of the operation of the Kulekhani reservoir.

The calculation process is similar to the calculation of the generation from Kaligandaki A. The process is described step by step below:

1. Reservoir volume at time step t is calculated according to the inflow volume and resrvoir volume at time step t-1 using the following equation

$V_{r(t)} = V_{r(t-1)} + V_{inflow}$

Where,

$V_{r(t)}$:	Volume of reservoir at time step t, m^3
$V_{r(t-1)}$:	Volume of reservoir at time step t-1, m^3
$V_{inflow(t)}$:	Inflow volume to reservoir at time step t = Q_t x 3600
Q_t:	Discharge at time step t in m^3/s

2. Maximum withdrawal capacity will be checked with following equation:

$V_{WC(t)} = V_{r(t)} - V_{min}$

Where,

$V_{WC(t)}$:	Maximum withdrawal capacity from the reservoir at time step t, m^3
$V_{r(t)}$:	Volume of reservoir at time step t, m^3
V_{min}:	Dead volume of reservoir corresponding to minimum reservoir level, m^3

3. Maximum generation capacity is calculated according to the maximum withdrawal capacity with following equations:

$P_{G\,max} = V_{WC(t)} \text{ x } EEK/1000$

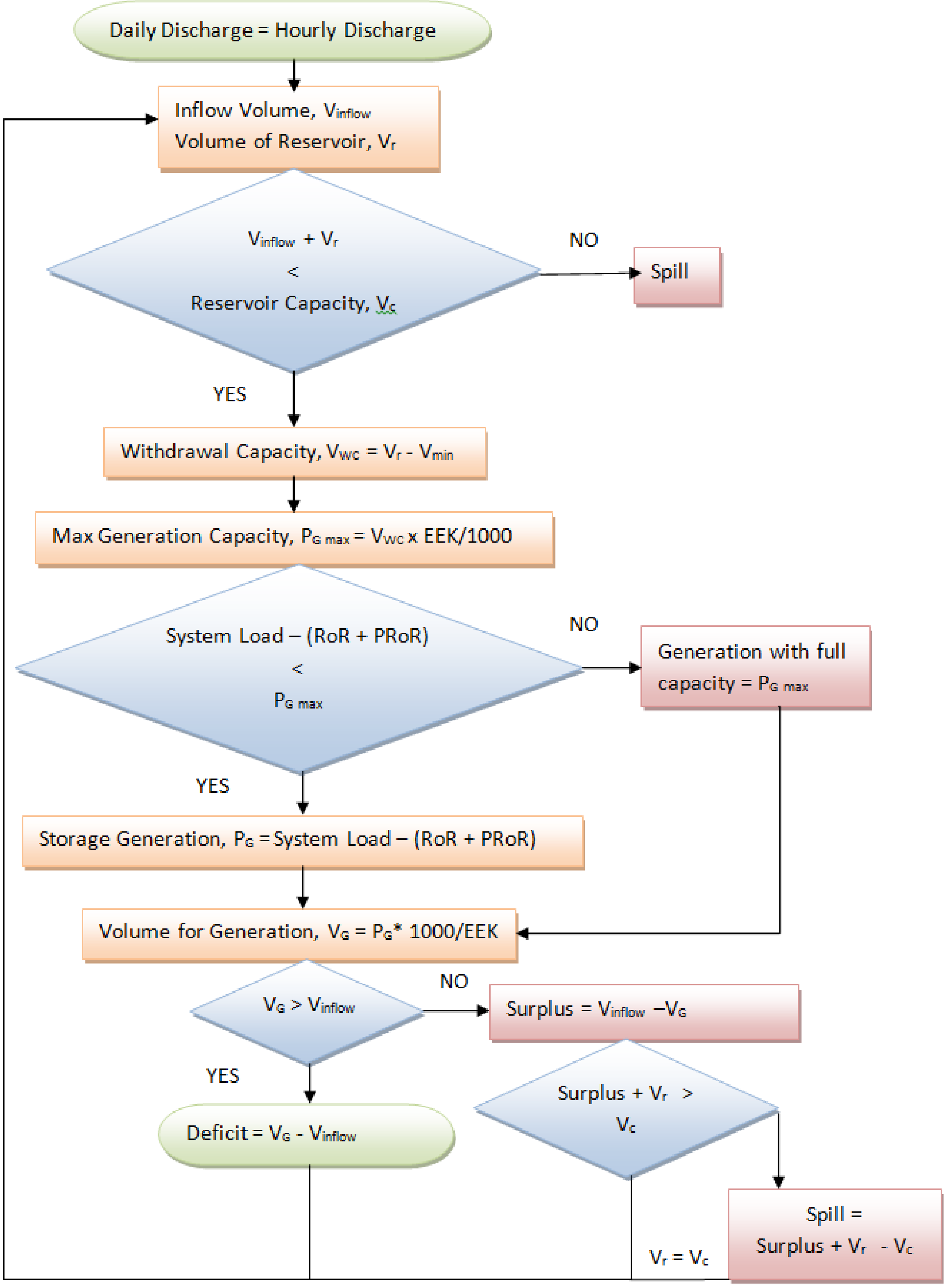


Figure 5-20: Steps involved in development of Kulekhani reservoir operation model

4. The calculated maximum generation capacity is compared with the System demand load and generation from RoR and PRoR plants and the required generation from Kulekhani I and II is calculated according to System Load Demand (SLD) and total generation from RoR and PRoR using following equations:
   $P_{G(t)} = SLD_{(t)} - (RoR + PRoR)_{(t)}$
   Where,
   $P_{G(t)}$: Required generation from Kulekhani hydropower plants at time step t, MW
5. Then required water volume for generation is calculated as:
   $V_{G(t)} = P_{G(t)}$ x 1000/EEK
   If the volume of generation is greater than the inflow volume (V_{inflow}) the deficit volume (V_G-V_{inflow}) will be withdrawn from the reservoir. If inflow is higher than the Generation volume, surplus volume (V_{inflow}-V_G) will be added to the reservoir. If the total volume in the reservoir including the surplus volume is greater than the reservoir capacity (V_c) the spilled volume is calculated as:
   Spilled volume $=V_r + (V_{inflow} - V_G) - V_c$, m^3 or
   Reservoir volume after the generation ($V_{r(t)}$) = $Vr – (V_G - V_{inflow})$, m^3
   According to the new reservoir volume, the reservoir level is determined according to the reservoir capacity curve (Figure 5-17).

These steps are repeated for each hour.

5.6.5 Output and Discussion

Summary

All the calculations are summarised in 'Summary' excel file. Figure 5-21 presents the actual and modeled annual generation from PRoR and storage, load shedding and spilled energy. It shows that the quantity of load shedding could be reduced by 65%. Figure 5-22 and Figure 5-23 show the actual and alternative daily energy with surplus and load shedding. These figures show that the load shedding could be totally avoided during the wet period and could be reduced significantly during the dry period. Figure 5-24 shows the actual and modeled load shedding on daily basis. It shows that the load shedding could be avoided during the wet season (from July to first week of November) and reduced by 80% during the dry season (from November to March). Daily spilled energy can also be reduced significantly as shown in Figure 5-25.

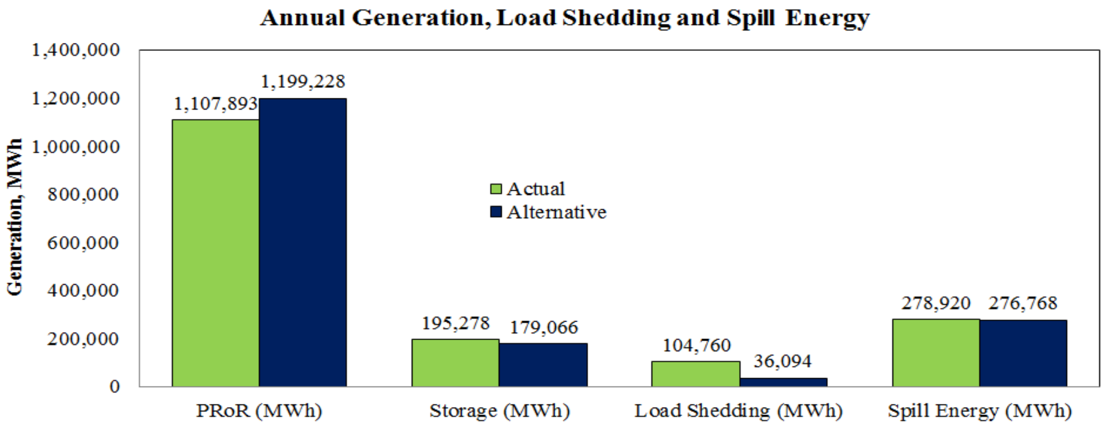


Figure 5-21: Annual generation, load shedding and spilled energy.

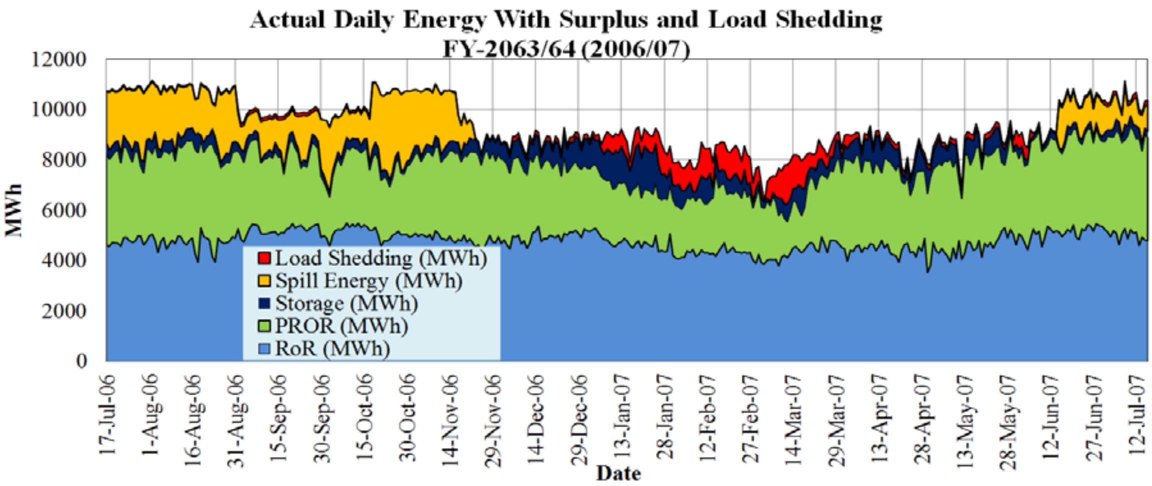


Figure 5-22: Actual daily energy with surplus and load shedding

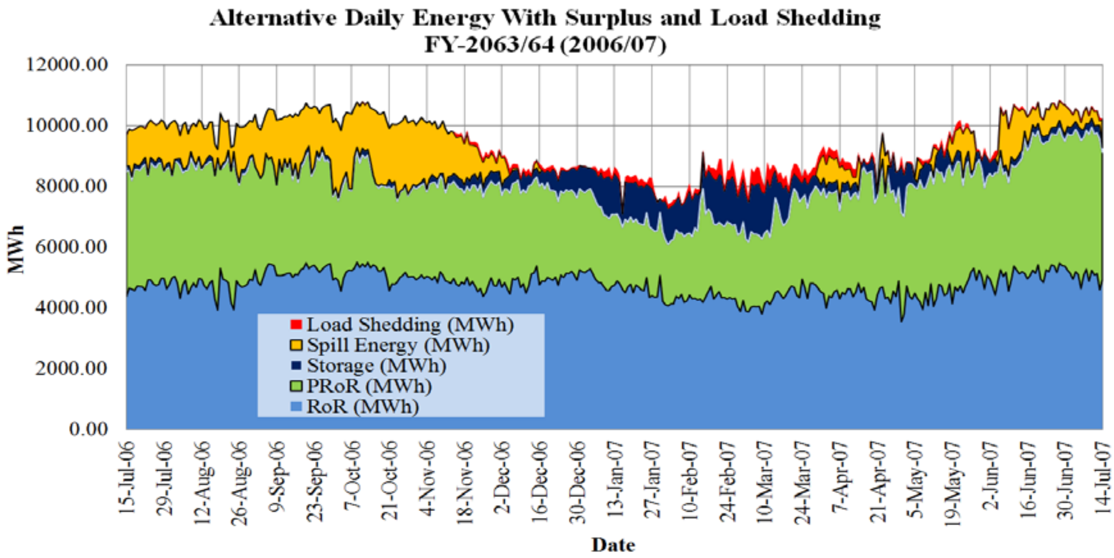


Figure 5-23: Alternative daily energy with surplus and load shedding

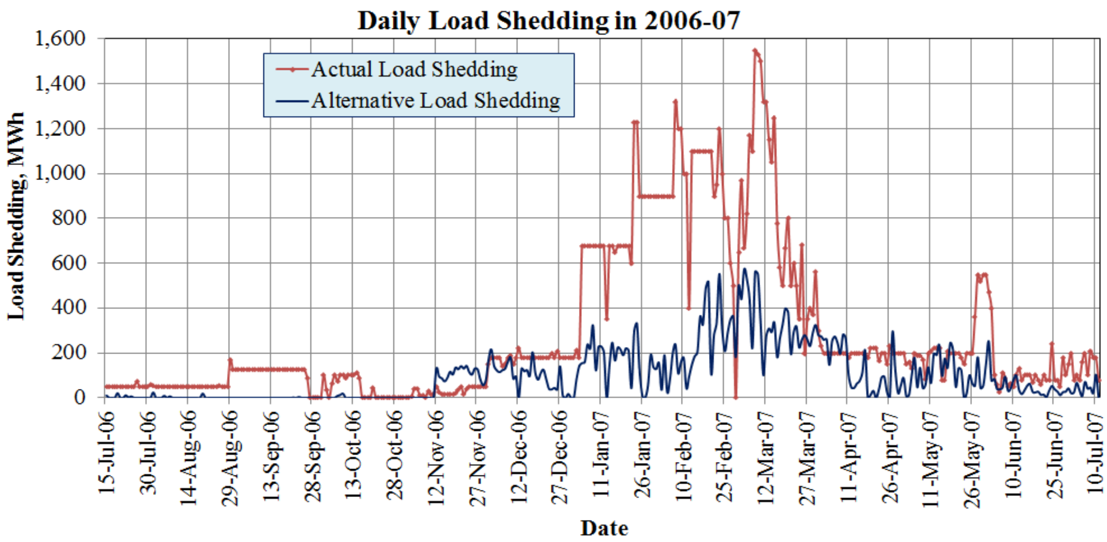


Figure 5-24: Daily load shedding

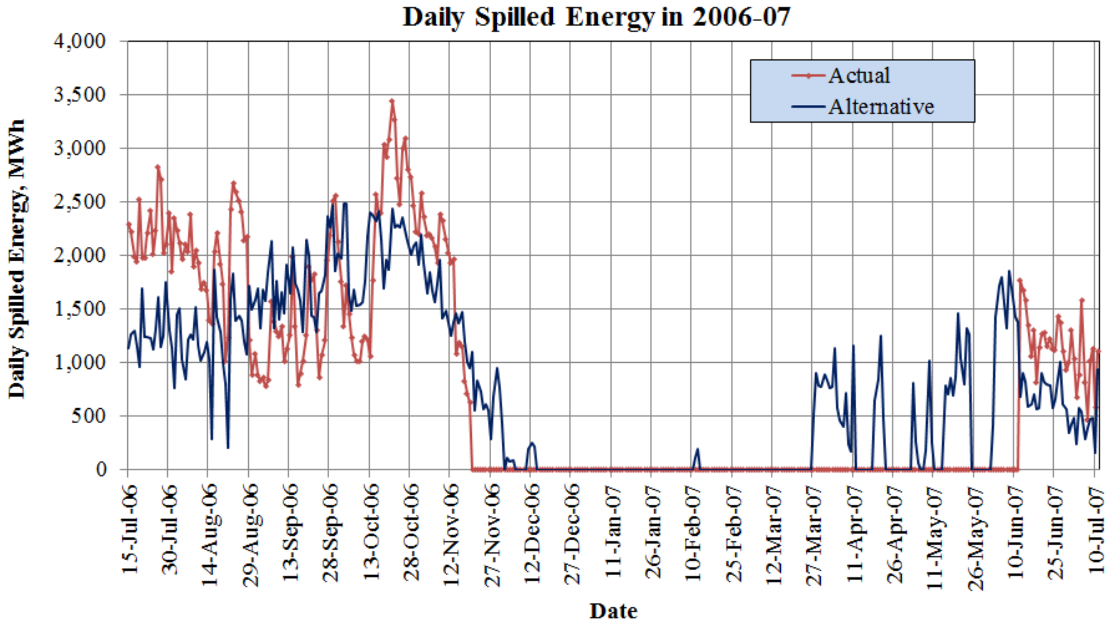


Figure 5-25: Daily spilled energy

Hourly System Load Curve

Actual and alternative system load curves for February 15, 2007 (Falgun 3, 2063) are shown in Figure 5-26 and Figure 5-27, respectively. They show that the load shedding could be limited to 260 MWh from 1,180 MWh if power plants were operated according to the developed model. The actual and alternative system load curves are presented in Appendix E.

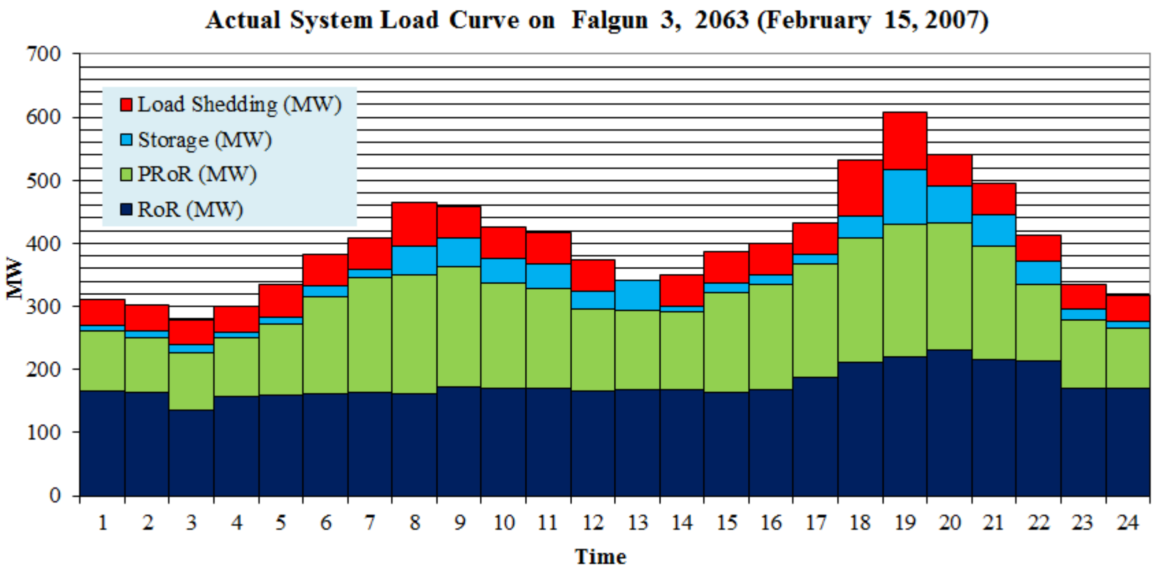


Figure 5-26: Actual system load curve on February 15, 2007 (modified from NEA, 2007)

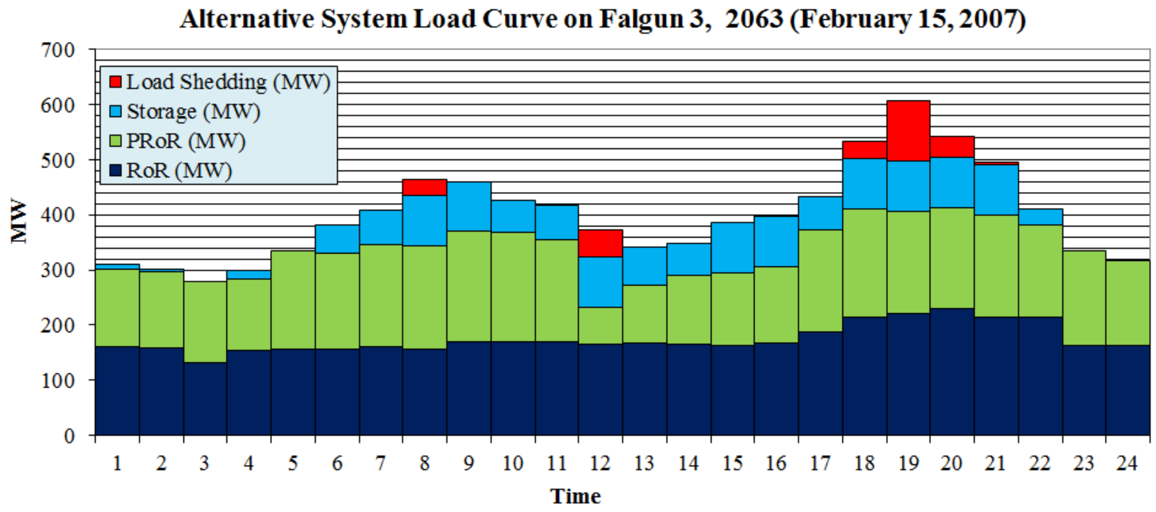


Figure 5-27: Alternative system load curve on February 15, 2007

Generation from PRoR

Figure 5-21 shows that the PRoR generation could be increased by 91 335 MWh in a year (2006/07) by modeled alternative operation of PRoR which is significant for the power system of Nepal and could reduce the load shedding significantly. Figure 5-28 presents the daily generation from PRoR hydropower plants. It shows that generation during the dry period is similar but during the wet season PRoR could generate more.

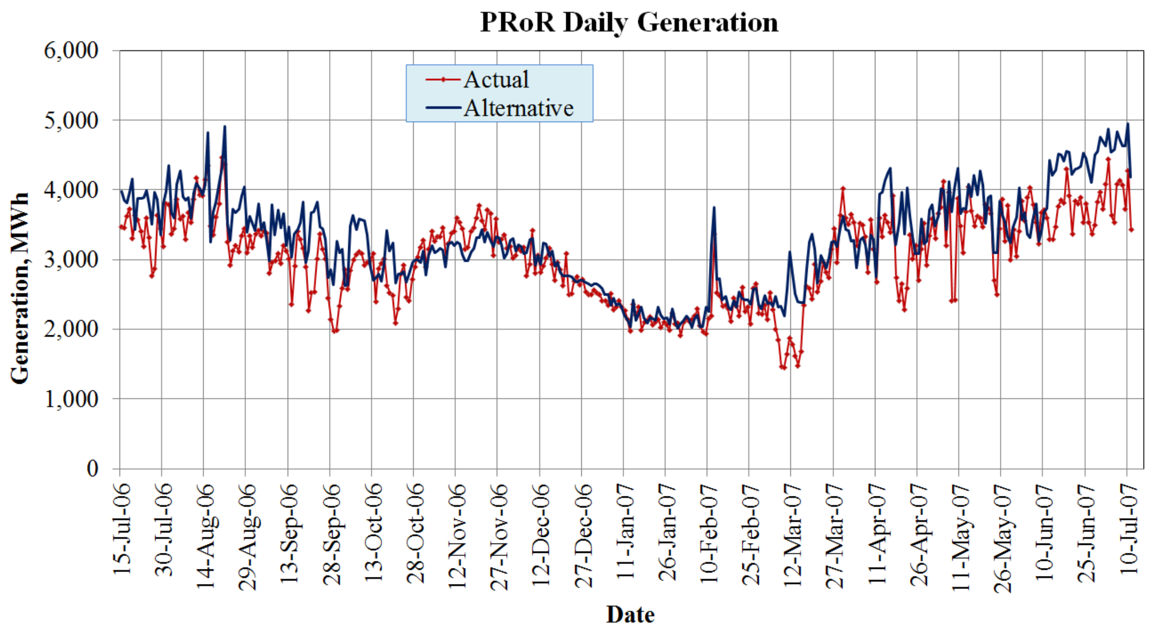


Figure 5-28: PRoR daily generation

Kulekhani Reservoir Operation

Figure 5-29 presents the actual and alternative reservoir operation curves. It shows that the Kulekhani reservoir could be operated at higher reservoir levels in whole year than the actual operated level. Figure 5-30 presents daily generation and spilled water from Kulekhani reservoir. It shows that the Kulekhani hydropower plants could generate more during the dry season and less during wet. Though, about 7.13 million m^3 could be spilled still the load shedding is 65% less. The spilled water can be used to remove deposits from the reservoir for its sustainability. Wet and Dry generation from the Kulekhani power plants are shown in Figure 5-31. It shows that Kulekhani power plants could generate 24% more and the load shedding could be reduced to 24,062 MWh from 77,464 MWh during dry season from December to March.

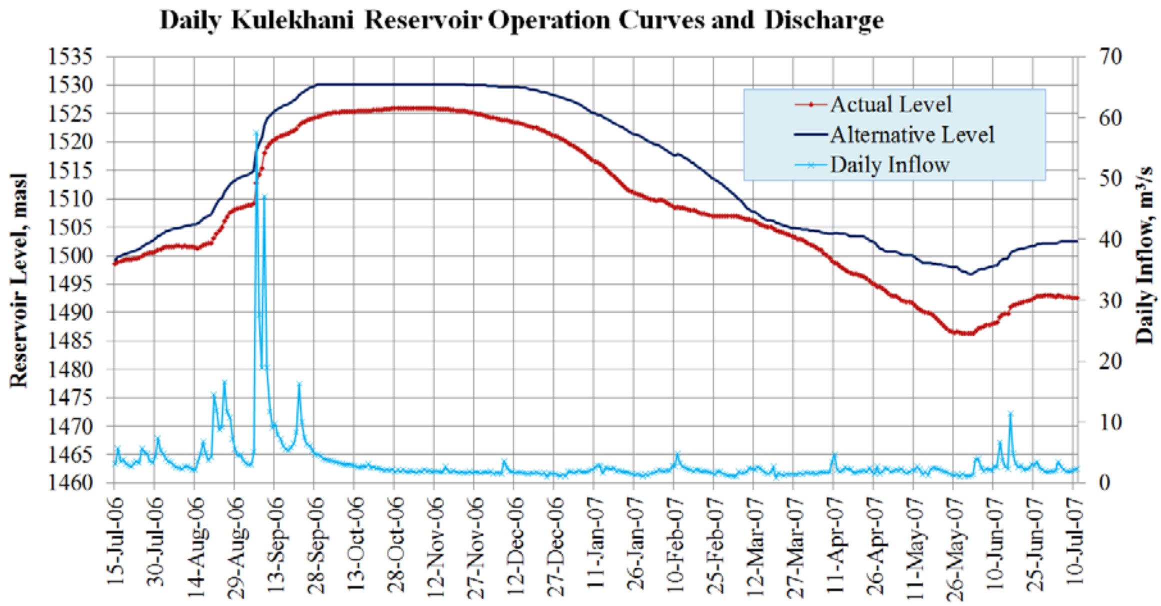


Figure 5-29: Daily inflow and operation curve of the Kulekhani reservoir

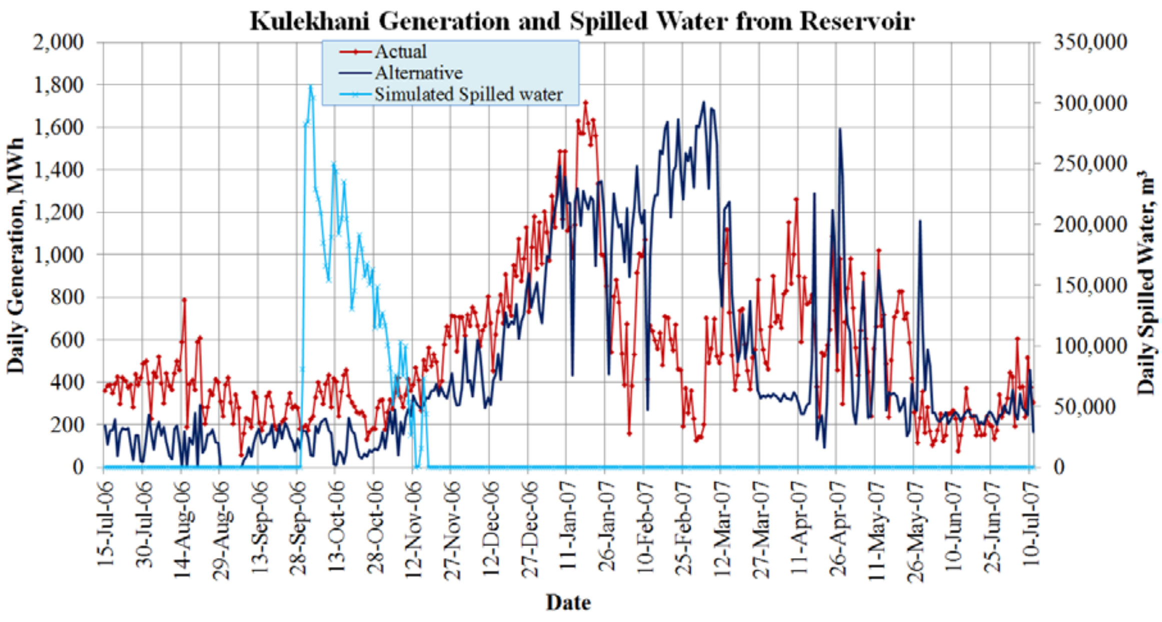


Figure 5-30: Generation and spilled water volume from Kulekhani reservoir

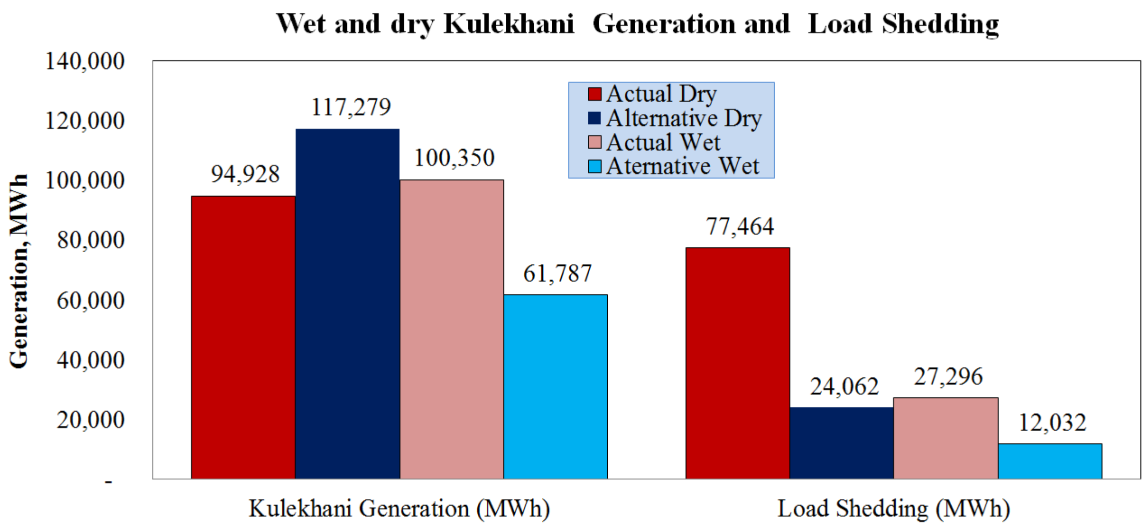


Figure 5-31: Kulekhani generation and load shedding in the wet and dry seasons

Discussion

The model focuses on water availability, generation and storage capacities and the power demand. The transmission part of the system is not considered. However, there are several major constraints in the transmission system. System Operation Department of NEA (2007) reported that Baktapur-Lamosanghu, Bardhghat-Bharatpur, Bharatpur-Hetauda, Lekhnath-Damauli-Bharatpur 132 kV line are the major constraints in the transmission system. It also reported that the capacity of Hetauda–Birganj Transmission line and 132/66kV transformers at Hetauda was a major constraint in the transmission system that compelled continuous load shedding in Birgunj corridor and forced operation of Kulekhani power plants.

System Operation Department, also known as Load Dispatch Centre (LDC) operates the Integrated Nepal Power System (INPS) using the computer based Supervisory Control and Data Acquisition (SCADA) system. The main responsibility of LDC is to make real time decisions to ensure security, economy and quality of the supply. They instruct power plants to generate according to the demand on INPS. Therefore, LDC plays the main role in optimum use of the power plants of Nepal.

The large amount of load shedding was due to capacity deficiency during October-November and March-April. Development of PRoR instead of RoR helps to reduce the load shedding during this time.

The power plants of NEA could not generate with its full capacity due to technical deficits at several power plants, operation & maintenance problems, management constrain and unavailability of spare parts. These problems should be addressed to achieve the improved performance of the power plants. In addition, the following measures are recommended:

- The electro-mechanical parts were damaged by abrasion and erosion during the monsoon due to poor trap efficiency of the settling basins. The most of the power plants were shut down due to repair and maintenance of electro-mechanical parts. Therefore, performance of the settling basins should be improved in some of the major power plants e.g. Kaligandaki A, Middle Marsyangdi and Marsyangdi power plants.

- Field Monitoring Report of NEA Power Plants 2068 (MoEN, 2012) reported that Unit 1 of the Kulekhani power plant stopped to generate for 9,024 hours in 2064 (2007/2008) for repair and maintenance. Similarly, Unit 2 was also stopped for 1,464 hours in 2066 (2009/2010) for the same purpose. Therefore, proper schedule of repair and maintenance and periodic overhauling should be prepared and implemented.
- Prepare the repair and maintenance schedule and periodic overhauling plan properly. The efficiency of turbine is significantly reduced due to severe damage of turbine parts during the monsoon. Therefore, the repair and maintenance of the electro-mechanical parts should be done immediately after monsoon so that the improved efficiency could be available for the entire dry season. If the repair and maintenance were done before monsoon, efficiency of turbines would have been lost during the monsoon.
- Kaligandaki A, Marshyangdi hydropower and some other power plants are losing some generation due to sediment flushing from the settling basins. They have conventional flushing system and power plants are shut down during the sediment flushing from settling basins. Therefore, possibilities of installation of sediment flushing system should be worked out for sediment flushing while the settling basins are in operation.
- As presented in Chapters 3 and 4 the reservoirs of Middle Marshyangdi and Kulekhani are losing their storage capacity due to reservoir sedimentation. Similarly, Sunkoshi, Trishuli and Chilime power plants are losing the storage capacity of the peaking ponds. Therefore, appropriate sediment management technique should be adopted to control the sediment deposition in reservoirs and peaking ponds. It will increase the power generation during the peak hours.
- Sufficient budget should be allocated for operation and maintenance of power plants.

In addition, incentives should be introduced to the staff of System Operation Department (SOD) and power plants. It will encourage the staff to generate more power during the dry season and peak hours, which will reduce the load shedding and the gap between demand and supply.

5.7 CONCLUSION AND RECOMMENDATION

Kulekhani power plant is the only storage power plant in Nepal. Therefore, capacity and energy drops in the dry season due to the shortage of available runoff, moreover, the fate of load shedding hours in the dry season depends largely on the availability of water in the Kulekhani reservoir.

The output of the model shows that the load shedding could be totally avoided during the wet period and could be reduced significantly during the dry period in 2006/07. The Kulekhani reservoir could be operated at higher reservoir levels. The water level could reach 1530 masl at the end of September and 7.13 million m^3 water could be spilled in 2006/07. The spilled water could be used for sediment removal from the reservoir.

The large amount of load shedding was due to capacity deficiency during October-November and March-April. Development of PRoR instead of RoR helps to reduce the load shedding during this time.

Enhanced Performance Reward for staffs will encourage the generation of more power during the dry season and peak hours. If power plants are operated properly with coordination of proper power evacuation system, production of energy can be increased and plant can be operated with optimum benefit hence load shedding could be reduced significantly.

The daily system load is prepared from the available system load and the daily peak load. The hourly generation data are prepared from the daily generation data and daily system load, thus the uncertainties in the model is daily system load and the hourly generation data from the RoR hydropower plants and Kulekhani hydropower plants. To avoid the uncertainties in the model more correct data should be collected as explained below:

- Generation data: the hourly generation data should be recorded in all power plants
- System load data: The system demand should be recorded each hour according to the actual demand on location basis. If total demand could not be supplied, the quantity of load shedding is estimated according to the supply of preceding day.
- Hydrological data: the discharge at the headworks should be measured at least six times in a day and to be calculated for each hour.
- Reservoir operation data: The reservoir level, discharge for the generation and inflow to the reservoir should be recorded each hour.

6 SUSTAINABILITY REVIEW OF THE KULEKHANI RESERVOIR BY THE RESCON METHOD

6.1 RESCON MODEL

RESCON (REServoir CONservation) is a World Bank sponsored project. It was developed in order to make preliminary decisions for policy makers for the assessment and promotion of sustainable management of reservoirs, with special emphasis on the economic evaluation of sediment management and the promotion of sustainable development. Brief description sediment management is presented in Appendix A. The model considers four main options of sediment management, namely: flushing, hydrosuction, traditional dredging and trucking. In addition, a "do-nothing" alternative, (i.e., no sediment removal) where eventual decommissioning will be required, is also analyzed. The program may be used for existing dams as well as proposed dams. Economic optimization results determine which sediment management technique is the most viable. In addition, sustainability of the evacuation methods is identified by the programme.

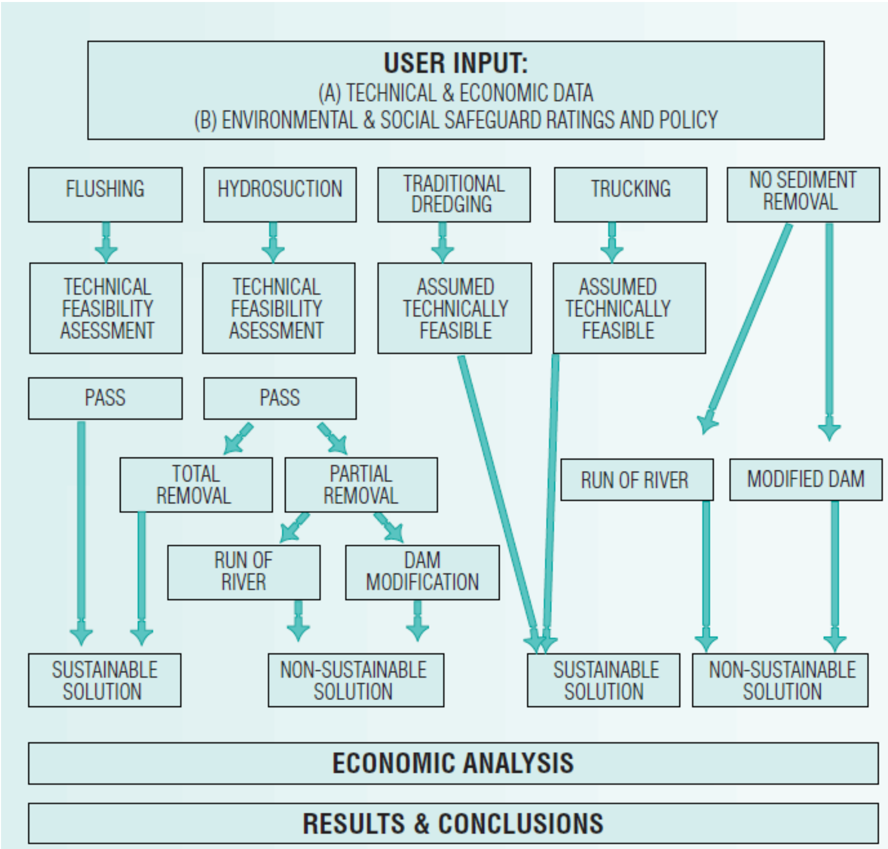


Figure 6-1: RESCON programme structure

RESCON is a spreadsheet-based program written in Visual Basic programming language and works with macros. Figure 6-1 illustrates the main steps involved in this model. The model requires project-specific technical and economic data in addition to environmental and social safeguards parameters. There are two sheets to enter the required data. Seven classes of data should be entered into the first sheet such as: reservoir geometry, water characteristics, sediment characteristics, removal parameters, economic parameters, flushing benefits parameters and capital investment. In the second sheet, strategy information about environmental and social safeguard policies is provided for the selection of a desirable sediment management.

6.2 APPLICATION OF RESCON MODEL FOR KULEKHANI RESERVOIR

6.2.1 INPUT PARAMETERS

There is not any structural facility at the Kulekhani dam to control water levels and outflows for sediment flushing. However, two diversion tunnels were constructed in the left bank abutment of the main dam. The inlets are already buried in sediment deposits. According to Sangroula (2005) one of the diversion tunnels can be reopened and used for sediment flushing with construction of appropriate intake and engineering facilities to control water levels and outflows. If the diversion tunnel can not be utilised, a new tunnel can be constructed along the left bank of the Kulekhani dam.

Sangroula (2005) determined the characteristics of sediment in laboratory experiments by taking different core samples from the Kulekhani reservoir. As a result of analyses, Sangroula (2005) reported that the average D_{50} is equal to 0.04 mm and that of D_{90} is 0.35 mm. The average density of sediment particles was 2.57 tons/m^3.

One of the Sediment Characteristics (ψ, multiplier in the Tsinghua University method for sediment loads prediction in flushing flow) is taken as 180 due to the relatively low flushing discharge (less than 50 m^3/s).

The Kulekhani dam was constructed in 1983. The original capacity of the reservoir was 85.3 mill. m^3. The current capacity is about 64 mill. m^3. About 21 mill. m^3 of sediment is already deposited in the reservoir over 28 years. It gives an average annual loss of about 0.76 mill. m^3/year. Considering dry density of the settled sediment as 1.3 ton/m^3, mean annual sediment deposit is about 1.0 mill. ton.

The amount of electricity produced from each m^3 (energy equivalent) of the power plant is computed as:

$EE = \eta \times \rho \times g \times \frac{\sum head}{3600}$, in kWh/m^3 (see Chapter 5 for definition of parameters)

If the overall efficiency is taken as 85% and the head of the Kulekhani I is 550 m and 284 m for the Kulekhani II, the energy equivalent is 1.93 kWh/m^3. Average Power Purchase Agreement (PPA) rate during winter is 8.40 NRs/kWh in year 2011/12. Multifuel Power plant and Diesel power plants are generating more than 9 GWh every year (NEA 2011). The cost of generation from these plants is more than 20 NRs/kWh. Therefore, the economic value of the energy can be assumed more than the PPA rate as the value of the energy produced by the Kulekhani hydropower plants is very high due to the 16 hours load shedding per day during the winter. In the worst case, if the energy rate is assumed 10 NRs/kWh, the unit value of the water will be 19.3 NRs/m^3. This is equivalent to 0.25 USD/m^3. If generation from Kulekhani III and other economic impacts from load shedding are considered, the energy rate will be higher than 10 NRs/kWh.

Based on the values discussed above, the main parameters used in RESCON for the Kulekhani reservoir are presented in Table 6-1.

Table 6-1: The main input parameters used in RESCON for the Kulekhani reservoir

Symbol	Units	Description	Value
Reservoir Geometry			
S_o	(m^3)	Original (pre-impoundment) capacity of the reservoir	85,290,000
S_e	(m^3)	Existing storage capacity of the reservoir	64,000,000
W_{bot}	(m)	Representative bottom width for the reservoir	30.0
EL_{max}	(m)	Elevation of top water level in reservoir	1,530.0
EL_{min}	(m)	Minimum bed elevation	1,460.0
EL_f	(m)	Water elevation at dam during flushing	1,485
L	(m)	Reservoir length at the normal pool elevation	7000
h	(m)	Available head--reservoir normal elevation minus river bed downstream of dam	110.0
Water Characteristics			
V_{in}	(m^3)	Mean annual reservoir inflow (mean annual runoff)	137,040,000
T	(^oC)	Representative reservoir water temperature	15.0
Sediment Characteristics			
ρ_d	(t/m^3)	Density of in-situ reservoir sediment, Typical values range between 0.9 - 1.35	1.30
M_{in}	(t)	Mean annual sediment inflow mass.	1,000,000
Removal Parameters			
Q_f	(m^3/s)	Representative flushing discharge	20
T_f	(days)	Duration of flushing after complete drawdown	4
N	(years)	Frequency of flushing events	1
Cw	(%)	Concentration by weight of sediment removed to water removed by traditional dredging	30
Economic Parameters			
c	(USD/m^3)	Unit cost of construction	1.14
r	decimal	Discount rate	0.1
Mr	decimal	Market interest rate	0.12
P1	(USD/m^3)	Unit benefit of reservoir yield	0.25
omc	decimal	Operation and maintenance coefficient	0.1
Capital Investment			
FI	USD	Cost of capital investment required for implementing flushing measures.	3,500,000
HI	USD	Cost of capital investment to install (HSRS)	3,044,000
DU	Years	The expected life of HSRS	10

6.2.2 EVALUATION OF RESCON RESULTS

Flushing Results

The flushing feasibility criteria in RESCON are calculated according to the criteria developed by Atkinson (1996). The calculated flushing feasibility criteria and guidelines are presented in Table 6-2. The RESCON analysis shows that the calculated Sediment Balance Ratio (SBR) is 1.65 and the Long-Term Capacity Ratio (LTCR) is 0.5, which are more than the recommended values. Therefore, flushing is a technically feasible solution because annual volume of sediment flushed from the reservoir is larger than the annual inflow of sediment in average.

Table 6-2: Flushing feasibility criteria and guidelines

Criterion	**Required**	**Calculated**	**Notes**
SBR	> 1	1.65	Can be flushed if > 1, otherwise not.
LTCR	preferably > 0.35	0.50	Use caution if < 0.35.

HSRS Results

The hydrosuction technical model is based on Hotchkiss and Huang (1995). The RESCON results of the feasibility of Hyrdosuction Sediment-Removal Systems (HSRS) are presented in Table 6-3. The results show that HSRS is technically feasible to have sustainable solution because annual volume (1.07 mill. metric tons/year) of sediment flushed from the reservoir is higher than annual inflow (1.00 mill. metric tons/year) of sediment.

Table 6-3: HSRS feasibility results of RESCON

Sediment Transport Rate, Qs = 0.03 (m^3/s) = 3,189 (tons/day) = 1.067 (mill. tons/year)
Reservoir Volume Restored = 2,450 (m^3/day) = 0.90 mill. m^3/year
Mixture Velocity, V_m = 3.7 (m/s)
Mixture Flowrate, Q_m = 2.45 (m^3/s)
Sediment Concentration through Hydrosuction Pipe, C = 5,040 (ppm)

Economic Results & Conclusions

As a result of economical optimizations RESCON gives information about sustainable and nonsustainable solutions, the aggregated net present value and strategy yielding the highest aggregate net benefit. Table 6-4 presents the summary of the economical results of the RESCON analysis. It shows that sustainable solution can be obtained for all of the strategies.

Table 6-4: Summary of economic results of RESCON

Possible strategies	**Technique**	**Aggregate net present value (mill. USD)**
Do nothing	Not applicable	245.28
Nonsustainable (Decommissioning) with partial removal	HSRS	Partial removal with HSRS is technically infeasible, see total removal with HSRS
Nonsustainable (Run-of-River) with no removal	Not applicable	245.28
Nonsustainable (Run-of-River) with partial removal	HSRS	Partial removal with HSRS is technically infeasible, see total removal with HSRS
Sustainable	Flushing	244.70
Sustainable	HSRS	249.77
Sustainable	Dredging	244.76
Sustainable	Trucking	243.28

Nonsustainable solution with partial removal using HSRS is technically infeasible. Aggregate Net Present Value is the discounted value of the money, which can be gained from this reservoir over entire life of the dam.

Information on economic conclusion is given in Table 6-5. This information includes whether the strategy is sustainable or nonsustainable, name of the strategy with the highest aggregate net benefit and the highest aggregate net benefit. As a result of the optimization, the highest net aggregate benefit, 249.8 mill. USD is achieved by using HSRS technique.

Table 6-5: Economic conclusion of RESCON

Strategy yielding the highest aggregate net benefit:	Sustainable
Technique yielding the highest aggregate net benefit:	HSRS
The highest aggregate net benefit: mill. USD	249.77

Sustainability

The sustainable long-term capacity of each technique is given in Table 6-6. Long-term capacity is the sustainable capacity for a reservoir. The values show that HSRS is capable to retain highest long-term reservoir capacity with 64 mill. m^3 which is the current capacity of the reservoir.

Table 6-6: Long-term reservoir capacity values for different techniques

Long-term reservoir capacity for Flushing	42.80	mill. m^3
Long-term reservoir capacity for HSRS	64.00	mill. m^3
Long-term reservoir capacity for Dredging	37.66	mill. m^3
Long-term reservoir capacity for Trucking	33.94	mill. m^3

6.3 SENSITIVITY ANALYSIS

Changes in input parameters in RESCON model may alter not only the ranking of desirable sediment management strategies, but could also affect the amount of sediment removed with each strategy and magnitude of variables. Sensitivity analysis is performed on key parameters of the Kulekhani reservoir to investigate these effects and test the model for consistency with economic sensitivity. In this study, the most viable and efficient option of the sediment management is explored for various scenarios of flushing discharges, duration of flushing and representative bottom width for the reservoir. Further sensitivity analysis is carried out for various unit values of reservoir yield, mean annual sediment inflow mass and reservoir (dam) operation and maintenance coefficient.

The RESCON model is run for 10-20 m^3/s of flushing discharges with 2-10 days of duration of flushing with the flushing width of 30 m, 40 m and 50 m. Table 6-7 and Figure 6-2 show the calculated SBR and LTCR for different discharges and durations.

Table 6-7: SBR and LTCR for different discharges, bottom flushing width and durations

For flushing discharge, Qf = 10 m^3/s

Flushing time, Tf (Days)	W_b = 30 m		W_b = 40 m		W_b = 50 m	
	SBR	LTCR	SBR	LTCR	SBR	LTCR
2	0.335	0.394	0.335	0.358	0.335	0.328
4	0.671	0.394	0.671	0.358	0.671	0.328
6	1.006	0.394	1.006	0.358	1.006	0.328
8	1.341	0.394	1.341	0.358	1.341	0.328
10	1.676	0.394	1.676	0.358	1.676	0.328

For flushing discharge, Qf = 15 m^3/s

Flushing time, Tf (Days)	W_b = 30 m		W_b = 40 m		W_b = 50 m	
	SBR	LTCR	SBR	LTCR	SBR	LTCR
2	0.568	0.452	0.568	0.411	0.568	0.377
4	1.136	0.452	1.136	0.411	1.136	0.377
6	1.704	0.452	1.704	0.411	1.704	0.377
8	2.272	0.452	2.272	0.411	2.272	0.377
10	2.840	0.452	2.840	0.411	2.840	0.377

For flushing discharge, Qf = 20 m^3/s

Flushing time, Tf (Days)	W_b = 30 m		W_b = 40 m		W_b = 50 m	
	SBR	LTCR	SBR	LTCR	SBR	LTCR
2	0.826	0.502	0.826	0.456	0.826	0.418
4	1.651	0.502	1.651	0.456	1.651	0.418
6	2.477	0.502	2.477	0.456	2.477	0.418
8	3.302	0.502	3.302	0.456	3.302	0.418
10	4.128	0.502	4.128	0.456	4.128	0.418

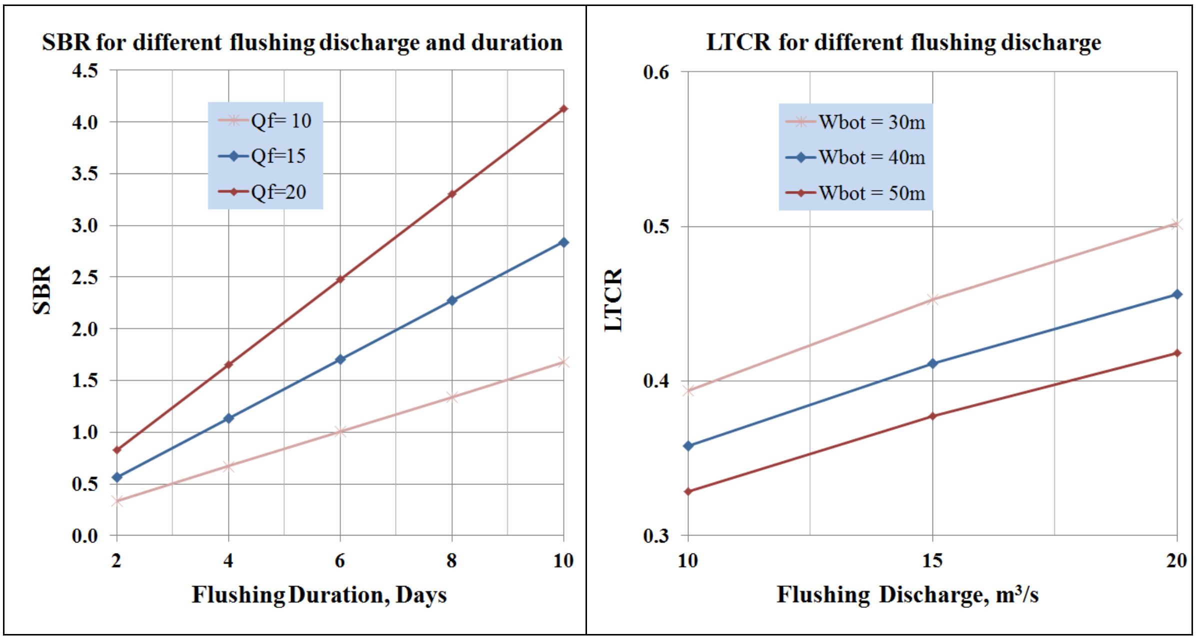


Figure 6-2: Calculated Sediment Balance Ratio (SBR) and Long-Term Capacity Ratio (LTCR) for different flushing discharges and duration

Table 6-7 and Figure 6-2 show that the calculated SBR increases with the increment of the flushing discharge (Q_f) and flushing duration (T_f) but it does not change with increment of bottom flushing width (W_b) for the same flushing discharge and duration. Similarly, calculated LTCR increases with increment of the flushing discharge but decreases with increment of the bottom flushing width and it does not change with increment of the flushing duration for the same bottom width. Table 6-7 shows that maximum LTCR is 0.50 for bottom width 30 m with 4 days flushing time and 1.65 SBR. The RESCON results with these parameters are presented from Table 6-2 to Table 6-6. The results of RESCON analysis show that changes in economic results and amount of sediment removed with changes in above-mentioned parameters are negligible. Therefore, the flushing discharge, flushing duration and bottom widths are assumed 20 m^3/s, 4 days and 30 m, respectively for further sensitivity analysis.

6.3.1 Unit Value of the Reservoir Yield

One of the inputs necessary for the RESCON analysis is the value of the water that is stored in the reservoir. While this parameter has great implications for optimal management of the reservoir, it is usually unavailable to the decision maker (Palmieri *et al.*, 2003).

As stated earlier, the value of the energy produced by the Kulekhani power plants is very high in the energy system of Nepal. Therefore, the sensitivity analysis is performed with the unit value of the reservoir yield with 0.25, 0.375 and 0.50 USD/m^3 assuming the energy price 10, 15 and 20 NRs/kWh, respectively. The results of the sensitivity analysis for the unit value of the reservoir yield with the mean annual sediment inflow mass of 1.0 mill. tons per year and Operation and Maintenance Coefficient of 0.025 are presented in Table 6-8.

Table 6-8: Sensitivity to value of unit reservoir yield

Economical results, aggregate net present value in mill. USD				
Possible strategies	**Technique**	**Value of unit reservoir yield, USD/m³**		
		0.250	**0.375**	**0.500**
Do nothing	Not applicable	245.3	380.1	514.9
Nonsustainable (decommissioning) with partial removal	HSRS	Partial removal with HSRS is technically infeasible, see total removal with HSRS		
Nonsustainable (Run-of-River) with no removal	Not applicable	245.3	380.1	514.9
Nonsustainable (Run-of-River) with partial removal	HSRS	Partial removal with HSRS is technically infeasible, see total removal with HSRS		
Sustainable	Flushing	244.7	379.4	514.0
Sustainable	HSRS	249.8	388.2	526.7
Sustainable	Dredging	244.7	379.5	514.3
Sustainable	Trucking	243.3	377.6	512.2
Remaining sustainable reservoir volume, in mill. m³				
LTCR for Flushing		42.80		
LTCR for HSRS		64.00		
LTCR for Dredging		37.66	36.91	36.92
LTCR for Trucking		33.94	35.43	36.17

Table 6-8 shows that the value of unit reservoir yield is an important parameter in the calculated aggregated benefits. As the unit value of reservoir yield (P1) increases, the total net benefit also increases. When P1 is doubled from 0.25 (USD/m^3) to 0.5 (USD/m^3), Net Present Value (NPV) for all strategies increases by nearly 270 mill. USD. However, the increase in NPV for sustainable solutions by HSRS is somewhat higher than other solutions. The long-term capacity ratio (LTCR) increases 6.5 percent for trucking. It follows that the higher water prices generates incentives to keep more storage capacity. In the case of flushing and HSRS, changes in P1 do not affect LTCR because it is determined by engineering features rather than economic optimization.

6.3.2 MEAN ANNUAL SEDIMENT INFLOW

Estimation of sediment inflow and deposition in a reservoir is not easy, as there are large variations in sediment production rates from place to place and from time to time. The Kulekhani reservoir sedimentation studies show that the annual sediment deposition in the reservoir varied from 0.023 mill. m^3 in 2001 to more than 10 mill. m^3 in 1994 (see Chapter 4). Therefore, to check the sensitivity of sediment inflow mass in different strategies of the sediment management, the sensitivity analysis is performed for the mean annual sediment inflow mass (M_{in}) at 0.5, 1.0 and 1.5 mill. metric ton/year. The results of the sensitivity analysis are summarized in Table 6-9.

Table 6-9: Sensitivity to mean annual sediment inflow mass

Economical results, aggregate net present value in mill. USD				
Possible strategies	**Technique**	**Mean annual sediment inflow mass, mill. metric ton**		
		0.50	**1.00**	**1.50**
Do nothing	Not applicable	249.7	245.3	239.0
Nonsustainable (Decommissioning) with partial removal	HSRS	Partial removal with HSRS is technically infeasible, see total removal with HSRS		246.8
Nonsustainable (Run-of-River) with No Removal	Not applicable	249.7	245.3	239.0
Nonsustainable (Run-of-River) with partial removal	HSRS	Partial removal with HSRS is technically infeasible, see total removal with HSRS		246.8
Sustainable	Flushing	249.6	244.7	239.4
Sustainable	HSRS	251.4	249.8	Total removal with HSRS is technically infeasible, See partial removal with HSRS
Sustainable	Dredging	249.7	244.8	238.7
Sustainable	Trucking	249.7	243.3	227.5
Remaining sustainable reservoir volume, in mill. m^3				
LTCR for Flushing		42.80		
LTCR for HSRS		64.00		Not applicable
LTCR for Dredging		33.94	37.66	40.64
LTCR for Trucking		33.20	33.94	35.06

The results of the sensitivity analysis show that the total net benefit (NPV) decreases with the increment of the mean annual sediment inflow mass (M_{in}). The NPV is increased 9.8% for trucking and 1.8% for HSRS when M_{in} decreased from 1.5 to 0.5 mill. ton/year. However, the NPV for sustainable solutions by HSRS is somewhat higher than other solutions for M_{in} of 0.5 and 1.0 mill. metric ton/year. Total removal with HSRS is technically infeasible for M_{in} of 1.5 mill. metric ton/year. Total removal with HSRS is not possible because maximum sediment evacuation capacity of HSRS solution (1.07 mill. ton/year) is very low compared to annual sediment deposition (1.5 mill. ton/year). However, the NPV for nonsustainable solutions by HSRS is somewhat higher than other solutions.

6.3.3 RESERVOIR OPERATION AND MAINTENANCE COEFFICIENT

The operations and maintenance coefficient (omc) is defined as the ratio of annual operations and maintenance cost to initial construction cost. Thus, an omc value of 0.0125 means that annual operations and maintenance cost is 1.25 percent of initial construction cost. The results of change in this coefficient from 0.0125 to 0.05 are summarized in Table 6-10.

Table 6-10: Sensitivity to reservoir operation and maintenance coefficient

Economical results, aggregate net present value in mill. USD				
Possible strategies	**Technique**	**Operation and maintenance coefficient**		
		0.0125	**0.025**	**0.050**
Do nothing	Not Applicable	257.5	245.3	220.9
Nonsustainable (Decommissioning) with partial removal	HSRS	Partial removal with HSRS is technically infeasible, see total removal with HSRS		
Nonsustainable (Run-of-River) with no removal	Not Applicable	257.5	245.3	220.9
Nonsustainable (Run-of-River) with partial removal	HSRS	Partial removal with HSRS is technically infeasible, see total removal with HSRS		
Sustainable	Flushing	256.9	244.7	220.3
Sustainable	HSRS	262.0	249.8	225.4
Sustainable	Dredging	257.0	244.8	220.4
Sustainable	Trucking	255.5	243.3	218.9
Sustainability, in mill. m^3				
LTCR for Flushing		42.80		
LTCR for HSRS		64.00		
LTCR for Dredging		37.66		
LTCR for Trucking		33.94		

As the omc decreases, the total net benefit increases. When omc is decreased from 0.5 to 0.0125, Net Present Value (NPV) for all strategies increases by 16.5 percent. However, the NPV for sustainable solutions by HSRS is somewhat higher than other solutions. Since annual operations and maintenance cost is independent of sediment management strategies, the change in omc reduces the NPVs for all strategies by the same amount. Long-term capacity, frequency of sediment removal and the amount of sediment removal are all independent of the change in annual operations and maintenance cost.

6.4 RESULTS AND COMMENTS

The RESCON analysis show that changes in economic parameters do not affect LTCR in the case of flushing and HSRS, because it is determined by engineering features rather than economic optimization.

Based on the RESCON analysis of SBR and LTCR values, the flushing sediment management option seems to be feasible for the Kulekhani reservoir. However, the flushing is ranked as the third sediment management option in economical results of the RESCON model.

All the strategies have yielded sustainable solution except the HSRS for the annual sediment inflow mass (M_{in}) of 1.5 mill. ton/year. The highest net aggregate benefit is achieved by using HSRS technique for all the sensitivity cases, which should be seen from Table 6-4, Table 6-8, Table 6-9 and Table 6-10. The highest long-term capacity is also achieved by the HSRS which is 100% of the present capacity (64 mill. m^3).

The reservoir flushing is favourable for the Himalayas where flows and sediment loads in the rivers are concentrated during the monsoon. Sangroula (2005) reported that about 80% of the annual precipitation in Nepal falls during the monsoon season and rivers carry more than 90% of sediment load during this time of the year. Further, he stated that there are few events, which carry most of the sediment load for the entire monsoon season and such hydraulic regime can be used to flush out deposited sediment. As already reported, there is not any engineering facility at the Kulekhani dam to control water levels and outflows for the sediment flushing. However, a new tunnel can be constructed along the left bank of the Kulekhani dam. The new tunnel can also be used for the flushing pipes of HSRS also. Therefore, the flushing can be combined with the HSRS for sedimentation management of Kulekhani reservoir.

The calculated sediment concentration through hydrosuction pipe of HSRS is only 5,037 ppm which is very low compared to the sediment concentration achieved (more than 100,000 ppm) by Jacobsen (1997, 2006) and Belete (2007) in the field work and laboratory test. Therefore, quantity of sediment removed by HSRS method could be very high compared to the calculated by RESCON method. Hence, the technical calculation of sediment transport and sediment concentration in RESCON method should be reviewed and updated.

Based on the results of the RESCON model, the HSRS is the best sediment management option. The result is based on the use of HSRS for the entire year. However, use of water during the dry season for flushing the sediment from the Kulekhani reservoir is very doubtful as Nepal is facing maximum load shedding during the dry season. The electricity generation from the Kulekhani hydropower project is crucial during the dry season in Nepal where not all other hydropower projects are able to generate with full capacity.

7 FIELD EXPERIMENT OF MODIFIED DOUBLE LAYER SEDIMENT SLUICER (MDLSS)

7.1 INTRODUCTION

Although there are several methods for sediment management in reservoirs, the author mostly focuses on hydrosuction removal of deposited sediment from the reservoir. Some studies and field investigations made by sediment researchers (see Chapter 7.2) in the past showed that hydrosuction is promising for sediment removal.

The Hydrosuction Sediment Removal System (HSRS) uses the hydraulic head available at the dam to provide energy for sediment removal as shown in Figure 7-1. It consists of a pipeline and valves to control the flow. The pipeline entrance is placed upstream at a location where sediment capture or removal is desired in a reservoir or pond. The pipeline extends downstream either over the dam or through low-level outlets to downstream (Figure 7-1).

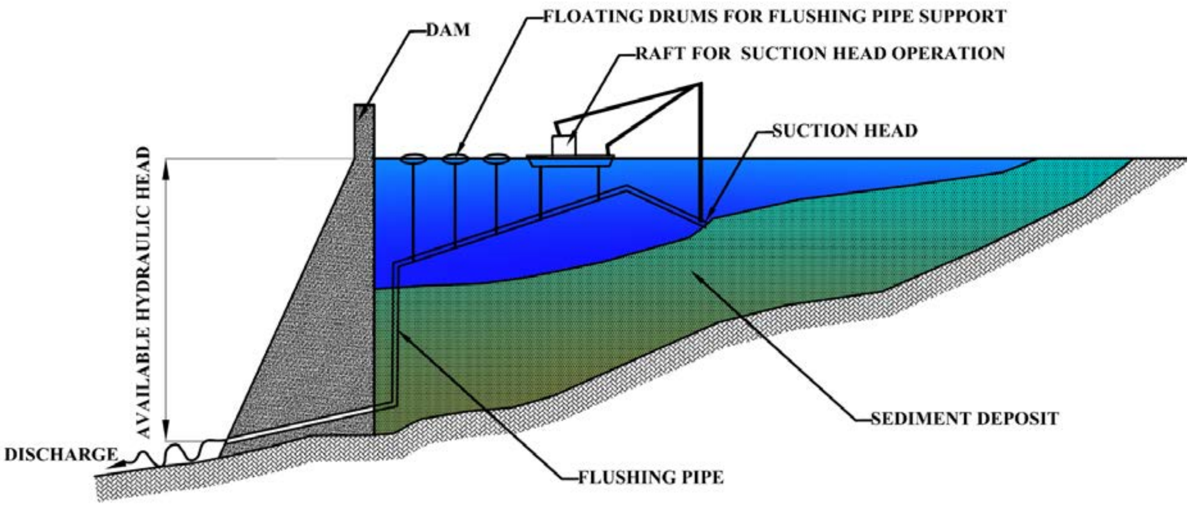


Figure 7-1: Hydrosuction Sediment Removal System (HSRS)

A double layer suction head of HSRS has been modified (see Chapter 7.4) to minimize problems related to clogging of sediment flushing pipe and water jetting arrangement incorporated to break up consolidated sediment deposit.

7.2 PREVIOUS STUDIES

Hotchkiss and Huang (1995) reported several cases of application of HSRS. HSRS dredging was first performed in Djidiouia reservoir in Algeria from 1892 to 1894 (Hotchkiss and Huang, 1995). According to them "hydroaspirator" method for removing sediment had been conceived in France about 100 years earlier but had not been used extensively. The People's Republic of China to the date has got the most experience with hydrosuction dredging. The Chinese have used either the siphon or the bottom withdrawal modes in 10 reservoirs, beginning in 1975 (Hotchkiss and Huang, 1995). They state that Shahriar Eftekharzadeh was responsible for beginning the investigation of hydrosuction sediment removal in the United States. Hotchkiss and Huang (1995) developed the “Dustpan” inlet for HSRS for the field test carried out at Lake Atkinson, on the Elkhorn river in Nebraska during the summer of 1993. Jacobsen (1997) developed the Slotted Pipe Sediment Sluicer (SPSS) and the Saxophone Sediment Sluicer (SSS) in 1993 during his PhD study. Jacobson (2006) has installed hydrosuction removal system in peaking pond of Malana hydro station (86 MW run-of-river plant) in the Kullu District of Himachal Pradesh India, in May 2005 for the removal of sediment. The Inverted Cone Sediment Sluicer (ICSS) type or the funnel shape suction head was developed in India and has been successfully used for sediment removal at Salal hydro project in India (Belete, 2007). The Double Layer Sediment Sluicer (DLSS) was developed by Kiflom Belete during his PhD study.

7.3 THE DOUBLE LAYER SEDIMENT SLUICER (DLSS)

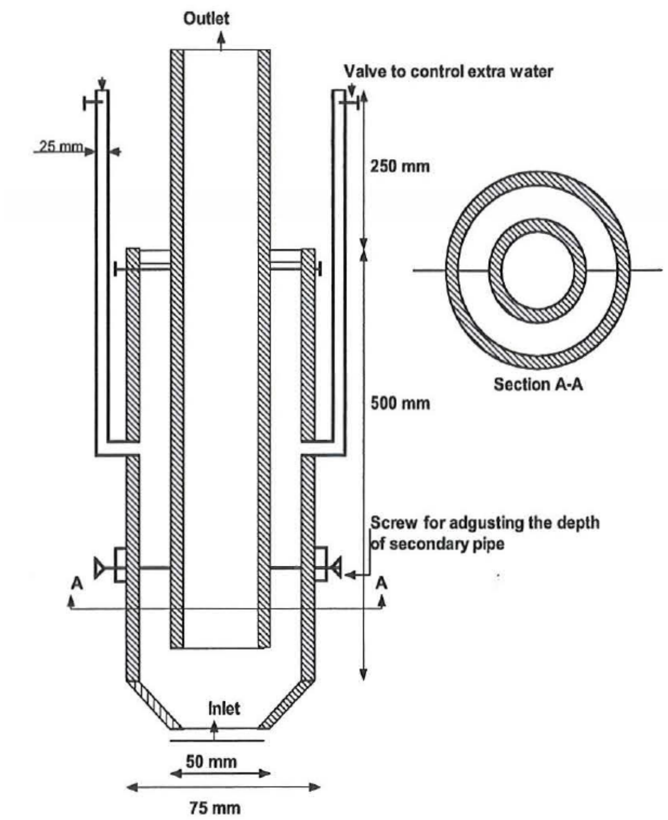
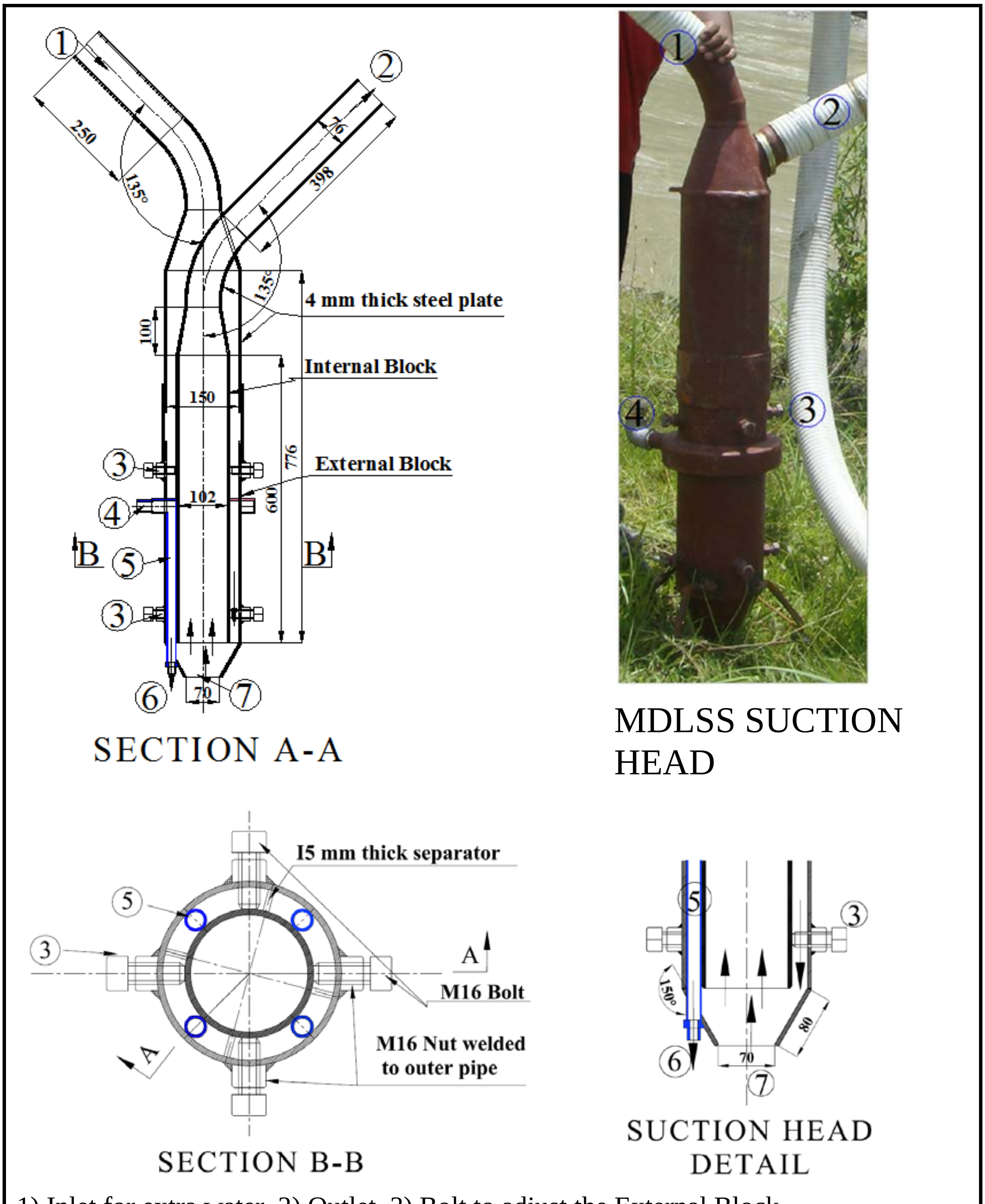


Figure 7-2: Double Layer Sediment Sluicer (Belete, 2007)

The Double Layer Sediment Sluicer (DLSS) was developed by Kiflom Belete during his PhD study. He reported that most of the water and all the sediment are drawn through the bottom mouth of the suction head during normal operation. Extra water was drawn into the main line through the two vertical opening. The suction head used extra water for self-clearing in case there was excess suction of sediment or the mouth was stacked inside the sediment. The opening for the extra water was located at the upper end to prevent blocking by sediment. The performance test was conducted using suction heads of similar sediment material and similar water level in the flume. The performance of suction heads based on the ratio of the average volume of sediment removed to average total volume of water used from the test showed that the performance of DLSS was found to be higher compared to the SSS and ICSS (Belete, 2007). Further, he concluded, “the DLSS suction head was found more promising to be used in the prototype reservoirs in the study area where the deposited sediment is dominated by more cohesive and consolidated materials”.

7.4 MODIFIED DOUBLE LAYER SEDIMENT SLUICER (MDLSS)

The Double Layer Sediment Sluicer (DLSS) is slightly modified in this study to minimize problems related to clogging of the outlet pipe and to incorporate water jetting arrangement to break up consolidated sediment deposit. The drawing and picture of the MDLSS is shown in Figure 7-3.



1) Inlet for extra water, 2) Outlet, 3) Bolt to adjust the External Block
4) 1” Nipple to connect the pressure pipe, 5) Pressure pipe for water jetting
6) Nozzle for water jetting, 7) Inlet of the Suction Head

Figure 7-3: Suction head of MDLSS

The Inlet (1) supplies extra water into the main suction head through the space between the external pipe and the main suction pipe. Water from the reservoir is used for self-clearing in case there is excess suction of sediment or sudden blockage of the inlet. The amount of extra water can be controlled with the valve located at the inlet pipe and/or adjusting the external block and main suction pipe with bolts (3) to control the gap between the outer and inner pipes. Four nozzles (6) are mounted at the inlet of the suction head for water jets to break up consolidated sediment (if required). The diameter of the water jet nozzles are 5 mm and connected with one inch pipe (5). A high pressure pump is used to supply clean water with high pressure which will create high velocity in the nozzles.

7.5 FIELD EXPERIMENT

Field test with the Modified Double Layer Sediment Sluicer (MDLSS) were performed at the settling basin of Sunkoshi small hydropower plant and the peaking pond of Sunkoshi hydropower plant from 19 to 24 July 2009.

7.5.1 SETTLING BASIN OF SUNKOSHI SMALL HYDROPOWER PLANT

Sanima Hydropower (P) Ltd. (SHPL) is the owner of the Sunkoshi small hydropower plant which is located in the Sunkoshi Khola about 88 km north-east of Kathamndu in Sindhupalchok district of central Nepal. The project is RoR type, which diverts a design flow of 2.7 m^3/s through a 2.6 km long, 1.3 and 1.2 meter diameters Glass-fibre Reinforced Plastic (GRP) pipe to the powerhouse. The water diverted from the river first reaches to the settling basin through an approach canal and it enters the headrace system through a forebay. Utilizing a rated head of 117.5 m, the project generates 2.5 MW of power.

The settling basin has two chambers each 4.5 m width and 45 m long settling zone. The maximum flow velocity in the settling zones is 0.2 m/s.

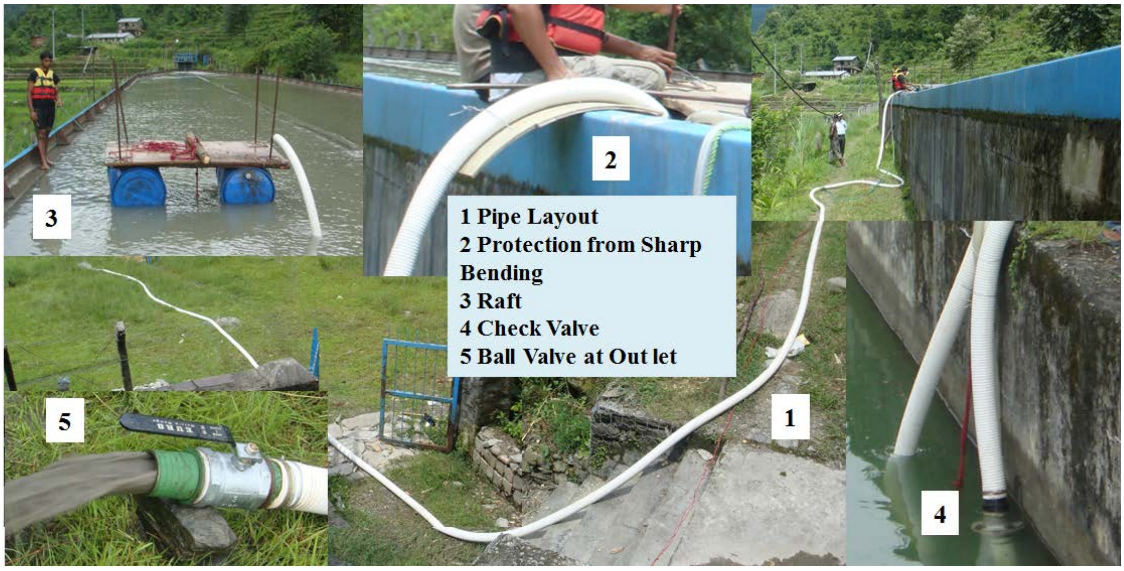


Figure 7-4: Experimental setup in settling basin of Sunkoshi small hydropower plant

Experimental Setup

Experimental setup for field experiments consist of suction head or intake, pipes, valves, pump and a raft. The Modified Double Layer Sediment Sluicer (MDLSS) was used as the intake for the system. A 40 m long, three inch diameter flexible corrugated High Density Polyethylene (HDP) pipe was used for sediment transport and to feed extra water for self-clearing in case there is excess suction of sediment or the mouth is stacked inside the sediment deposit. The sediment transport pipe was connected at the outlet of the suction head and equipped with a check valve (to stop water flow towards intake during priming) near the connection and a ball valve at the outlet of the pipe. A 2 kW centrifugal pump was used for priming and water jetting. A locally produced raft from drums was used for moving the suction head.

The experimental setup in settling basin of Sunkoshi small hydropower plant is shown in Figure 7-4. The flushing pipe was laid over the side wall (0.4 m above the water level) of the settling basin. The outlet of the pipe was kept near the sediment flushing canal.

Sediment Removal Operation

As the pipe passed over the side wall and thus above the water level, priming of the pipe was done by pumping water from the settling basin by the pump. During priming, the ball valve at the outlet was closed, air was released from the system through an air release valve located at the highest point of the pipe and a check valve stopped the water flow towards the intake during priming. After the pipe was successfully filled with water, the air release valve was closed, the downstream valve was opened, suction developed and initiated the transportation of the sediment water mixture. An area with sediment deposits more than one metre in thickness was chosen for the experiment. Sediment deposit level was measured manually with one inch square steel pipe with a 15 cm x 15 cm flat plate at the end before and after sluicing of the sediment deposit, as shown in Figure 7-5. The suction head (inlet) was slowly lowered to the bed and moved to different locations when the concentration of the sediment mixture in the outflow pipe was reduced. All inlet movements were done manually as shown in Figure 7-5. Sediment water mixture was collected in a 200 liters drum to measures the discharge as shown in Figure 7-5.

Figure 7-5: Measurement of sediment deposition level, discharge and relocation of suction head

Figure 7-6: Flow at the outlet after priming, normal and clogged flow

The sluicing was performed with supplying and without supplying the extra water at different opening width of the inlet (gap between the internal and external pipe at suction head inlet). The available head for sediment sluicing was 3.8 m and 3.0 m (deducting the height of drum) at the inlet and the outlet area of the settling basin, respectively.

When the suction head and/or the transportation pipe were blocked, they were cleared by lifting the suction head up from the sediment deposit to draw extra water and injecting extra water into the pipe with the pump. If it did not work, the suction head was brought to the water surface level to check if it was blocked by debris and after removal of the blocking material, the work was resumed immediately.

Operational Difficulties

The following difficulties were faced during the field experiment:

- Pipeline clogging occurred only when extra water was not supplied to the suction head. Once the transportation pipe was clogged, clearing the sediment from the pipe was difficult.
- It was quite difficult to know if the suction head was positioned vertically or not. At the beginning, vertical orientation of the suction head was checked with the water depth and length of the rope dropped. However, after some relocation of the suction head, it was quite difficult to know its location. Location of the suction head was detected with water jetting.

7.5.2 PEAKING POND OF THE SUNKOSHI HYDROPOWER PLANT

Sunkoshi hydropower plant is located about 7 kilometers downstream of Sunkoshi small hydropower plant. Nepal Electricity Authority (NEA) owns this power plant. It is a peaking run of the river type with an installed capacity of 10.05 MW and consists of the headworks, canal, peaking pond and the powerhouse. The powerhouse is in operation from January 1972. The peaking pond and powerhouse area of Sunkoshi power plant is shown in Figure 7-7.

Figure 7-7: Peaking pond and powerhouse area of Sunkoshi power plant (Picture from Google Earth, May 2012)

Experimental Setup

The experimental setup in the peaking pond of Sunkoshi power plant is shown in Figure 7-8. The pipe was laid over the flushing gate (1.2 m above the water level). The outlet of the pipe was kept at the stilling basin of the spillway of the forebay.

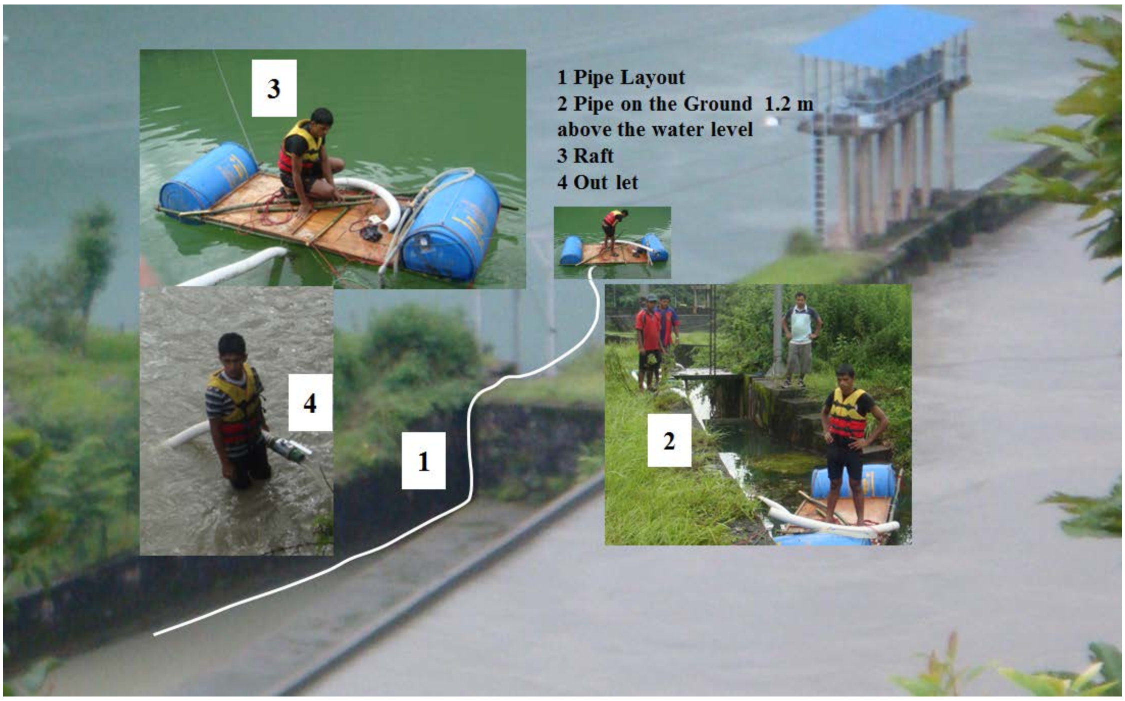


Figure 7-8: Experimental setup in the peaking pond of Sunkoshi hydropower plant

Sediment Removal Operation

As pipe layout was above the water level, priming of the pipe system was done by pumping water from peaking pond, similar to the priming in settling basin of the Sunkoshi small hydropower plant. Available head for sediment sluicing was 3.0 m.

Figure 7-9: Dense weed growth in the peaking pond and debris catching on the check valve plate.

The peaking pond of the Sunkoshi hydropower plant is utilized during the dry (winter) season when there is relatively very low sediment concentration in the river. During the wet (monsoon) season the inlet gate of the peaking ponds are closed to stop the access of sediment-loaded water. As the peaking pond is located just upstream of the forebay and used only during the dry season, the annual sediment deposition rate in the peaking pond is low. Sediment was not flushed for a long time, therefore, sediment accumulated year by year; about 20% volume of the peaking pond was already filled with sediment. The sediment deposit was already highly consolidated with a relatively dense weed growth (Figure 7-9). Therefore, to evaluate and improve the performance of the hydrosuction sediment removal, the test was carried out with and without water jetting system. The water jetting was used to disintegrate the consolidated sediment deposit. The jets through the nozzles directed into the sediment deposited to break the cohesive materials and easily transported through the pipeline system.

7.5.3 EXPERIMENTAL RESULTS

There was no exact measurement of removed sediment, as it was difficult to take measurements. However, the performance of the sluicing system was assessed based on sample analysis in the laboratory. During the field experiment, samples were collected in bottles for sediment concentration calculation. The sediment concentration removed from settling basin and peaking pond is tabulated in Table 7-1 and Table 7-2, respectively.

Table 7-1: Recorded sediment concentration from settling basin of Sunkoshi small hydropower plant

S.No.	Sampling date	Opening for extra water	Total concentration (PPM)	Head, (m)	Discharge, (l/s)	Velocity in pipe, (m/s)
1	20-Jul-09	Closed	102,600	3.8	6.06	1.37
2	20-Jul-09	Closed	98,100		6.25	1.42
3	21-Jul-09	Closed	105,000		6.06	1.37
4	21-Jul-09	2 cm	92,000		6.45	1.46
5	21-Jul-09	2 cm	83,100		6.67	1.51
6	22-Jul-09	2 cm	61,000	3.0	5.56	1.26
7	22-Jul-09	4 cm	55,800		5.71	1.29
8	23-Jul-09	4 cm	38,500		6.06	1.37
9	23-Jul-09	4 cm	44,700		5.88	1.33

Table 7-2: Recorded sediment concentration from the peaking pond of Sunkoshi hydropower plant

S.No.	Sampling date	Water jetting	Total concentration (PPM)	Head, (m)	Discharge, (l/s)	Velocity, in pipe (m/s)
1	24-Jul-09	No	3,900	3.0	5.41	1.22
2	24-Jul-09	No	4,700		5.41	1.22
3	24-Jul-09	No	12,300		5.56	1.26
4	24-Jul-09	Yes	23,600		5.71	1.29
5	24-Jul-09	Yes	24,700		5.71	1.29
6	24-Jul-09	Yes	28,400		5.88	1.33

The sediment concentration in the river was high during the field experiment. The frequency of sediment filling in the settling basin of Sunkoshi small hydropower plant was high and sediment was flushed every second day. Therefore, the sediment in the settling basin was not consolidated and water jetting was not needed. The highest observed sediment concentration was 105,000 ppm when performed closing the gap between the inner and outer pipe but the pipeline clogging occurred two times. There was not any clogging after feeding extra water, about 40 percent of the total flushing discharge, with 2 cm of opening the gap between the inner and outer pipe. The highest observed sediment concentration was 92,000 ppm with this opening. The sediment concentration was less up to 38,500 ppm only when the extra feeding was increased to 60 percent.

Sediment deposit in the peaking pond of the Sunkoshi power plant was highly consolidated; therefore, the test was carried out with and without water jetting system and without feeding of extra water. The highest observed sediment concentrations with water jetting was 28,400 ppm. The lowest observed sediment concentrations without water jetting was 3,900 ppm.

7.6 DISCUSSION

Pipeline clogging occurred two times within 10 hours of operation when the gap between the inner and outer pipe was closed, rather the sediment concentration was higher in the flow. It shows that some extra water should be supplied to balance the suction capacity of the suction head and transport capacity of the pipe to avoid clogging of the pipe. Clogging was not occurred after feeding the extra water but the sediment concentration was less. When the opening at intake between the outer and inner block was 2 cm, the concentration varied from 61,000 to 92,000 ppm. Similarly, when the opening was 4 cm, the concentration lowered to 44,700 and 55,800 ppm. It shows that clogging could be avoided by feeding extra water into the pipeline through the proposed suction head.

Sediment deposit in the peaking pond was highly consolidated. The hydrosuction system could sluice only 3,900 to 12,300 ppm, but with water jetting it was increased up to 28,400 ppm. The water jetting was performed only with 2 KW centrifugal pump. Higher sediment concentration could probably be achieved with higher capacity of the water-jetting pump.

7.7 CONCLUSION AND RECOMMENDATION

The field test of HSRS with the Modified Double Layer Sediment Sluicer (MDLSS) indicates that it is a promising technology for sediment removal from peaking ponds and reservoirs.

The field test study shows that the efficiency of the hydrosuction sediment removal system can be increased when hydrosuction is combined with water jetting for consolidated sediment deposits.

If the suction head was not moved inside the settling basin for a while, the sediment concentration decreased with time and eventually approached zero. This is because the effective entrainment suction velocity is limited to a small area around the inlet.

The suction head must be placed where the deposits are and it is also important to maintain the vertical orientation of the suction head. The performance of the system can be improved with the adjustment of the opening between the inner and the outer pipe according to consolidation level of sediment deposit.

8 SUSTAINABILITY OF THE KULEKHANI RESERVOIR

8.1 INTRODUCTION

It is very important to define suitable sediment control techniques to minimise the impact of reservoir sedimentation and thus ensuring long-term sustainable use of the reservoir. The main criteria for selection of the control measures are technical, economic and environmental feasibility, which depends on a number of site specification factors such as:

- Availability of suitable bottom outlet facilities
- Water available for flushing
- Characteristics of the sediment load and the reservoir
- Consequences of flushing/dredging with respect to sediment disposal
- Consequences of control measures interfering with the reservoir operation
- Environmental impacts
- Institutional and political limitations

Several studies carried out by Nippon Koei (1994), Shrestha (2001) and Sangroula (2005) recommended different sediment control measures for the sustainability of the Kulekhani reservoir. One of them is the HSRS method. Based on the results of the RESCON model, HSRS is the best sediment management option (see Chapter 6). However, the author recommends to study the feasibility of reservoir flushing and HSRS separately.

The following section will focus on reservoir flushing and HSRS as sediment management options in Kulekhani reservoir. The technical feasibility of these sediment management options for the Kulekhani reservoir is evaluated separately.

8.2 RESERVOIR FLUSHING

One option for sediment control of the reservoir sedimentation is to flush sediment through outlet works. This technique is effective under certain favorable conditions and is not universally applicable (Atkinson, 1996). Study carried out by HR Wallingford (Atkinson, 1996) evaluates the feasibility of flushing sediment from reservoirs using a simple feasibility criterion. Atkinson (1996) developed the methodology for estimating efficiency of flushing at a given reservoir based on reservoir geometry, sediment characteristics, discharge regimes, flushing capacity etc. He further reported several reservoirs that had been successfully flushed.

White (2001) looked at fifty reservoirs which were being or had been flushed. In some cases, the flushing was successful, in others there was little or no success. Based on his studies White (2001) suggests several requirements for successful flushing. Some of them are:

- The hydrology and sedimentology of the catchment need to be fully understood in the planning of flushing facilities
- Flushing is more practicable in hydrologically small reservoirs with a storage capacity less than 30% of the mean annual inflow.
- Flushing is vital in reservoirs where the annual sediment deposition potential is greater than 1% to 2% of its volume (Palmieri *et al.*, 2003).

- Narrow steep-sided reservoirs in valleys with a steep longitudinal slope are the easiest to flush.
- Full drawdown and flushing have been found to be much more effective than partial drawdown and flushing.

8.2.1 FLUSHING SYSTEM ARRANGEMENT

Sangroula (2005) has proposed a sediment flushing system arrangement for Kulekhani reservoir. He has addressed the possibility of using diversion tunnel No. 1, which was designed as a river bypass facility and drawdown of the reservoir if necessary (Nippon Koei, 1983). Sangroula (2005) further has reported that the diversion system (outlet facility) was operated to drawdown the water in the reservoir for maintaining purpose in 1984 but during the construction of the sloping intake in 1997, the operating mechanism was not used.

Nippon Koei (1983) reported that the outlet facility provided in the diversion tunnel No. 1 consists of a 26.2 m high intake tower, a steel conduit of 1.0 m in diameter and guard and hollow jet valves with a valve chamber. The inlet was set at 1,461 masl. The tunnel was plugged with a 5 m long concrete plug in front of the intake tower.

Bathymetric survey carried out in 2010 December shows that the reservoir bed level around the inlet portion of the diversion tunnel is about 1,460 masl (Figure 8-2). Reopening of the intake to the outlet facility is a great technical challenge. The main challenge is to draw down the water level below the intake level of the sloping intake, which is at 1,480 masl. Therefore, the inverted level of the flushing should be kept higher than 1,480 masl. Appropriate intake and the structures (gates) should be built to control water levels and outflows. The invert level of the intake should be kept around 1,485 masl so that the Kulekhani power plants can run during the construction time. A new vertical shaft (tunnel) should be constructed to connect the intake area with the existing diversion tunnel (Figure 8-1). The diversion tunnels were not designed for sediment flushing purposes and the abrasive resistivity of the surface of the tunnel has to be investigated if it is considered for flushing. The layout and longitudinal section of the sediment flushing system using existing tunnel No. 1 are shown in Figure 8-1 and Figure 8-2, respectively.

Another option is to construct a new tunnel through the left bank abutment from the plunge pool and upstream into the reservoir. Figure 8-3 shows the longitudinal section of a sediment flushing system with a new tunnel. The invert level of the intake is kept at 1,485 masl so that the Kulekhani power plants could run during the construction time. The intake and the engineering facilities (gates) are similar to the structures, proposed as in the existing tunnel option. A new tunnel with minimum practicable size (2.2 m wide and 2.5 m high) to construction should be constructed to flush the sediment.

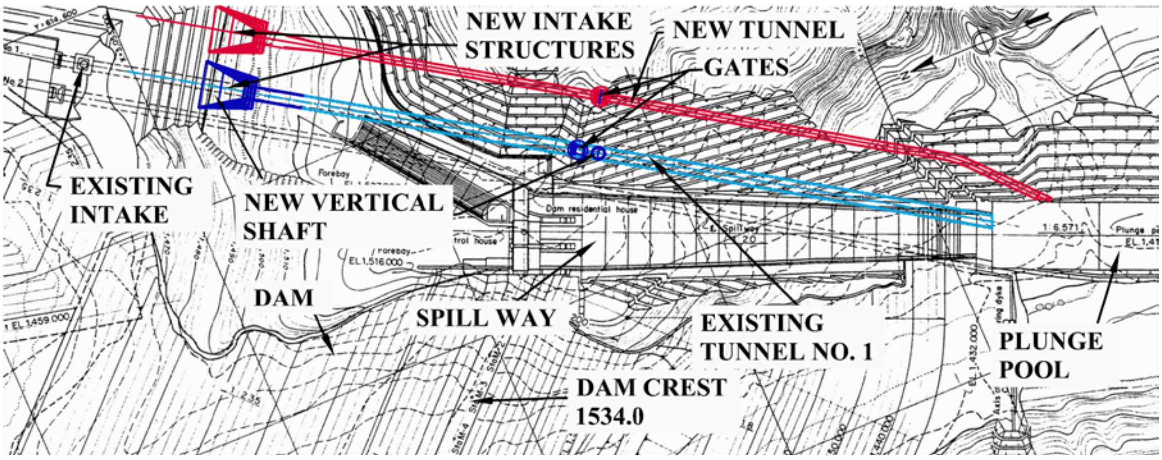


Figure 8-1: Flushing tunnel layout plan with alternatives

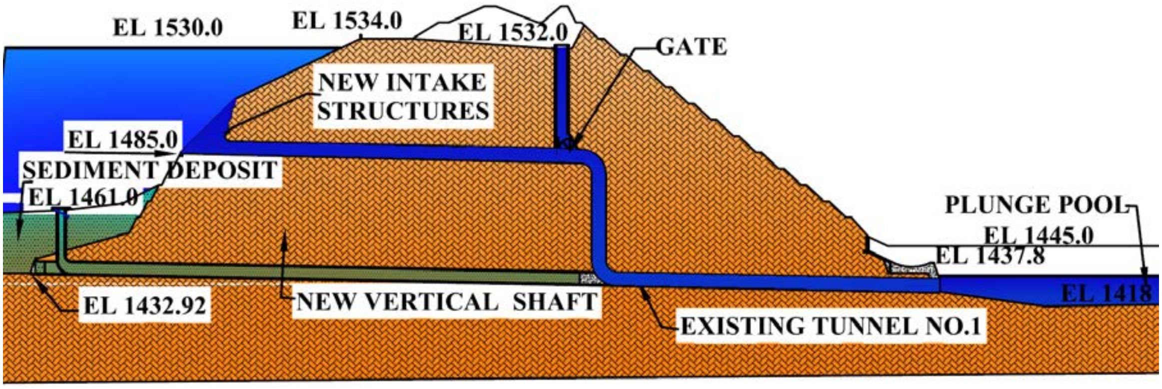


Figure 8-2: Longitudinal section of flushing tunnel using the existing tunnel No. 1

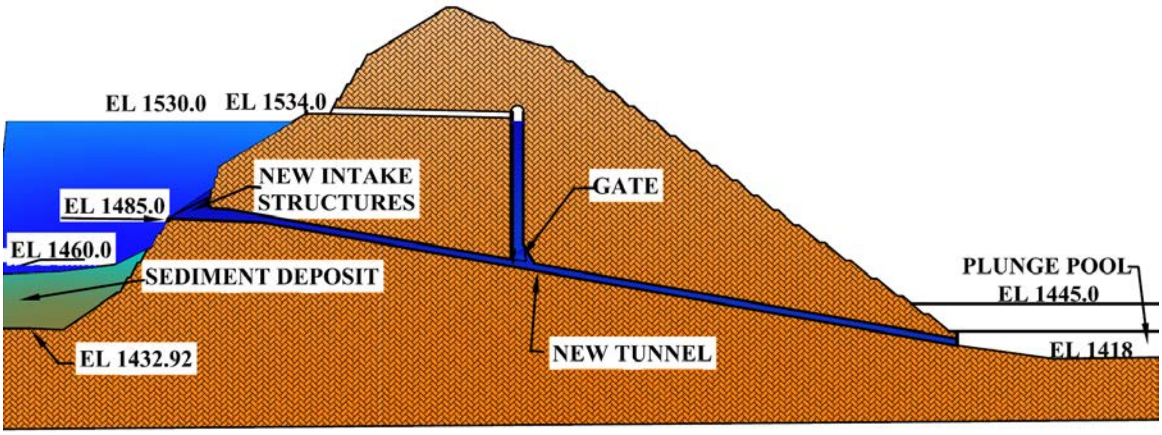


Figure 8-3: Longitudinal section of flushing tunnel constructing new tunnel

8.2.2 Feasibility Criteria for Flushing

The feasibility of flushing of the Kulekhani reservoir is evaluated based on the two major criteria developed by Atkinson (1996). They are sediment balance ratio (SBR) and long-term capacity ratio (LTCR). The calculated SBR and LTCR are compared to the results of the RESCON model. This model does also calculate the SBR and LTCR according to the criteria developed by Atkinson (1996). The sensitivity of the adopted criteria will be evaluated for different input parameters using the RESCON model.

The parameters used to estimate the flushing feasibility of the Kulekhani reservoir are presented in Table 8-1.

Table 8-1: Input parameters to estimate the flushing feasibility criteria

Reservoir geometry			
Symbol	**Units**	**Description**	**Value**
S_o	(m^3)	Original capacity of the reservoir	85,290,000
S_e	(m^3)	Existing storage capacity of the reservoir	55,190,000
W_{bot}	(m)	Representative bottom width for the reservoir	60.0
SS_{res}		Representative side slope for the reservoir.	1.0
EL_{max}	(m)	Elevation of top water level in the reservoir	1,530.0
EL_{min}	(m)	Minimum bed elevation	1,485.0
EL_f	(m)	Water elevation at dam during flushing	1,490
L	(m)	Reservoir length at the normal pool elevation.	7,000
Water characteristics			
V_{in}	(m^3)	Mean annual reservoir inflow (mean annual runoff)	137,040,000
Sediment characteristics			
ρ_d	(ton/m^3)	Density of in-situ reservoir sediment.	1.30
M_{in}	(ton)	Mean annual sediment inflow mass.	1,000,000
Removal parameters			
Q_f	(m^3/s)	Representative flushing discharge.	20
T_f	(days)	Duration of flushing after complete drawdown.	10
Capital investment			
FI	USD	Cost of capital investment required for implementing flushing measures.	3,500,000

SBR Calculation

i. The representative reservoir width "W_{res}" at the flushing water surface elevation is calculated as:

$$W_{res} = W_{bot} + 2 \text{ x } SS_{res} \text{ x } \left(El_f - El_{\min}\right)$$

W_{res} = 60 + 2 x 1 x (1490 – 1485) = 70.0 m

ii. Actual Flushing Width

$$W_f = 12.8 \text{ x } Q_f^{\;0.5}$$

$W_f = 12.8 \times 20^{0.5} = 57.2$ m

iii. Take the minimum of W_{res} and W_f as the representative width

W = 57.2 m

iv. The longitudinal slope during flushing

$$S = \frac{EL_{\max} - EL_f}{L}$$

$$S = \frac{1530 - 1490}{7000} = 0.006\ (1{:}175)$$

v. The value of Ψ is 180.

vi. The sediment load during flushing is:

$$Q_s = \Psi \frac{Q_f^{1.6} \times S^{1.2}}{W^{0.6}} = 180 \frac{20^{1.6} \times 0.0057^{1.2}}{57.24^{0.6}} = 3.9 \text{ ton/s}$$

Then this value is reduced by a factor of three according to Atkinson's criteria, since the reservoir is not similar to Chinese reservoirs.

Q_s =1.30 ton/s

vii. Sediment Mass flushed annually is:

$$M_f = 86{,}400 \times T_f \times Q_s$$

86,400 is the number of seconds in a day

$M_f = 86{,}400 \times 10 \times 1.3 = 1{,}123{,}200$ t

viii. Determine the sediment mass deposited (M_{dep}) in the reservoir

$$M_{dep} = M_{in} \times \left(\frac{TE}{100}\right)$$

where, TE is the trap efficiency.

The Brune curve (Figure 8-4) method of determining trap efficiency is used. TE is determined according to the capacity inflow ratio. The capacity inflow ratio is:

$$CIR = \frac{S_o}{V_{in}} = \frac{85{,}290{,}000}{137{,}040{,}000} = 0.62$$

As CIR = 0.62 then TE= 93.9 %

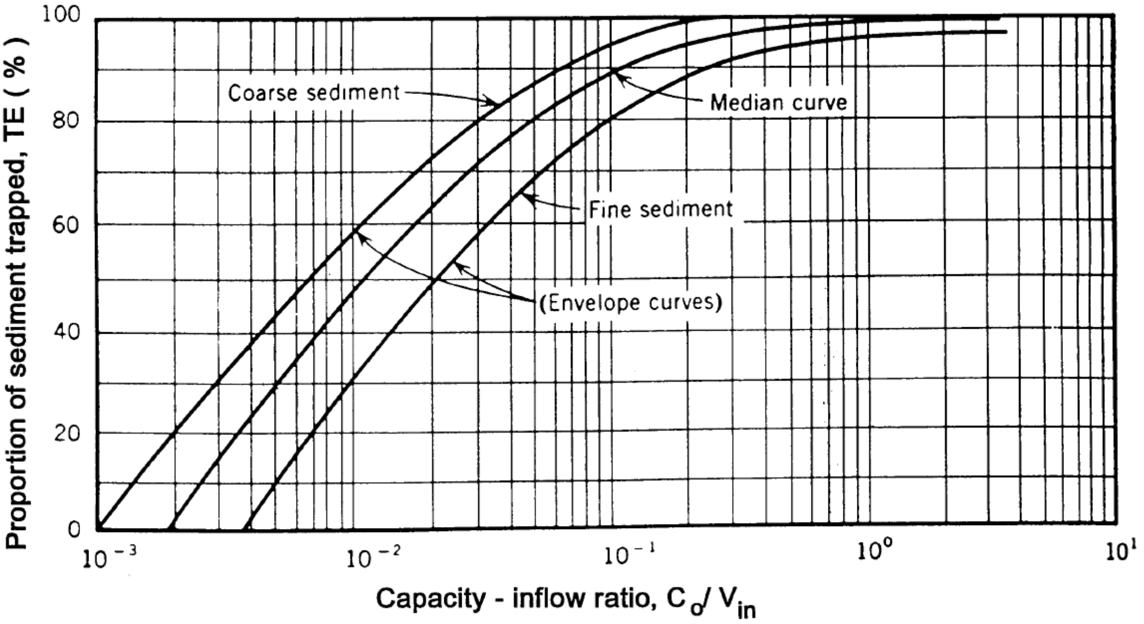


Figure 8-4: Brune's curve for estimating reservoir trapping efficiency (Atkinson, 1996)

ix. Sediment mass deposited annually is:

M_{dep} = 1,000,000 x 0.939 = 939,000 ton

x. Finally the sediment balance ratio is calculated as

$$SBR = \frac{M_f}{M_{dep}}$$

SBR = 1,122,102 / 939,000 = 1.19

The SBR value is bigger than unity. This means that the sediment mass which can be flushed annually is higher than the sediment mass deposited annually. Kulekhani reservoir can therefore, be sustainable with the flushing operation outlined above.

LTCR Calculation:

To calculate LTCR values, Atkinson (1996) proposed a simplified trapezoidal cross section immediately upstream of the dam for simplified reservoir geometry and the scoured channel constricted by reservoir sides as shown in Figure 8-5, which is used to determine the ratio of cross-sectional area of the channel formed by flushing to the original reservoir cross-section area.

i. The Scoured valley width is calculated as:

$$W_{tf} = W + 2 \text{ x } SS_s \text{ x } \left(El_{\max} - El_f\right)$$

Where, SS_s is the representative side slope for the deposits exposed during flushing and calculated from the Migniot's equation:

$$\tan\alpha = \frac{31.5}{5} \text{ x } \rho_d^{4.7}$$

Where, ρ_d is dry density in ton/m^3.

$\tan\alpha = \frac{31.5}{5} \times 1.3^{4.7} = 21.62$ (for correction divide by 10) =2.162

$SS_s = 1/\tan\alpha = 1/2.162 = 0.46$

$W_{tf} = 57.2 + 2 \times 0.46 \times (1530 - 1490) = 94.24$ m

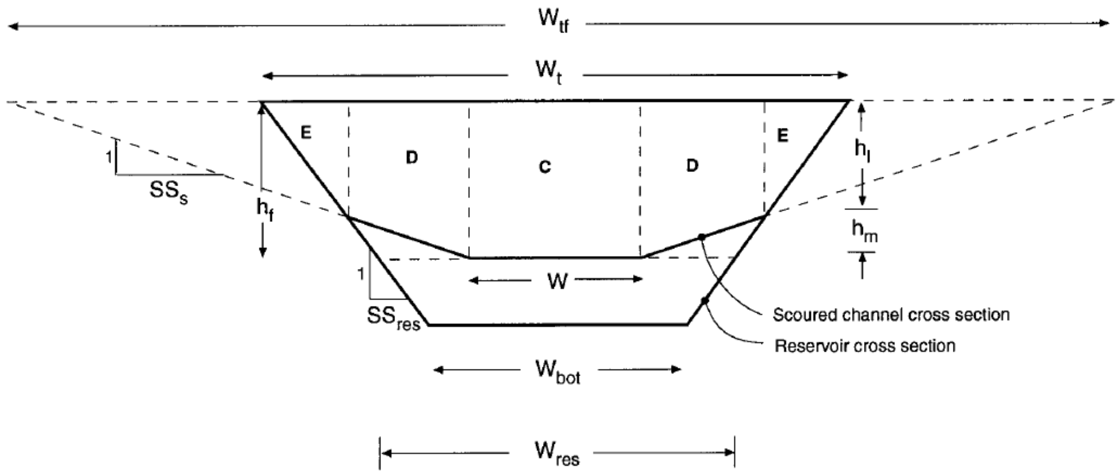


Figure 8-5: Cross section immediately upstream of dam for simplified reservoir geometry and the scoured channel constricted by reservoir sides (Atkinson, 1996)

ii. The reservoir width at this elevation is:

$$W_t = W_{bot} + 2 \times SSres \times \left(El_{\max} - El_{\min}\right)$$

$W_t = 60 + 2 \times 1 \times (1530 - 1485) = 150$ m

iii. Since Wtf ≤ Wt then scoured valley cross sectional area is calculated as:

$$A_f = \frac{W_{tf} + W}{2}\left(El_{\max} - El_f\right)$$

$A_f = ((94.24 + 57.2)/2) \times (1530 - 1490) = 3{,}030\ m^2$

iv. Reservoir cross section area is calculated as:

$$A_r = \frac{W_t + W_{bot}}{2}\left(El_{\max} - El_{\min}\right)$$

$A_r = ((150 + 60)/2) \times (1530 - 1485) = 4{,}725\ m^2$

v. Finally long-term capacity ratio is calculated as:

$$LTCR = \frac{A_f}{A_r}$$

LTCR = 3,030 /4,725 =0.64

The LTCR value 0.64 is bigger than 0.5 thus the flushing system is effective regarding this criterion.

The calculated and the RESCON model results of SBR and LTCR are summarized in Table 8-2. The results show that the calculated and the RESCON model results have given similar results.

Table 8-2: Summary of the calculated and RESCON model results of SBR and LTCR

Symbol	Unit	Calculated	RESCON Model	Symbol	Unit	Calculated	RESCON Model
SBR				**LTCR**			
W_{res}	(m)	70	70	W_{tf}	(m)	94.2	94.2
W_f	(m)	57.2	57.2	W_t	(m)	150	150
M_f	(tonnes)	1,122,102	1,122,102	A_f	(m^2)	3,030	3,030
M_{dep}	(tonnes)	939,000	939,373	A_r	(m^2)	4,725	4,725
SBR		1.19	1.19	LTCR		0.64	0.64

8.2.3 ECONOMICAL EVALUATION OF FLUSHING SYSTEM

The RESCON model is used for the economic evaluation of the flushing system. The economical evaluation is carried out for the existing tunnel and new tunnel options. Further sensitivity of the economical parameters is evaluated for unit value of the reservoir and the cost of capital investment required for implementing the flushing measures.

Cost Estimate for Flushing System

Cost estimate of the flushing systems are based on the layout of the system as presented in section 8.2.1. Unit rates for the cost estimate are adopted according to the current contractor's rate in Nepal for hydropower projects. The unit rate, quantity and cost estimate are presented in Appendix F. The total cost of the flushing system with existing tunnel and new tunnel are 2.48 and 3.52 mill. USD, respectively. The summary of the total cost of the flushing systems are presented in Table 8-3.

Table 8-3: Summary of the total cost of the flushing systems

Description	Existing	New
	Cost, (USD)	Cost, (USD)
Intake	211,600	211,600
Access to gate operation shaft	41,600	98,700
Vertical shaft for gate operation	179,800	208,100
Tunnel	418,400	1,110,700
Vertical shaft	185,500	-
Concrete plug	38,100	-
Treatment for abrasive resistivity of the surface	403,100	627,900
Radial gate	376,300	376,300
Stop log	183,000	183,800
Ladders and fencing	31,300	31,300
Total	2,069,300	2,931,700
Unforeseen/miscellaneous	413,800	586,400
Total cost of flushing system	**2,483,100**	**3,518,100**

Unit Value of the Reservoir Yield

The unit value of the reservoir capacity in terms of KWh/m^3 can be calculated as:

$$\text{Unit value of reservoir} = \frac{\text{Average annual electricity generation}}{\text{Effective reservoir capacity}}$$

Annual electricity generation from Kulekhani I power plant for the last 15 years (from 1997 to 2011) is shown in Figure 8-6. Annual generation from the power plant varies from minimum 75,114 MWh (in 2009/10) to maximum 249,680 MWh (in 2000/01) with the average 153,042 MWh.

The average effective reservoir capacity between 1997 and 2011 is 55.5 mill. m^3. Therefore, the unit value of reservoir = $153{,}042 \times 10^3/(55.5 \times 10^6) = 2.76\ KWh/m^3$.

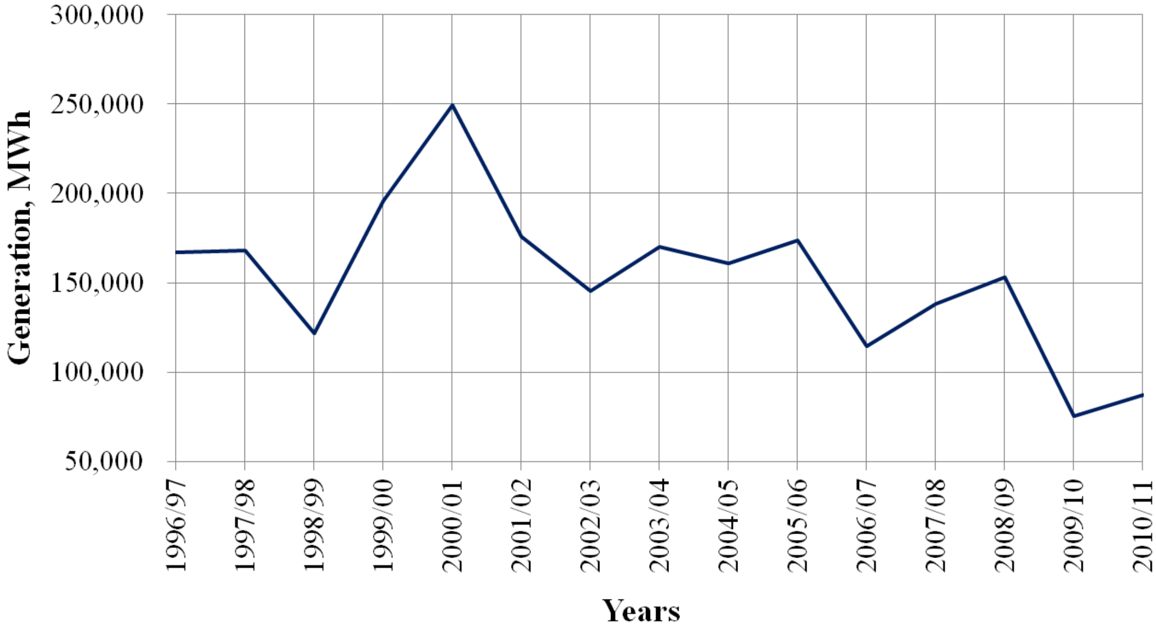


Figure 8-6: Annual generation from Kulekhani I power plant (modified from NEA, 2011)

As the power discharge volume is also used for power generation from Kulekhani II and Kulekhani III, the actual water value is calculated as:

For Kulekhani I and II,

2.76x(1+Net head for Kulekhani II/Net head for Kulekhani I)

$= 2.76 \times (1+284/550) = 4.18\ KWh/m^3$

For Kulekhani I, II and III,

$2.76 \times (1+284/550 + 102/550) = 4.69\ KWh/m^3$

According to NEA (2011) the average price of the electricity for 2011 is 6.72 NRs/KWh which is less than the buying rate (8.00 NRs/KWh) of electricity from independent power producers (IPP) during the dry season. As already mentioned earlier, the energy price from Kulekhani reservoir should be higher as it is the only storage power plant in Nepal and it generates most of the electricity during the dry period. If electricity price generated from Kulekhani reservoir is assumed 8.00 NRs/KWh, the unit value of the water is 37.54 NRs/m^3 or 0.47 USD/m^3.

Sensitivity Analysis

The sensitivity analysis is performed with the unit value of the reservoir yield with 0.39, 0.47 and 0.59 USD/m^3 assuming the energy price 6.72, 8.0 and 10 NRs/kWh, respectively, and with the cost of capital investment of 2.48, 3.52 and 4.5 mill. USD. The sensitivity analysis is carried out for flushing at minimum bed level (EL_{min}) of 1485 masl and water elevations (El_f) of 1490 masl at dam during flushing. The results of the sensitivity analysis are presented in Table 8-4.

Table 8-4: Summary of economic sensitivity analysis for Flushing System

Economical Results, Aggregate Net Present Value in mill. USD			
Cost of capital investment required for implementing flushing measures (mill. USD)	**Value of unit reservoir yield, USD/m^3**		
	0.39	0.47	0.59
2.48	358	436	555
3.52	357	436	554
4.50	356	435	553

Table 8-4 shows that the increment in the unit value of reservoir increases net present value (NPV) significantly. As the unit value of reservoir yield (P1) increases from 0.39 to 0.59 USD/m^3 (51%), the total net benefit increases by 55%. However, NPV does not change noticeably with increment of the cost of capital investment. It means the net benefit from the flushing system is much higher than the cost of capital investment required for implementing the system.

8.3 HYDROSUCTION SEDIMENT REMOVAL SYSTEM (HSRS)

Brief information about Hydrosuction Sediment Removal System (HSRS) is given in chapter 7.

8.3.1 HSRS PIPE LAYOUT OPTIONS

Shrestha (2001) and Sangroula (2005) recommended the following alternatives of HSRS flushing layout in the Kulekhani reservoir:

- Pipeline laid through the side spillway
- Pipeline along the spillway
- Tunnel boring along the left bank of the dam (between the dam and the spillway)
- Pipeline along the diversion tunnel

For reliability of the HSRS the pipeline should be located as low as possible to avoid vacuum in the pipe. There is a high possibility of vacuum development at the crest point of the pipe alignment if the pipeline is laid along the spillway. The water level in the reservoir should be high enough during the sluicing to avoid the vacuum to occur. The required level may not be available during the sluicing time. Therefore, pipe layouts along the spillways are not recommended for HSRS.

Tunnel boring along the left bank of the dam (between the dam and the spillway) is very risky with respect to the dam safety because of the development of vibration during tunnelling. Therefore, two alternatives of the pipe alignment have been chosen for HSRS:

1. Pipeline using the existing diversion tunnel No. 1
2. Pipeline constructing a new flushing tunnel

HSRS consist of four suction head, four flushing pipes of 0.5 m diameter, the inclined shaft, the flushing tunnel, the raft for suction head operation and floating drums for flushing pipe support. The layout and longitudinal section of the HSRS using existing tunnel No. 1 are shown in Figure 8-7 and Figure 8-8, respectively. Figure 8-9 shows the longitudinal section of HSRS with the new tunnel option. The only difference between the layouts is the use of the existing tunnel and construction of a new tunnel.

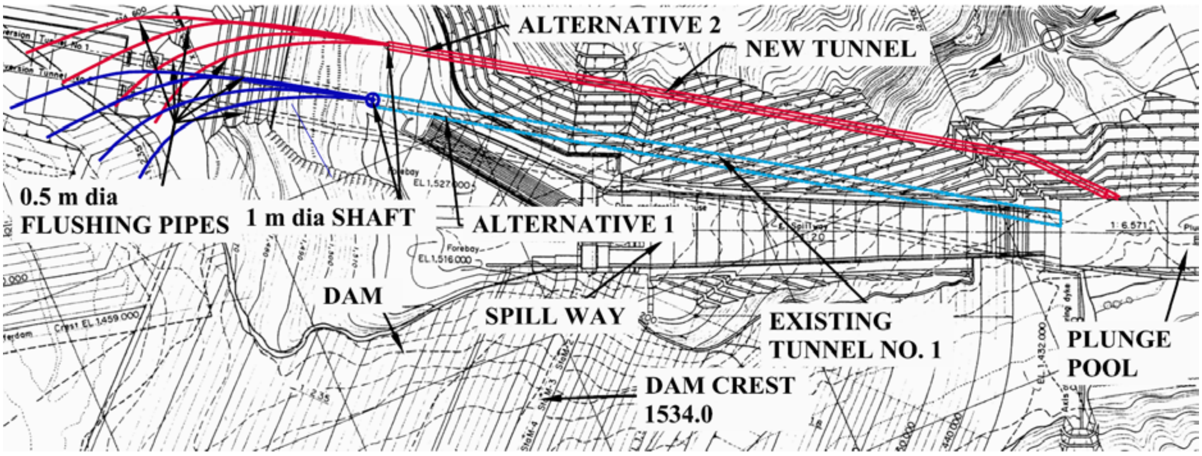


Figure 8-7: HSRS layout plan with alternatives

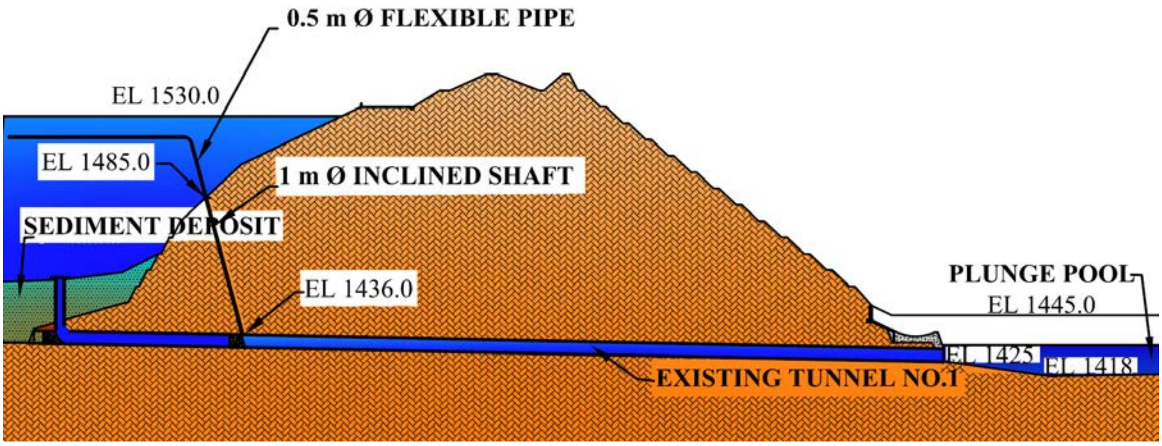


Figure 8-8: Longitudinal section of HSRS layout with existing tunnel No. 1 (Alternative 1)

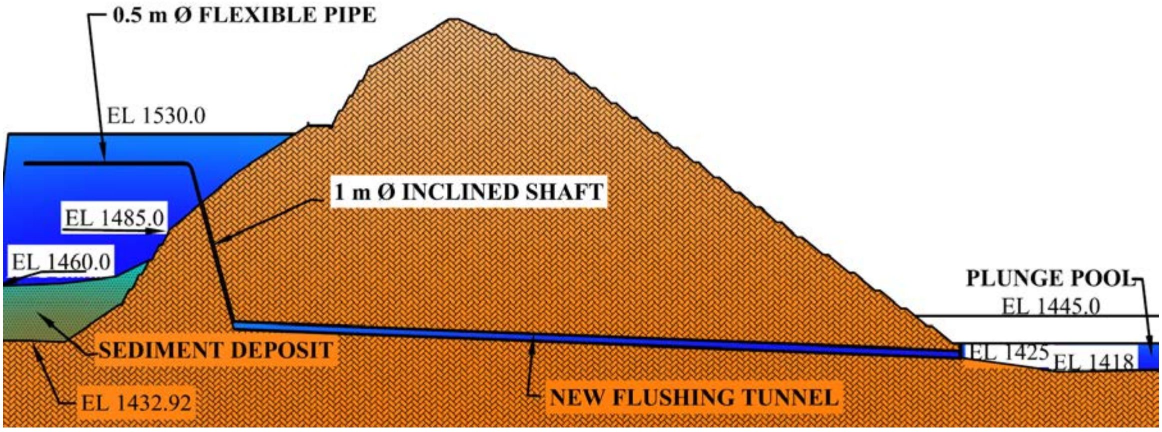


Figure 8-9: Longitudinal section of HSRS layout with new tunnel (Alternative 2)

8.3.2 SEDIMENT TRANSPORT ANALYSIS

Hydraulics of sediment transport refers to transport of sediment by using fluid i.e. water as a transporting medium. Factors that limit the rate of sediment transport in a conduit include the energy gradient, the size and shape of the conduit and the characteristics of the sediment. In hydraulic transport of sediment from the reservoir, the energy gradient is a limiting factor. The challenges are thus to optimise the conduit so that it can transport sufficient sediment, preferably with minimum use of water.

As a result of the analyses of different core samples from the Kulekhani reservoir, Sangroula (2005) reported that the D_{50} and D_{90} varied from 0.025 mm to 0.1 mm and 0.15 mm to 0.55 mm, respectively. The average D_{50} and D_{90} was 0.04 mm and 0.35 mm, respectively. The average dry density of sediment particles was 2.57 tons/m^3. Therefore, sediment particles from 0.025 mm to 0.2 mm and density of sediment particles with 2.6 tons/m^3 are adopted in the analysis. Although, the Darcy-Weisbach friction coefficient varies with pipe diameters and other characteristics, for simplicity, it is assumed to be 0.012. Field-tests of HSRS in the settling basins and peaking pond of Sunkoshi power plants show the sediment concentration by volume (C_v) variation from 5 per cent to 15 per cent by volume.

The pipe diameters of pipes 0.3 m, 0.4 m and 0.5 m are considered for the analysis as larger diameters are difficult to install and maintain.

Limit Deposit Velocity

The lowest velocity that sustains all material in motion without forming a stationary bed is known as the limit deposit velocity (V_L). This velocity is normally very close to the velocity at which the sediment is transported with a minimum head loss. The Durand equation is much used and is a rather simple way to determine the limit deposit velocity. However, Jacobsen (1997) suggested to increase V_L slightly which will reduce the possibility of blockage. As suggested by Jacobsen (1997), the velocity (V_L) is increased by 20% for further calculation.

Limit deposit velocity (V_L) can be expressed in terms of the factor F_L (Jacobsen, 1997):

$$V_L = F_L \sqrt{2gD(S_s - 1)} \qquad \text{Equation 8-1}$$

Where,

V_L Limit deposit velocity

F_L Durand factor based on grain size and volume concentration

g Acceleration due to gravity

D Diameter of pipe

S_s Relative density of solid; ρ_s/ρ_w

ρ_s Density of solids

ρ_w Density of water

The coefficient value F_L can be estimated from Figure 8-10. Durand (1953) developed the chart for the Durand factor F_L.

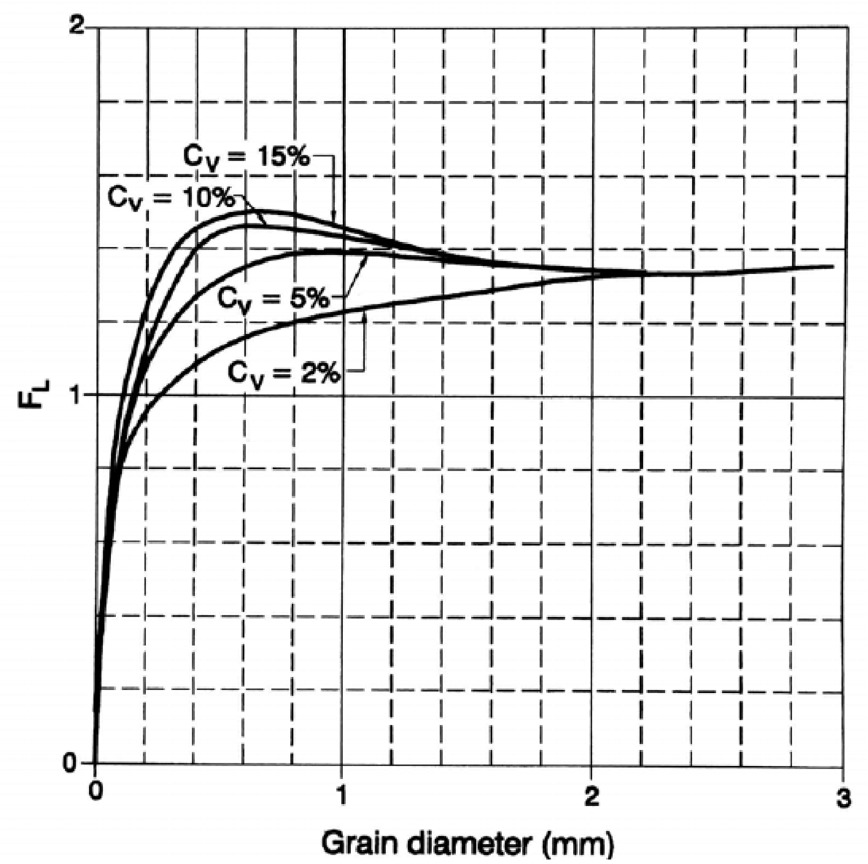


Figure 8-10: Coefficient F_L depending on grain size (Durand, 1953, ref. Vanoni 1975)

Figure 8-11 shows the relationship between the limit deposit velocities depending on the various particle sizes with 10 per cent sediment concentration by volume. It shows that the limit deposit velocity increases with increment of particle size and diameter of the pipes.

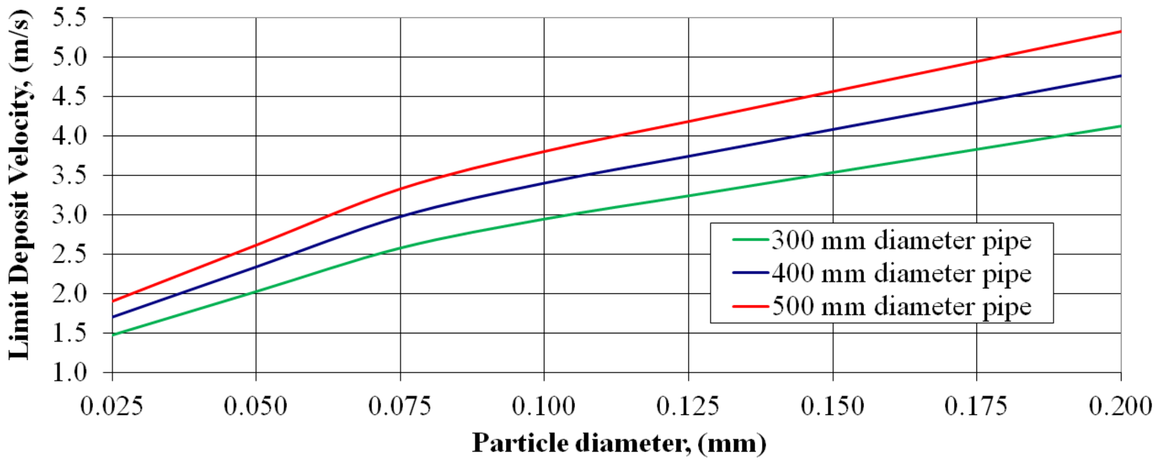


Figure 8-11: Limit deposit velocities for 10% sediment concentration by volume (C_V)

Pipeline Head Loss

One of the most frequently used equations for head loss in pipelines of water-sediment mixtures with a density of 2,680 kg/m^3 is the Durand-Condolios equation:

$$i_m = i\left\{1 + 176\left(\frac{\sqrt{gD}}{V}\right)^3 \left(\frac{1}{\sqrt{C_D}}\right)^{1.5} C_V\right\} \qquad \text{Equation 8-2}$$

Where,

$$i = f\frac{V^2}{2gD} \qquad \text{Equation 8-3}$$

f Darcy-Weisbach friction factor

V Velocity (m/sec)

g Acceleration of gravity

D Diameter of the pipe (m)

C_v Sediment concentration by volume

C_D Drag coefficient for d_{50}

Drag coefficient can be calculated from (Shook and Roco, 1991, ref. Jacobsen, 1997)

$$C_D = a_1 Ar^{b_1} \qquad \text{Equation 8-4}$$

Where,

a_1 and b_1 Constants taken from Table 8-5

Ar Archimedes number,

$$Ar = \frac{4d^3 g(S_s - 1)\rho_L{}^2}{3\mu_L{}^2} \qquad \text{Equation 8-5}$$

Where,

ρ_L Density of liquid and

μ_L Viscosity of liquid

Table 8-5: Constants for computing drag coefficient (C_D) for sand. (Jacobsen, 1997)

Range	a_1	b_1
Ar<24	576	-1
24<Ar<2760	80.9	-0.475
2,760<Ar<46,100	8.61	-0.193
46,100<Ar	1.09	0

Figure 8-12 shows the variation of energy gradient for water sediment mixture (slurry), i_m (m Water Column/m) for the different size of particles with different values of sediment concentration by volume, Cv.

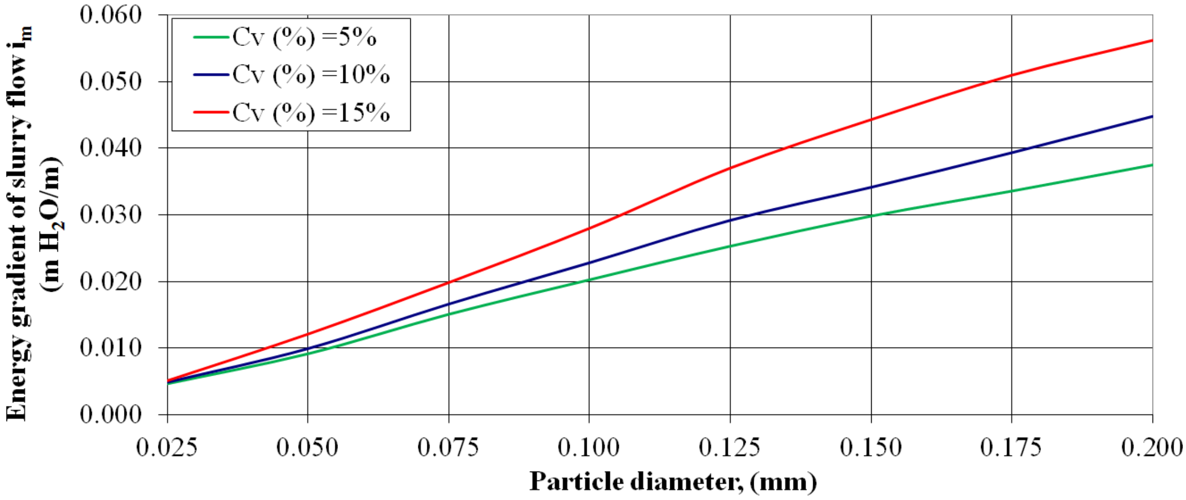


Figure 8-12: Energy gradient variation for sediment slurry

Figure 8-12 shows that head loss increases with increment of the particle size and the sediment concentration.

Sediment transport capacity

The summary of calculation of discharges of water (Q_w) and sediment deposit (Q_d) for different sediment particle size and various pipe diameters with 10% sediment concentration by volume are shown in Table 8-6.

The following steps are carried out to compute the discharge:

1. Discharge of sediment-water mixture (Q) is calculated as:

$$Q = V_L A \qquad \text{Equation 8-6}$$

Where

V_L: Limit deposit velocity

A: Area of the pipe

2. Discharge of sediment particles Q_s is calculated as:

$$Q_s = QC_v \qquad \text{Equation 8-7}$$

Where

C_v: Sediment concentration by volume

3. Discharge of sediment deposit (Q_d) is computed as:

$$Q_d = Q_s \frac{\rho_s}{\rho_d} \qquad \text{Equation 8-8}$$

Where,

ρ_s: Density of sediment particles (material density)

ρ_d; Density of sediment deposit

Table 8-6: Summary of sediment transport calculation through pipes

Sediment particle diameter d_i (mm)	Pipe Diameter D (mm)	Limit deposit velocity V_L (m/sec)	Energy gradient for clean water i (m H_2O/m)	Energy gradient for sediment water mixture i_m (m H_2O/m)	Discharge of water Q_w (m^3/hour)	Discharge of sediment deposit Q_d (m^3/hour)
0.05	300	2.03	0.008	0.010	464	103
	400	2.34			952	211
	500	2.61			1,663	369
	1,000	3.70			9,405	2,090
0.10	300	2.95	0.018	0.023	674	150
	400	3.40			1,384	308
	500	3.80			2,418	537
	1,000	5.38			13,680	3,040
0.20	300	3.98	0.032	0.043	910	202
	400	4.59			1,869	415
	500	5.13			3,265	725
	1,000	7.26			18,468	4,104

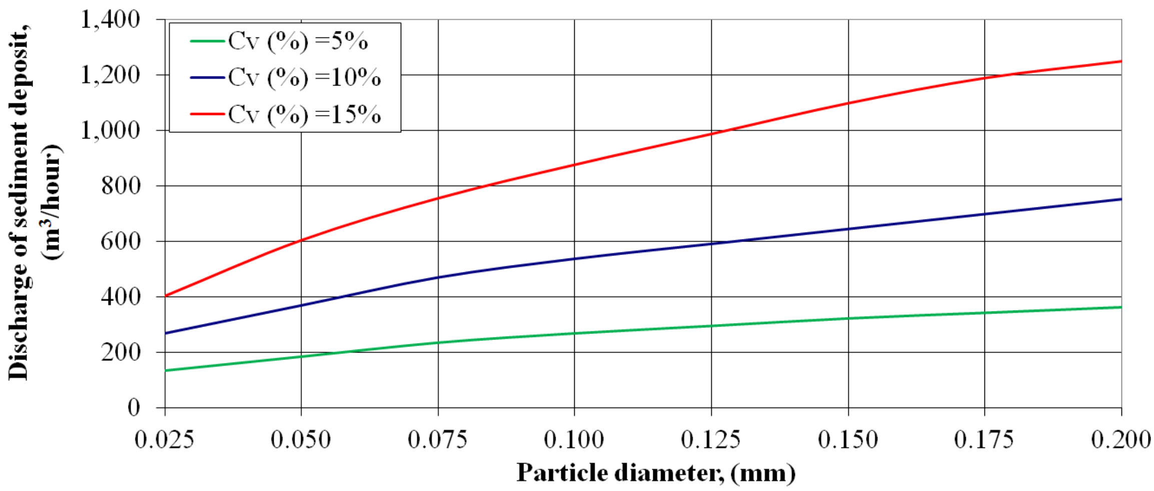


Figure 8-13: Discharge capacity curve for various sediment concentration and sediment particle size with 500 mm diameter pipe

Discharge capacity curve of sediment deposit for various sediment concentration and sediment particle size with 500 mm diameter pipe is shown in Figure 8-13.

Sediment transport analysis

The main objective of the analysis of the sediment transport is to determine the energy gradient of sediment water mixture (slurry), diameter of pipe and discharge of sediment deposit.

The schematic diagram of the proposed pipeline layout with hydraulic grade line and bed level parameters is shown in Figure 8-14. The water-sediment mixture is transported through the pipeline. The pipelines are floated below the hydraulic grade line with the help of rope and floating drums. The pipelines are connected at the entrance of inclined shaft at elevation of 1,485 masl. The inclined shaft is connected with flushing tunnel at 1,440 masl. The sediment deposit is flushed into the stilling basin along the inclined shaft and the tunnel. From the basin, evacuated sediment will be discharged to the river downstream.

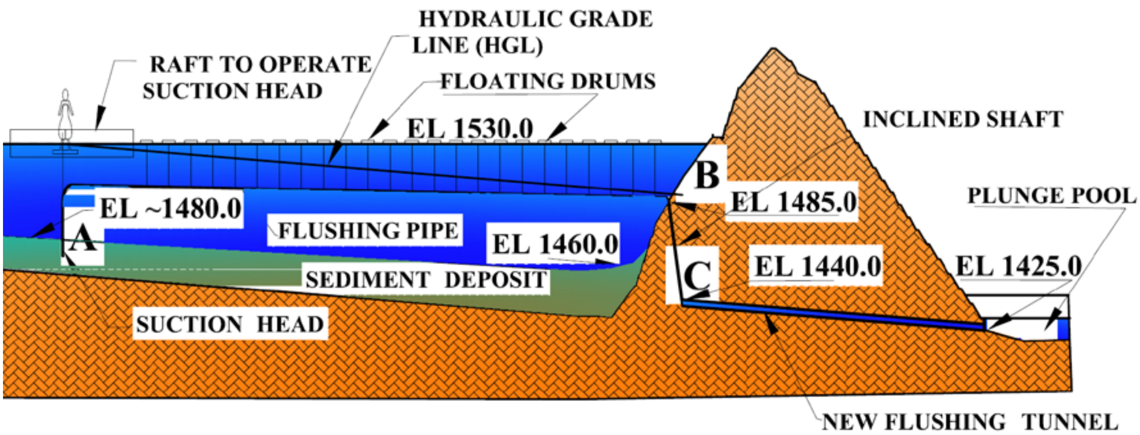


Figure 8-14: Schematic diagram of the pipeline layout

The maximum possible upstream distance from the entrance of the inclined shaft for sluicing the sediment deposit according to the available reservoir water level with particle size of 0.1 mm is shown in Figure 8-15. The distance is calculated with residual head of 2 m at entrance of inclined shaft (B). The summary of the computation is presented in Table 8-7.

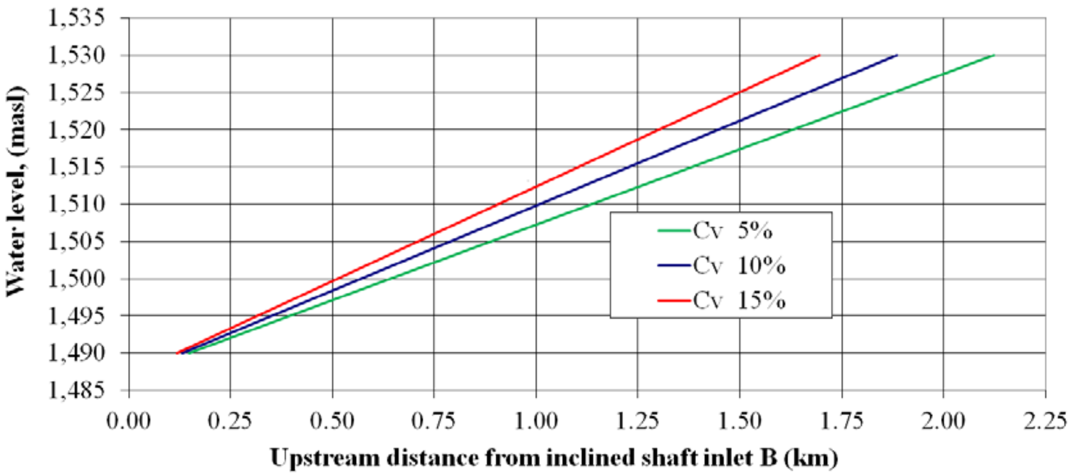


Figure 8-15: Upstream distance from inclined shaft entrance for sluicing

Table 8-7: Summary of upstream distance computation

Water level (masl)	Sediment particle diameter d_i =0.05 mm			Sediment particle diameter d_i =0.1 mm			Sediment particle diameter d_i =0.2 mm		
	C_v 5%	C_v 10%	C_v 15%	C_v 5%	C_v 10%	C_v 15%	C_v 5%	C_v 10%	C_v 15%
	Distance from dam (km)			Distance from dam (km)			Distance from dam (km)		
1530	4.70	4.33	4.01	2.12	1.88	1.69	1.15	1.01	0.90
1525	4.15	3.83	3.55	1.88	1.67	1.50	1.01	0.89	0.79
1520	3.61	3.32	3.08	1.63	1.45	1.30	0.88	0.77	0.69
1515	3.06	2.82	2.61	1.38	1.23	1.10	0.75	0.66	0.58
1510	2.51	2.32	2.15	1.14	1.01	0.91	0.61	0.54	0.48
1505	1.97	1.81	1.68	0.89	0.79	0.71	0.48	0.42	0.38
1500	1.42	1.31	1.21	0.64	0.57	0.51	0.35	0.30	0.27
1495	0.87	0.81	0.75	0.39	0.35	0.32	0.21	0.19	0.17
1490	0.33	0.30	0.28	0.15	0.13	0.12	0.08	0.07	0.06

Figure 8-15 and Table 8-7 show that it is possible to evacuate sediment from 1,880 m upstream of the dam with the sediment concentration by volume C_v = 10% for particle size, D_{50} =0.1 mm. From the analysis, it seems that the critical point is B (Figure 8-14) for the suction pressure and minimum water level of the reservoir. The upstream distance can be increased by allowing some suction at point B.

The average annual sediment deposit is about 760,000 m^3 based on the reservoir sedimentation survey in 2010. The sediment transport capacity analysis shows that the 500 mm diameter pipe can transport 537 m^3/hour with 10% sediment concentration (C_v) and 0.1 mm particle size (D_{50}). It will take about 1,450 hours to flush all the average annual sediment deposit with a 500 mm diameter pipe. The number of pipes can be increased to reduce the flushing time. For instance, four numbers of 500 mm diameter pipe can flush the annual sediment deposit in less than 360 hours, which is 45 days for 8 hours working hour in a day. Hence, HSRS is one of the long-term sedimentation solutions for the sustainability of the Kulekhani reservoir.

8.3.3 ECONOMIC ASSESSMENT OF HSRS

Cost of Sediment Removal

Cost estimate for the HSRS are based on the layout of the system as presented in section 8.3.1. Unit rates for the cost estimate are adopted according to the current contractor's rate in Nepal for hydropower projects. The unit rate, quantity and cost estimate are presented in Appendix F.

The following assumptions are made to estimate the cost of sediment removal:

- The amount of sediment deposit to be removed annually is assumed 760,000 m^3 and d_{50} of the deposit is 0.1 mm.
- Four 500 mm pipes with the 2,000 m of the pipeline are used, assuming that sediment deposit of 2 km upstream from the dam will be removed. Each pipe can

transport sediment deposits at the rate of 537 m^3/hour with a water discharge of 2,420 m^3/hour in the pipeline. Upstream sediment deposit will be transported to downstream to "cleared volume" in the dead storage.

- The amount of water required annually to remove the annual sediment deposit is:

  Water discharge x Time of flushing

  = 2,420 m^3/hour x 1,450 hours = 3,509,000 m^3 ≈ 3.51 mill. m^3

- The flushing is carried out during wet season. Therefore, the electricity rate for the removal of the sediment can be chosen from the electricity tariff rate of NEA during wet period i.e., 4.8 NRs/kWh. The annual cost of the water for sediment flushing is therefore, calculated as:

  3.51 mill m^3 x 2.16 kwh/m^3 x 4.8 NRs/kWh= 36.39 mill NRs (0.45 mill. USD)

  The energy equivalent (EE) includes the generation from Kulekhani I, Kulekhani II and Kulekhani III.

- The cost of the removal system and its operation and maintenance cost is difficult to assess correctly without a detail design of the system. Therefore, a simple way of assessing is used.

The summary of the total cost of the HSRS is presented in Table 8-8.

Table 8-8: Summary of the total cost of HSRS

Description	**Existing Tunnel**	**New Tunnel**
	Cost, (USD)	**Cost, (USD)**
Intake	2,300	2,300
Inclined Shaft	34,600	34,600
Concrete plug	38,100	6,500
Flushing tunnel	-	652,300
Treatment for abrasive resistivity of the surface	534,400	350,600
Hydro-mechanical parts	43,800	43,800
Steel pipes	60,300	60,300
Four suction heads, outlets sections, valves	100,000	100,000
High quality suction hose pipe	125,000	125,000
Flexible high quality pipe	1,125,000	1,125,000
Floaters for pipe suspension, Rafts to handle the suction head	50,000	50,000
Grouting	35,000	35,000
Sub total	2,148,600	2,585,500
Unforeseen/miscellaneous	429,700	517,100
Total cost of Flushing System	2,578,300	3,102,600

Value of Recovered Reservoir Volume

The value of the recovered reservoir volume is computed as:

- The unit value of the water is adopted 37.54 NRs/m^3 or 0.47 USD/m^3 as calculated in Chapter 8.2.3.
- Annual deposition of sediment is 0.76 mill m^3. If all the annual sediment is removed and 75 per cent of the removed volume is utilised for energy generation, the annual value of the extra production is therefore:

  75% of 0.76 x 37.54 = 21.40 mill NRs (0.27 mill. USD)

If the sediment control measures for the reservoir sedimentation are not applied, the reservoir volume loss will be 0.76 mill m^3 each year. Therefore, for the economic analysis, the value of the extra production is added each year.

Economic Analysis

The objective of an economic analysis is to determine the economic soundness and economic viability of a proposed sediment removal system in order to justify its investment.

The three discounting technique, Net Present Value (NPV), Internal Rate of Return (IRR) and the Benefit to Cost Ratio (B/C) are used to evaluate the economic viability of the HSRS.

NPV - This is the net benefit in each year discounted over project lifetime.

$$NPV = \sum_{t=1}^{t=n} \frac{Bt - Ct}{(1+i)^t}$$

IRR - This is the discount rate for which NPV is equal to zero.

IRR for $NPV = \sum_{t=1}^{t=n} \frac{Bt - Ct}{(1+i)^t} = 0$

B/C - This is the ratio of discounted benefit to discounted cost over project life.

$$B/C = \frac{\sum_{t=1}^{t=n} \frac{Bt}{(1+i)^t}}{\sum_{t=1}^{t=n} \frac{Ct}{(1+i)^t}}$$

Where

B_t: Benefits in each year

C_t: Cost in each year

t : 1, 2, …., n

n: Number of years

i: Discount rate

For the proposed project to be economically viable and financially sound, the following criteria is used:

IRR $\geq$ discount rate

NPV $\geq$ 0

B/C $\geq$ 1

For the analysis, the following data have been used for the base case:

- Total cost of the system: 248.2 mill NRs (3.10 mill. USD)
- Discount rate: 10%
- Interest during construction: 10%
- Operation and maintenance cost: 100,000 USD
- Energy price for the dry season: 10 cents/kWh
- Energy price for the wet season: 6 cents/kWh
- Construction period: 2 years
- Economical life of the HSRS system: 20 years

The cash flow chart for the base case is shown in Figure 8-16.

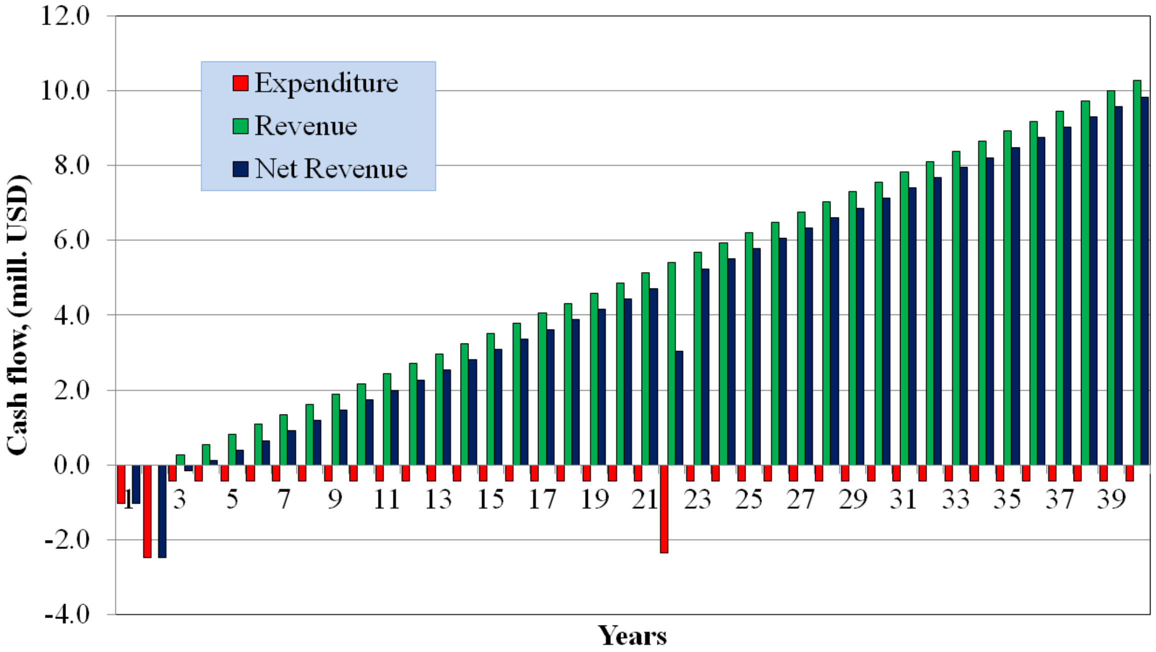


Figure 8-16: Cash flow chart for base case

The computed Net Present Value (NPV), Internal Rate of Return (IRR) and the Benefit to Cost Ratio (B/C) are 14.94 mill. USD, 24.7% and 2.24, respectively.

Sensitivity Analysis

The sensitivity analysis is performed with the increment by 10%, 20% and 30% of the total cost of HSRS, the annual cost of the water for sediment flushing, interest rate during construction (IDC) and discount rate. However, the value of the recovered reservoir volume is declined by the same percentage. The summary of the sensitivity analysis are presented in Table 8-9.

Table 8-9 shows that NPV, IRR and B/C are 1.25 mill USD, 16% and 1.31, respectively though the expenditure is increased by 30% and revenue is decreased by 30%.

Table 8-9: Summary of economic sensitivity analysis for HSRS

Description	Scenario			
	Base	Increment		
Input		**10%**	**20%**	**30%**
HSRS total cost, (mill. USD)	3.10	3.41	3.72	4.03
HSRS equipment cost, (mill USD)	1.75	1.93	2.10	2.28
Cost of water for sediment flushing, (mill USD)	0.46	0.50	0.55	0.59
Value of the recovered reservoir volume, (mill USD)	0.27	0.24	0.22	0.19
Interest during construction	10%	11%	12%	13%
Discount rate	10%	11%	12%	13%
Output				
NPV, (mill USD)	13.94	8.63	4.50	1.25
IRR	24.7%	21.5%	18.7%	16.0%
B/C	3.24	2.41	1.78	1.31

The results of the economic analysis show that the proposed sediment removal system (HSRS) is economically sound and viable. However, further investigations into the economic viability of the system should be studied when detailed investigation and design drawing with estimated cost of the system will be available.

8.4 CONCLUSION

The storage capacity the Kulekhani reservoir is deteriorating by one per cent in average every year. As the Kulekhani hydropower project plays a very significant role in the Nepalese energy sector, its sustainability is an important issue. Historical operation data of the Kulekhani reservoir shows that about 28 mill. m^3 water is being used every year for electricity generation during the monsoon (June to August) (Chapter 5). This water can be used for sediment handling every year.

Based on the above analysis, both, the reservoir flushing as well as HSRS seems to be feasible for the Kulekhani reservoir. Conventional flushing is not recommended as preferred sediment management option due to the following reasons:

- The current capacity inflow ratio (C:I) of the Kulekhani reservoir is about 0.5, but the capacity inflow ratio of the reservoir should not be more than 0.2 for the flushing (Atkinson, 1996)
- The sustainable long-term capacity with the flushing system is only 54 mill. m^3 (LTCR = 0.64) which is less than the capacity with HSRS (64 mill. m^3)
- Generally, flushing is not considered as environmentally friendly sediment management option

HSRS can evacuate sediment deposit from the downstream area of the reservoir (about two km from the dam) and sediment from the upstream parts will be transported from the live storage to "cleared volume" in the dead storage. It is possible to evacuate the total incoming average annual sediment load from the reservoir every year with HSRS. HSRS can establish sustainable reservoir with present storage capacity.

The hydrosuction removal system (HSRS) is economically attractive and viable for the sediment handling in the Kulekhani reservoir. Hence, HSRS is one of the long-term sedimentation solutions for the sustainability of the Kulekhani reservoir.

9 CONCLUSION AND RECOMMENDATION

9.1 CONCLUSION

As conclusions have been drawn in respective chapters, only the main conclusion is presented here.

The main factors that affect the accuracy of survey data or volume computations from the bathymetric survey are terrain irregularity, data density and track lines of the survey. These effects can be minimized by increasing survey point data density with coverage of all area of the reservoir.

The Kaligandaki A and Middle Marsyangdi reservoirs have lost 51% and 65% of their total volume, respectively. The total losses within the live storage are 6.7% and 14.1% of the total live storage capacity, respectively. Still, both power plants can generate energy at their full capacities during the peak load demand in Nepal with the remaining live storage. Hence, reservoir capacity loss within the dead volume in the peaking reservoir does not affect generation capacity.

The bathymetric surveys of the Kaligandaki A and Middle Marsyangdi reservoirs carried out by the author in 2010 can be considered as a base line survey for future research and study of the reservoirs.

The annual average thickness of the sediment deposition in the Kulekhani reservoir is 0.16 m from 1996. Similarly, the annual average sediment deposition volume is 0.56% of total reservoir capacity.

It is found out that the total volumes of the Kulekhani reservoir based on the 2009 and 2010 surveys are more than the total volume from 2003 and 2004 surveys and NEA data. The main reason of such difference in volume computation may be the error in survey due to the differences in density of survey points, survey tracks and reservoir levels during the survey.

The excel model, developed for the optimum operation of the major hydropower plants of Nepal, shows that the load shedding could be totally avoided during the wet period and could have been reduced significantly during the dry period in 2006/07 and Kulekhani reservoir could have been operated at higher reservoir levels.

If the hydropower plants of NEA were operated and maintained properly, they could generate with their full capacity and the load shedding could be reduced significantly. The large amount of load shedding in Nepal was due to the capacity deficiency during the peak hours in October-November and March-April. Development of PRoR instead of RoR helps to reduce the load shedding during this time.

Based on the RESCON analysis, both flushing and HSRS sediment management options seem to be feasible for the Kulekhani reservoir. However, HSRS is found to be best sediment management option in regards with economy.

The field test of HSRS indicates that it is a promising technology and has relatively low-cost and low-power requirement system for the sediment removal from the reservoirs and peaking ponds. The field test study showed that the efficiency of the hydrosuction sediment removal could be increased if it is combined with water jetting to break consolidated sediment deposits. The efficiency and performance of the HSRS can also be improved with the adjustment of the opening between inner and outer pipe according to the consolidation level of sediment deposit.

The HSRS can evacuate sediment deposit from the downstream area of the reservoir (about two km upstream from the dam), and sediment from the upstream parts will be transported from the live storage to "cleared volume" in the dead storage. In principle, it is possible to evacuate the total incoming average annual sediment load from the reservoir every year with HSRS. HSRS can help to establish sustainable reservoir with present storage capacity.

HSRS is economically attractive and viable too for the sediment handling in the Kulekhani reservoir. Hence, HSRS is one of the long-term sedimentation solutions for the sustainability of the Kulekhani reservoir.

9.2 RECOMMENDATION

The accuracy of the bathymetric survey can be improved adopting the proper survey track lines and point density. For example:

- The survey track lines should be parallel to the shoreline.
- Density of the survey points should be defined according to the depth gradient and irregularity of terrain. The distance between the two survey points along the track lines (a) and distance between track lines (b) should be smaller when the depth gradient and irregularity of terrain are high.
- The distance between the track lines should be closer in the shoreline area with high depth gradient than the middle part of the reservoir.

To determine the schedule and frequency of bathymetric survey of the reservoir, sediment yield, the capacity reduction rate and sediment deposition thickness should be assessed with respect to the accuracy of the survey technique.

In daily peaking reservoirs which are designed to regulate only during the peak hours of a day, the reservoir surveys should be conducted to determine the sediment deposition pattern, amount of flushed sediment after flushing, the shift in the sediment deposition area and capacity-elevation curves.

The annual average thickness and volume of the sediment deposition in the Kulekhani reservoir are lower than the average error of the bathymetric survey with the DGPS method. According to the current sediment yield, capacity reduction rate and sediment deposition thickness in the Kulekhani reservoir and the accuracy of the DGPS bathymetric survey, the bathymetric survey in the Kulekhani reservoir should be conducted at the interval of at least five years.

The power plants of NEA could not generate with its full capacity due to technical deficits at several power plants, operation and maintenance problems, management constrain and unavailability of spare parts. Therefore, these problems should be addressed to achieve improved performance of the power plants.

In RESCON model, the calculated sediment concentration through Hydrosuction Pipe of HSRS is very low compared to the sediment concentration achieved in the field work. Therefore, quantity of sediment removed by HSRS method could be very high compared to that calculated by RESCON method. Hence, the technical calculation of sediment transport and sediment concentration in RESCON method should be reviewed and updated.

The bathymetric surveys in the Kulekhani reservoir show that sedimentation level around the intake area is increasing and approaching to the intake level. To prevent further deposition in the intake area, hydraulic dredging can be used to shift the sediment deposits from the intake area to dead volume area to the left bank of the reservoir, close to the dam and spillway area.

The adjustment of the opening the gap between the inner and outer pipe of MDLSS is recommended to achieve the optimum concentration during sediment removal.

The study of the sediment management in the Kulekhani reservoir shows that HSRS is the best feasible method for removing sediment deposit from the lower portion of the reservoir. The field test also proves that it is possible to remove even consolidated sediment deposit from the bottom of reservoirs. Therefore, it is recommended to carry out the detail study of HSRS to evacuate the sediment deposition from the Kulekhani reservoir.

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APPENDIX A

RESERVOIR SEDIMENTATION AND SEDIMENT HANDLING IN RESERVOIRS

A.1 GLOBAL WATER RESERVOIRS

Several researchers and organizations have created their own global and regional spatial data sets of dams and reservoirs, mostly by identifying the largest of them on paper maps and compiling information from various sources including national archives and the Internet. These databases vary in their number of records, the quality of attribute data, and their spatial resolution. The overall world storage capacity estimated by Mahmood (1987), Morris & Fan (1998), White (2001), Palmeri *et al*. (2003), ICOLD (2006) and Lehner B. et al. (2011) are 4,880, 7,000, 6,815, 7,000, 7,000 and 6,197 km^3 respectively.

The most comprehensive global dam database has been compiled by the International Commission on Large Dams (ICOLD). According to ICOLD (2006) there are about 50,000 dams higher than 15 m and / or storing over 3 mill. m^3 of water called "large dams". There are over 100,000 smaller dams with storage over 100,000 m^3 and millions under 100,000 m^3. The overall storage capacity is close to 7,000 km^3 of which 98 % for the "large dams". The live storage is in the range of 4,000 km^3 i.e. 10 % of the worldwide yearly rivers flow. The overall area of reservoirs is 500,000 km^2, one third of the area of Earth's natural lakes.

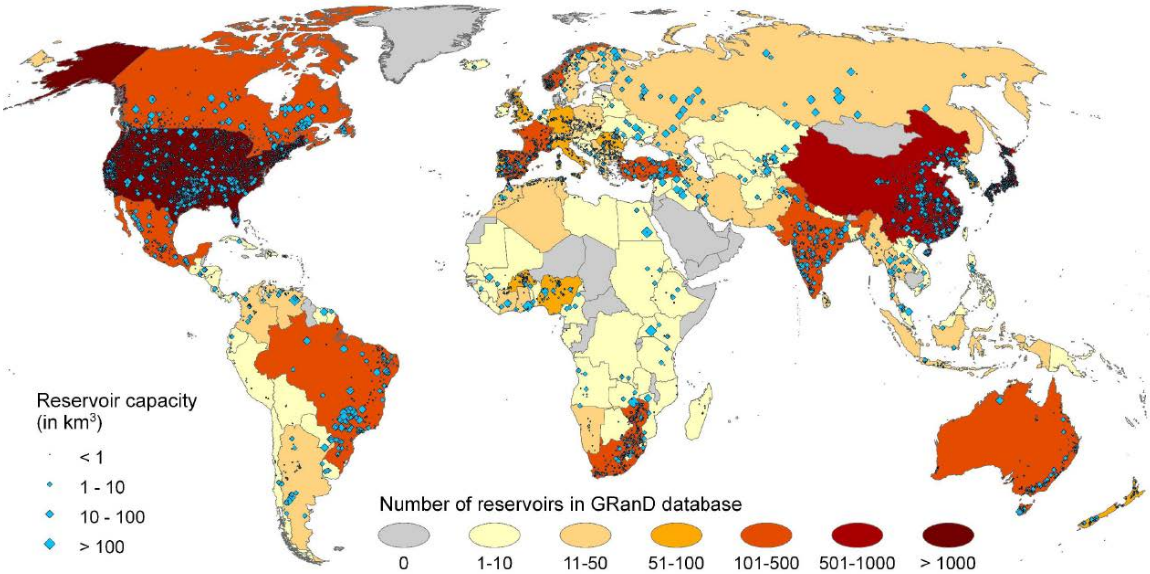


Figure A-1: Global distribution (by country) of large reservoirs in GRanD database (Lehner B. *et al.*, 2011)

The Global Water System Project (GWSP), a joint project of the Earth System Science Partnership (ESSP), has produced the Global Reservoir and Dam database (GRanD), a high-resolution global database of large dams and reservoirs (Figure A-1). The GRanD Database provides the location and main specifications of large global reservoirs and dams with a storage capacity of more than 0.1 km^3. GRanD shows the global distribution of large dams and reservoirs. According to *GRanD database – Technical documentation*

– *Version 1.1* published on March 2011, GRanD currently contains information regarding 6,862 dams and their associated reservoirs with a total storage capacity of 6,197 km^3.

The historic rate of construction of storage worldwide is shown in Figure A-2. The overall annual growth rate for the century as a whole has been 6·5%. Development of reservoir was slow until mid 1950s when large projects began to build. Dam building continued apace until the 1980s. The number of large dams completed per year during the 1960s or 1970s was five or ten times the present number, and this is often used to demonstrate "the dramatic decline in the trend of dam building over the past two decades" (Lempérière, 2006).

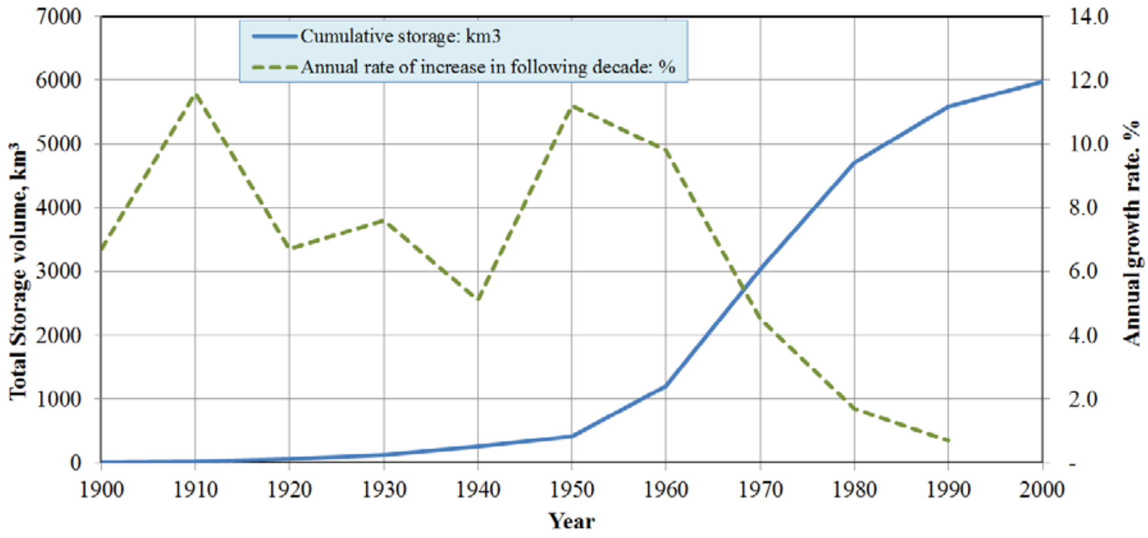


Figure A-2: Historic growth in reservoir storage (White, 2001)

A.2 Reservoir Sedimentation In Global Perspective

Reservoir sedimentation caused by sediment particles from erosion processes in the catchment are propagated through the river flow. When the flow of a river is stored in a reservoir, the sediment builds up in the reservoir and reduces its capacity. According to the most comprehensive study, the global erosion rate is 0.09 mm/year corresponding to 132 t/km^2/year (White, 2010).

The rate of erosion is very variable and depends on a complex interaction of the following factors (White, 2010):

- Climate: precipitation and run-off, temperature, wind speed and direction
- Geotechnics: geology, volcanic and tectonic activity soils
- Topography: slope, catchment orientation, drainage, bassin area, drainage density
- Vegetation cover
- Land use and human impact

The estimates as presented by Mahmood (1987), Atkinson (1996) and White (2001) present a wide range of reservoir storage loss. It is interesting to note that the annual loss of world storage estimated by White (2001), 0.48% per annum, is about half that estimated by Mahmood (1987) and Basson (2008), who quote a figure of 1% and 0.96% respectively. Mahmood (1987) roughly estimates the storage loss as 1% of the world's gross reservoir storage in 1986. The same percentage of storage loss is quoted by Atkinson (1996), and by Morris & Fan (1998). White (2001) has updated this figure, through review of data of 2,300 dams, in 31 countries and information from various sources. The summary of the results of the analysis is given in Figure A-3 White (2001). He estimated a total loss of storage of 567 km^3 since start of twentieth century out of gross reported storage volume of 5,976 km^3. It amounts to approximately 10% of gross storage loss by year 2000.

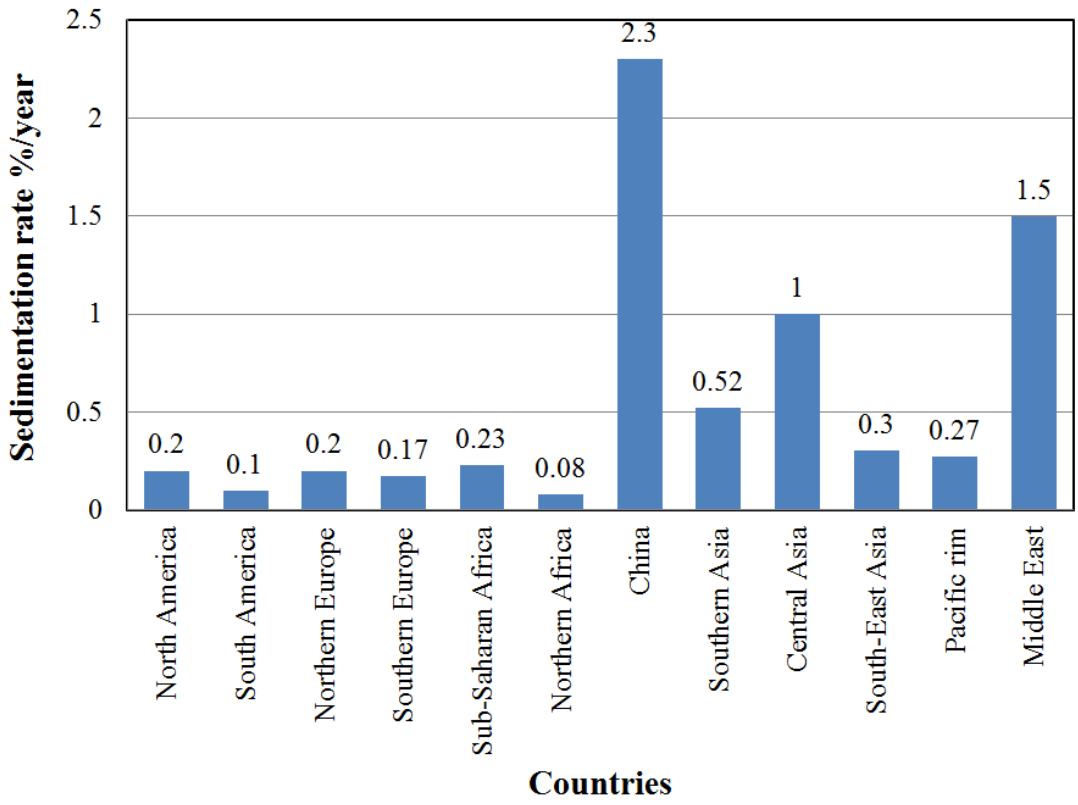


Figure A-3: Regional sedimentation rates (Modified from White, 2001)

Gerrit Basson (2008) gave an overview on global sedimentation rates and loss of storage volume in the International Workshop Erosion, Transport and Deposition of Sediments, hosted by the International Sediment Initiative (ISI) in April 2008 in Berne. The predicted average global sedimentation rate lies at 0.96 % and by the year 2050, a large number of countries could reach critical sedimentation volumes, such that reservoirs cannot fulfill their function anymore. The observed storage capacity loss due to reservoir sedimentation of man-made reservoirs is shown in Figure A-4.

The global distribution of the rate of loss of storage varies considerably. China and India are losing 2.9% and 0.72% of storage capacity annually, but the annual loss rate respectively in Japan and Korea are only 0.42% and 0.26%. Lowest sedimentation observed in Egypt (0.05%) and Germany (0.17%).

According to Basson (2010), 35% of the total storage capacity for hydropower had been filled with sediment in 2006, increasing to 70% by the year 2050. For dams for any other purpose (non-hydro), in 2006, 33% of the available capacity was filled with sediment, rising to a predicted value of 62% by 2050. It is expected that non-hydro dams will be severely impacted on when they reach a 70% sedimentation level. At this sedimentation level there will be about a 40-50% water yield reduction, and there could be problems at the intakes. Based on the global data this will occur by the year 2065, and will occur per region as indicated in the Table A-1.

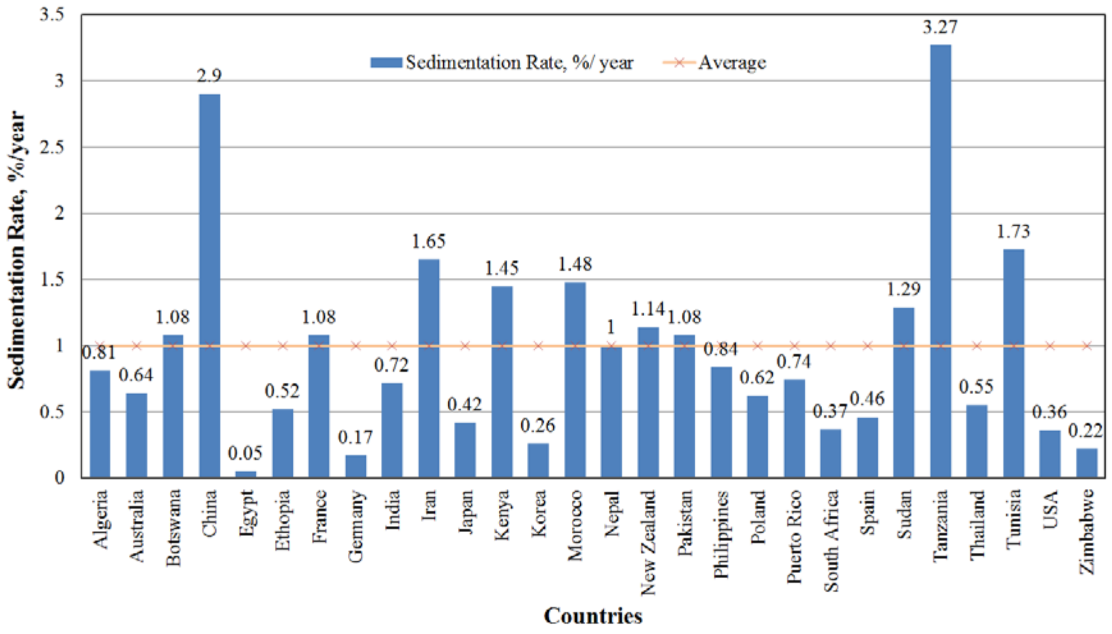


Figure A-4: Observed Reservoir Sedimentation Rate (Modified from Basson, 2008)

"Hydropower dams can generally be filled with sediment to a higher level than non-hydropower dams, as it is mainly necessary to maintain the head for the power generation, and a storage capacity sufficient to meet all expected demands for power. It is expected that hydropower dams will be severely impacted when they reach a level of sedimentation of 80%. Based on the global data this will occur by the year 2070, and per region as indicated in the Table A-1" (Basson, 2010).

The global reservoir sedimentation rate is shown in Figure A-5 and the year that current storage will reach critical sedimentation levels is presented in Table A-1.

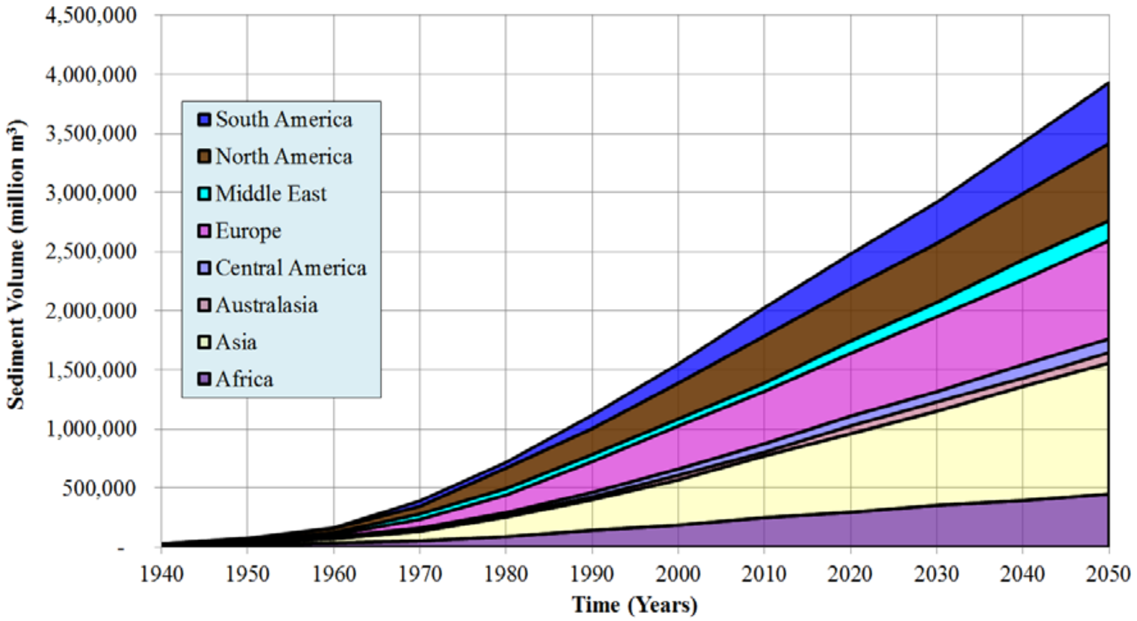


Figure A-5: The global reservoir sedimentation rate (Modified from Basson, 2010)

Table A-1: The year that current storage will reach critical sedimentation levels

Region	Hydropower dams: Date 80 % filled with sediment	Non-hydropower dams: Date 70 % filled with sediment
Africa	2,100	2,090
Asia	2,035	2,025
Australasia	2,070	2,080
Central America	2,060	2,040
Europe and Russia	2,080	2,060
Middle East	2,060	2,030
North America	2,060	2,070
South America	2,080	2,060

According to White (2001), the rate of loss of storage varies substantially and shows a striking inverse relationship between the rate of loss of storage and reservoir capacity, as shown in Figure A-6. The highest rates of loss of storage are found in the smallest reservoirs and the lowest rates in the largest. As part of this study, data from 1,105 reservoirs studied, 730 have a storage volume of less than 1,233 mill. m^3 and an average rate of loss of storage in excess of 1 % per annum. At the other extreme, 23 of the reservoirs studied had a storage volume in excess of 1233 mill. m^3 and an average rate of loss of storage of 0.16% per annum (White, 2001).

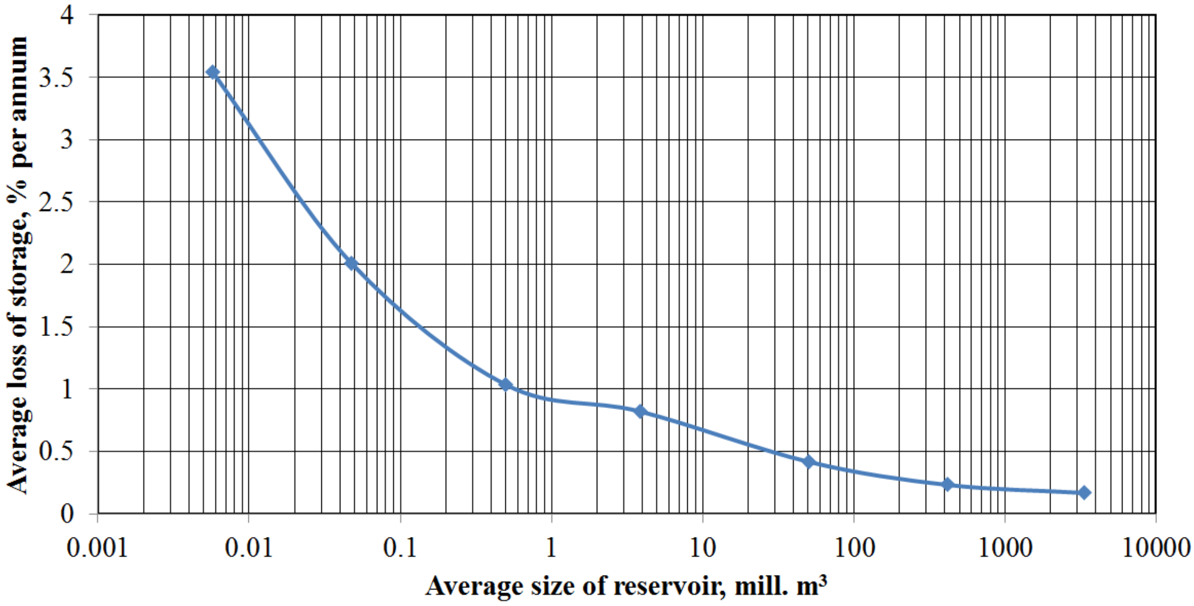


Figure A-6: Reservoir size and rate of loss of storage

According to François (2006), reservoir sedimentation problem was underestimated worldwide 40 years ago, because it was a minor issue in most countries where the largest dams were built at that time, and because most studies focused on short-term benefits. This problem has now been underlined by difficulties encountered with some large reservoirs; it is now often globally presented, or at least understood, as one of the main reasons for alleged non- sustainability and limited future of dams. A simplified presentation is the following: “existing reservoirs lose 0.5 to 1 per cent of their storage yearly and are on average already 35 years old; the best dam sites have already been used and the yearly increase of storage does not and will not balance the yearly loss by sedimentation". It is thus understood that the existing reservoirs will have lost more than 50 per cent of their capacity before 2050 and will have little usefulness by 2100.

A.3 SEDIMENT TRANSPORT IN NEPALESE RIVER

Sediment transport rates are high in Himalayan Rivers and specific sediment transport rates exceeding 10,000 t/km^2/yr is reported (WECS, 1987). A few examples of sediment transport rates in some rivers in Nepal are shown in Table A-2; but it is worth to note that only the data from Kulekhani are based on measurements in a reservoir.

Table A-2: Sediment transport rates in some rivers in Nepal. (Jacobsen, 1997)

River:	**Kulekhani**	**Sun Koshi**	**Marshyangdi**	**Arun**	**Tamur**
Water runoff (10^6 m^3/yr)	137	10,800	6,600	15,400	8,100
Sediment runoff (10^6 m^3/year)	1.6	72	30	46	39
Spec. sediment runoff (t/km^2/yr)	12,400	3,800	7,700	1,320	6,800

Sediment loads for some major rivers recorded by Department of Hydrology and Meteorology (DHM) are given in the Table A-3. The table shows that the sediment yield varies from 175 to 8,284 tonnes/km^2. But the sediment yield measured in Kulekhani Reservoir and Kulekhani station is far different. The sediment yield measured in Kulekhani reservoir is 12,400 tonnes/km^2 but only 175 tonnes/km^2 in Kulekhani station.

Table A-3: Sediment load in some of the major rivers of Nepal (Sharma, 2001).

S. No.	**Station no**	**River**	**Area, (km^2)**	**Sediment transport, (million tonnes)**	**Sediment yield, (tonnes/km^2)**
1	795	Kankaii at Mainachuli	1,148	5.83	5,078
2	240	Karnali at Asara	19,260	17.20	893
3	280	Karnali at Chisapani	42,890	105.00	2,448
4	430	Seti at Pokhara	582	3.31	5,687
5	260	Seti at Banga	7,460	19.00	2,547
6	570	Kulekhnai at Kulekhani	126	0.02	175
7	350	Rapti at Bagasoti	3,512	16.60	4,727
8	410	Kaligandaki	7,310	32.20	4,516
9	447	Trishuli at Trishuli	4,640	3.41	735
10	470	Lothar at Lothar	169	1.40	8,284
11	550	Bagmati at Chovar	585	0.76	1,308
12	450	Narayani at Narayanghat	31,100	176.00	5,659

A.4 MEASURES AGAINST RESERVOIR SEDIMENTATION

If there are no effective measures undertaken, many reservoirs during the next 50 years will seriously be affected by sedimentation and the major part of the worldwide useful volume will be lost by the end of the 21st century.

The rate of development of new reservoirs is decreasing. This is because of lack of reservoir sites, environmental and social costs of reservoir development and the increasing role of private financing, which favours smaller, less controversial and less time-consuming projects. The amount of fresh water circulating through the hydrologic cycle is fixed, but the demand for water use has grown rapidly during the twentieth century and fresh water is becoming an increasingly scarce resource (Morris & Fan, 1998).

The cost of replacement storage is much debatable. For example, Mahmood (1987) calculated it to be USD 6 Billion per annum, White (2001) mentioned it to be about USD 360 Million per annum and Palmieri *et al.* (2003) quoted it to be USD 13 Billion per annum. According to Basson (2010) the total yearly loss linked with sedimentation problems is about USD 15 Billion (excluding downstream impacts) and with downstream impacts considered the annual cost is about USD 17 Billion.

According to ICOLD (2006), within next five decades, the role of dams will probably double for hydropower and will much more than double for water supply, dry season releases and flood mitigation. The storage for irrigation may also double with a better utilization i.e for up to two Billion people. However, the total present water storage, which is presently close to 7,000 Billion m^3 will probably increase only by about 3,000 Billion. The average cost per m^3 of reservoirs built in the 21st century will be much higher than between 1950 and 2000 because the yearly investment may remain the same for 100 years instead of 50 years and for a storage of 4,000 km^3 instead of 7,000 km^3. The overall benefits of dams during the 21st century will be five times what they have been since 1950.

There is a strong need to limit sediment accumulation in reservoirs in order to ensure their sustainable use. Management of sedimentation in any reservoirs cannot be performed by a standard generalized rule or procedure. Furthermore, sediment management is not limited to the reservoir itself. It begins in the catchment areas and extends to the downstream river. Every situation has to be analysed for itself in order to determine the best combination of solutions to be applied. It happens that the simultaneous implementation of several alternatives is necessary in order to stabilize reservoir sedimentation.

Palmeri *et al*. (2003) considered several ways of sediment management techniques in the Reservoir Conservation (RESCON) model. The techniques can be categorized as follows:

1. Reducing sediment inflows
    - watershed management
    - upstream check structures
    - reservoir bypass
    - off-channel storage
2. Managing sediments within the reservoir
    - operating rules
    - tactical dredging
3. Evacuation of sediments from the reservoir
    - flushing
    - sluicing
    - density current venting
    - mechanical removal (dredging, dry excavation, hydrosuction)
4. Replacing lost storage

- increased dam height
- construction of new dam

5. Decommissioning

Reducing sediment inflows technique includes watershed management issues such as encouraging people upstream to get involved in the practices that are not going to contribute to soil erosion. The other alternative is trapping sand before it reaches reservoirs; for example, by constructing check dams, though they are not very effective. The purpose of a bypass is to divert sediment laden flood flows around a reservoir. By-passing a reservoir by making use of conveyance structures is often only feasible when favorable hydrological and morphological conditions exist (Palmeri *et al.*, 2003). Off-channel storage reservoirs are built adjacent to the main river channel (e.g., a small tributary or on the flood plain). Water from the main river is diverted into the reservoir during times of low sediment concentrations.

If the reservoir level is kept high during the flood season, incoming sediments will tend to deposit in the upstream of the reservoir. This holds the sediments upstream from the Intake area but the sedimentation occurs in the live storage. Conversely, if the reservoir level is operated at minimum level during the flood season, the incoming flows will erode the previously deposited delta and move the sediments towards the dead storage zone but sedimentations occurs in the Intake area. Tactical dredging is the term given to localized dredging. It is used to keep outlets clear of sediments and can be an effective means of prolonging the useful life of a reservoir which is filling up with sediments (Palmeri *et al.*, 2003).

Flushing involves opening a low level outlet to temporarily establish riverine flow through which eroded sediment is flushed. The timing aspect of sediment release also makes flushing not very popular among environmentalists. Dredging involves mechanically digging up the coarse deposit and removing them from the reservoir. A detailed description of these methods can be found in Morris and Fan (1998) and Palmeri *et al.* (2003).

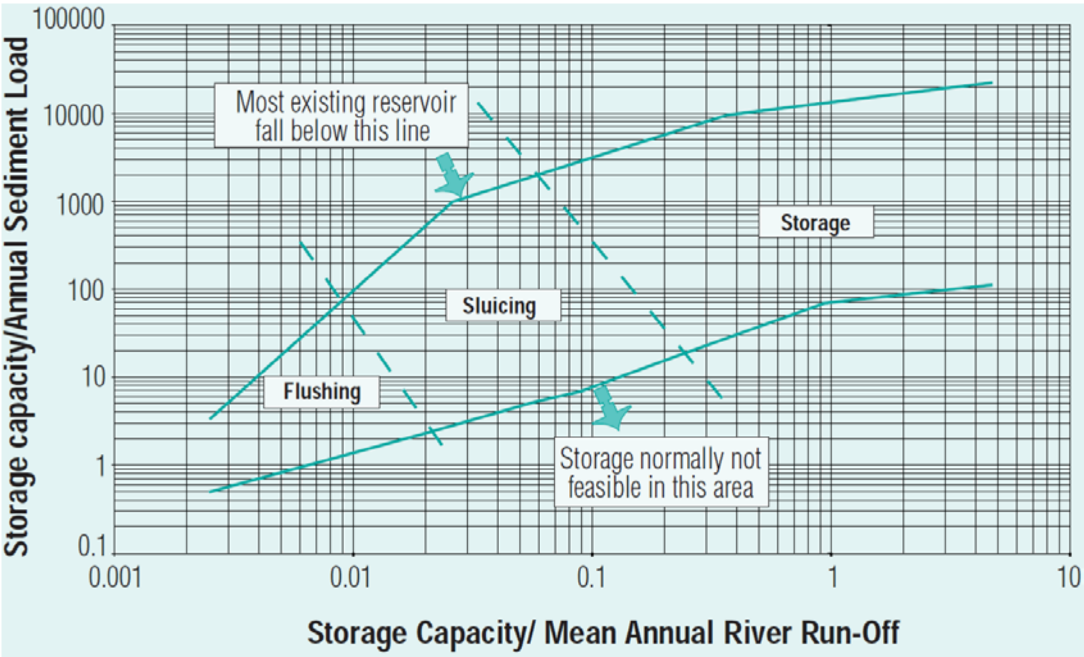


Figure A-7: Reservoir operation diagram (Palmeri *et al.*, 2003)

Reservoir operation diagram is shown in Figure A-7. This is the basis to adopt the evacuation of sediments technique from the reservoir. The diagram's entry data are: a) hydrological size of the reservoir which provides an indicator of excess water available for sediment management; b) sediment inflow relative to reservoir capacity, which provides an indicator of reservoir life span. The diagram is based on observations from many dams in the world and shows which operation model is likely to be effective (Palmeri *et al.*, 2003).

If sedimentation can be managed successfully, as it has been in some reservoirs, the loss in reservoir storage space due to this phenomenon can be lowered significantly. The benefit of effective reservoir sedimentation management is therefore clear. Due to the large investment in hydroelectric projects, it becomes imperative to consider measures to limit accumulation of sediments in reservoir.

APPENDIX B

ENERGY SITUATION IN NEPAL

B.1 EXISTING AND PLANNED HYDROPOWER PROJECTS

The power generating facilities in Nepal consists of hydro, diesel and solar power plants but it is basically a hydropower-oriented system. According to NEA (2011) total installed capacity in Nepal is 705 MW in 2010/2011, and 652 MW (92.5%) is generated by hydropower in the integrated system. Among them major hydropower plants of NEA and IPP generate 472 MW (67%) and 157 MW (22.3%).

The major existing and under connstructing hydropower projects of NEA and Independent Power Producers (IPPs) connected to Integrated Nepal Power System (INPS) are preseented in Table B-1.

Table B-1: Major Hydopower Stationed owned by NEA and IPP (NEA, 2011)

Existing				
S.No.	NEA		IPP	
	Hydro Power Station	Installed Capacity (MW)	Hydro Power Station	Installed Capacity (MW)
1	Middle Marsyangdi	70.0	Khimti Khola	60.0
2	Kaligandaki 'A'	144.0	Bhotekoshi Khola	36.0
3	Marsyangdi	69.0	Chilime	22.0
4	Kulekhani I	60.0	Indrawati - III	7.5
5	Kulekhani II	32.0	Jhimruk Khola	12.0
6	Trishuli	24.0	Andhi Khola	5.1
7	Devighat	14.0	Khudi Khola	3.5
8	Modi Khola	14.0	Piluwa Khola	3.0
9	Sunkoshi	10.1	Sunkoshi Khola	2.5
10	Puwa Khola	6.2	Ridi Khola	2.4
11	Gandak	15.0	Thoppal Khola	1.7
12	Small Hydropower Stations	13.8	Chaku Khola	1.5
	Total	**472.1**		**157.1**
Under Construction				
1	Upper Tamakoshi	456.0	Mai Khola	22.0
2	Chamelia	30.0	Lower Modi 1	9.9
3	Kulekhani III	14.0	Sipring Khola	9.6
4	Upper Trishuli 3"A"	60.0	Ankhu Khola 1	8.4
5	Rahughat	32.0	Siuri Khola	5.0
	Total	**592.0**		**54.9**

B.2 AVAILABLE ENERGY AND PEAK LOAD DEMAND

NEA has published the total energy available and the peak load demand in NEA's annual report 2010/11 from 2003 to 2011. Table B-2 and Figure B-1 show that the peak load demand before 2003 was only 470 MW which is less than the total capacity of 606 MW including IPP. Kaligandaki A power plant was commissioned in 2001 and brought the total generation capacity of 578 MW. After the Kaligandaki A hydropower plant there was not any major power plant implemented in Nepal until Middle Marsyangdi Hydropower Plant was commissioned in the beginning of 2009. Significant load shedding started from year 2005 during off-peak hours but there were always spilled energy during the wet season.

The scheduled outages, called "load shedding" in Nepal, are a regular part of the winter. Because of Nepal's monsoon climate and dependence on "run of the river" electricity generation, power production falls dramatically in the dry season.

Table B-2: Available energy and total peak load demand (NEA, 2011)

Particulars	2003	2004	2005	2006	2007	2008	2009	2010	2011
Peak Demand (MW)	470	515	558	603	648	722	813	885	946
Available Capacity (MW)	606	606	609	612	616	616	689	698	706
Available winter Capacity (MW)	451	452	456	456	456	456	490	495	500
Available Energy (GWh)	2,261	2,381	2,643	2,781	3,052	3,186	3,131	3,689	3,858
1. Hydro	1,478	1,345	1,523	1,569	1,747	1,793	1,840	2,105	2,122
2. Thermal	4	10	14	16	13	9	9	13	3
3. Purchase (Total)	779	1,026	1,106	1,196	1,291	1,384	1,282	1,572	1,733
India	150	187	241	266	329	425	356	613	694
Nepal	629	839	865	930	962	959	926	959	1,039

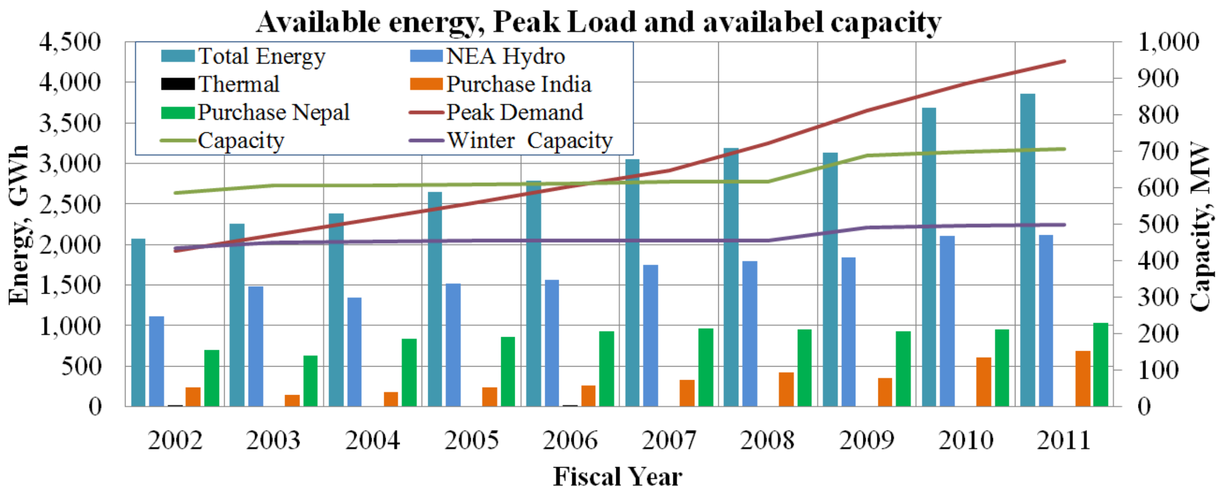


Figure B-1: Available energy and total peak load demand (NEA, 2011)

Figure B-2 shows the system load curve of the peak load demand on January 28, 2011. The figure shows that the peak load demand starts from 5 PM and ends at 10 PM. Load demand at day time is only about 60% of the peak demand.

Peak power demand of the Integrated Nepal Power System (INPS) over the year 2010/2011 reached 946.1 MW at 18:30 hours on January 28, 2011 registering 6.87% increase over peak demand of previous year 885.28 MW. Likewise, annual energy demand was estimated at 4,833 GWh, out of which only 3,850 GWh (79.67%) could be served from available sources and the rest 982 GWh (20.33 %) had to be curtailed as load shedding to keep the system in operational balance (NEA, 2011).

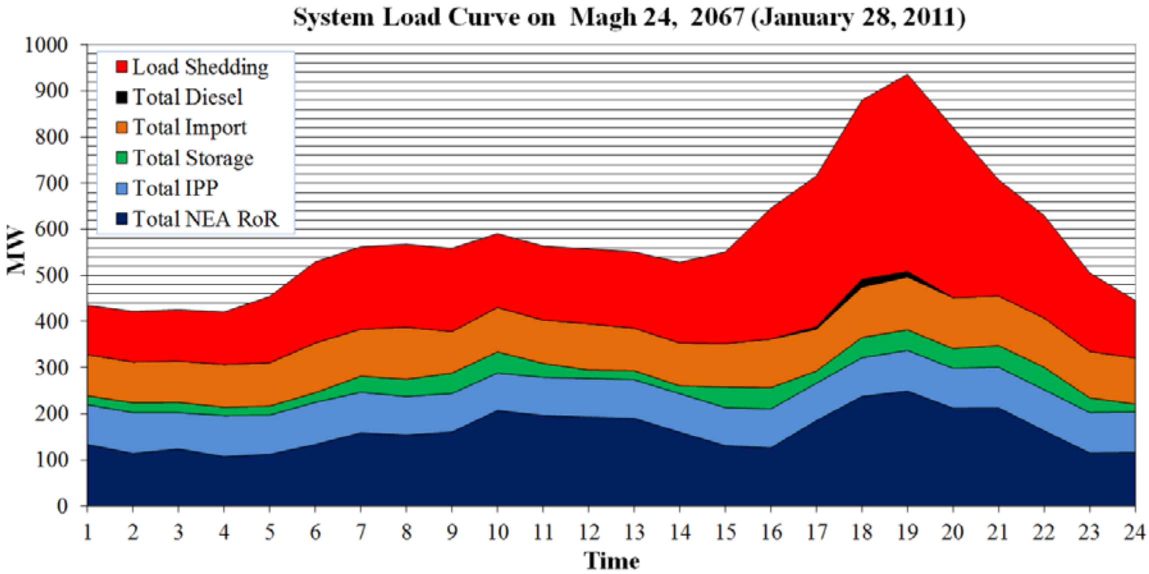


Figure B-2: System load curve on 28 January 2011 (modified from NEA, 2011)

B.3 POWER DEMAND FORECAST AND FUTURE PLAN

NEA made the power demand forecast for the 18 years period from 2011 to 2027/28 as shown in Figure B-3. According to the forecast, the peak load is estimated to be 1,272 MW in 2013/14. The average annual growth rate for peak demand is estimated to be about 10% for the next 5 years. The energy demand is forecasted to be 5,860 GWh in 2013/14. The average annual growth rate is estimated to be 10% for next 5 years

Nepalese are already facing an acute shortage of electricity the whole year, if proper initiative is not taken to develop more hydropower projects, it seems that the situation will be more severe and power cut-off will be increased for more hours per week during the dry period as well as in the wet season.

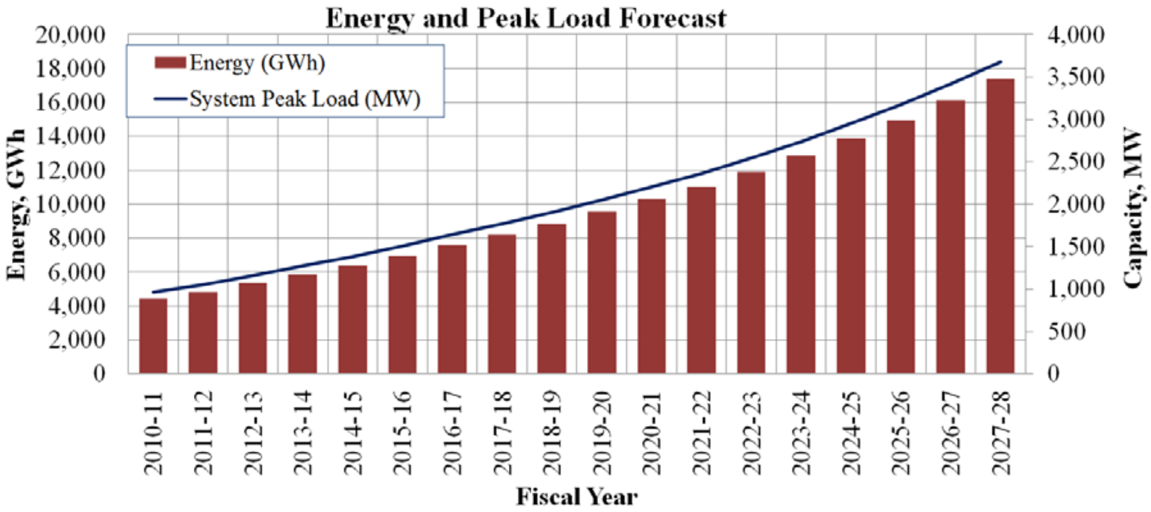


Figure B-3: Energy and peak load demand forecast charts (NEA, 2010)

In February 2008 the Government of Nepal formed a committee for solving the load shedding problem. The committee had prepared a report and recommended 25 points for solving the load shedding problem. Some of them are development of storage power project like Upper Seti, speed up the construction of Upper Tama Koshi, Chameliya, Upper Trishuli A, Upper Trishuli B, Kulekhani III, Rahughat and encourage IPPs for hydropower development. They have also studied the energy production from existing and forthcoming hydropower projects up to 2013/14 and developed energy and capacity balance per day of the month from 2009/10 to 2013/14 (Provided that all mentioned projects will be in operation by 2013/14). Figure B-4 presents the energy and capacity balance per day of the month for year 2013/14. The figure shows that capacity will be surpluses in wet season and deficit during the dry season. But there will be surplus energy in every month.

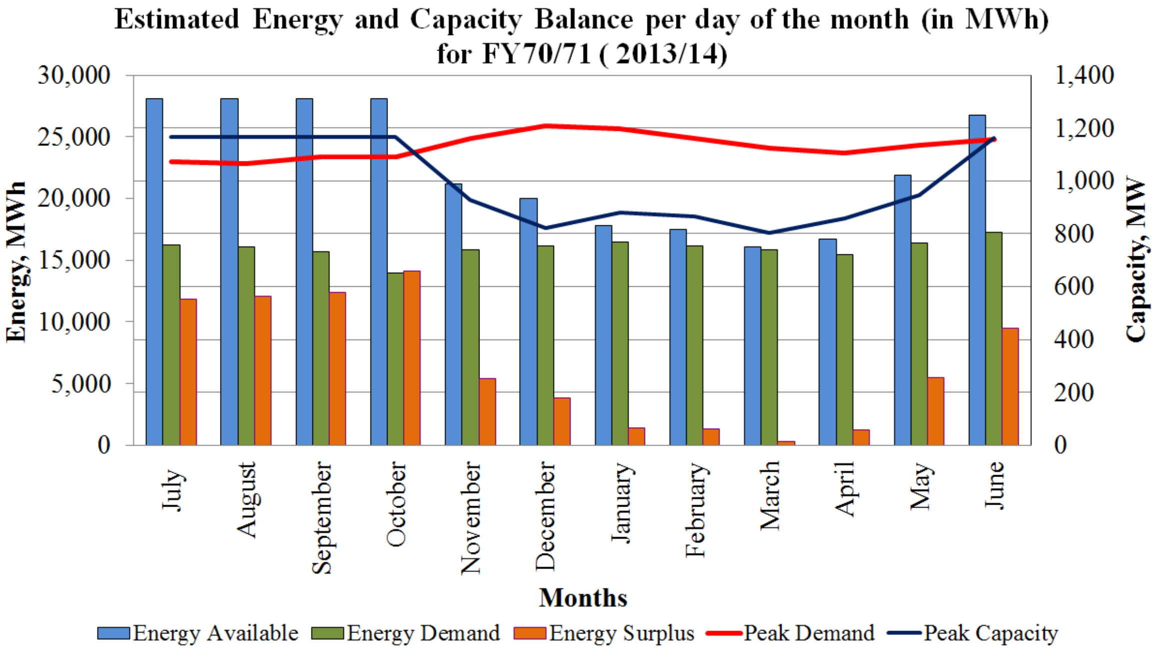


Figure B-4: Estimated capacity and energy balance in 2013/14 (modified from GoN, 2008)

APPENDIX C

ENERGY GENERATION FROM MAJOR HYDROPOWER PLANTS IN 2006/2007

Daily Generation (MWh) from Major Hydropower Plants
July, 2006

Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
7/1	1497.2	319	116	2,329	990	384	123	1,352	837	475	564.5
7/2	1497.2	282	107	2,570	1,158	336	126	1,417	790	498	569.0
7/3	1497.1	287	112	2,297	1,287	353	101	1,420	878	499	571.5
7/4	1497.1	304	118	2,132	1,217	362	80	1,416	877	499	555.4
7/5	1497.0	306	114	2,235	1,394	440	94	1,405	807	499	568.8
7/6	1496.9	294	107	2,258	1,344	373	103	1,413	874	498	551.0
7/7	1496.9	255	91	1,813	948	469	96	1,387	878	499	547.6
7/8	1496.9	194	78	1,736	762	389	93	1,392	859	491	545.1
7/9	1496.9	343	137	1,995	1,076	433	94	1,405	878	336	561.2
7/10	1496.9	420	164	1,998	1,332	375	78	1,409	878	275	542.6
7/11	1496.9	417	163	1,745	1,235	412	103	1,412	877	497	549.7
7/12	1497.2	381	158	1,832	1,294	411	73	1,405	873	499	546.8
7/13	1498.0	163	72	1,811	1,105	410	113	1,414	879	499	566.7
7/14	1498.3	314	127	2,462	642	439	95	1,409	874	500	565.7
7/15	1498.3	368	139	2,314	851	313	96	1,404	871	463	565.5
7/16	1498.4	369	185	1,952	1,345	384	76	1,404	878	495	564.3
7/17	1498.5	281	98	2,317	1,367	361	134	1,369	884	502	541.1
7/18	1498.6	260	102	2,487	978	348	135	1,380	875	497	570.0
7/19	1498.9	279	105	2,105	1,348	385	136	1,364	882	501	564.9
7/20	1499.0	285	102	2,256	1,362	409	112	1,414	878	501	565.8
7/21	1499.2	253	97	2,406	1,322	403	133	1,328	879	501	574.9
7/22	1499.3	284	107	2,527	781	352	189	1,413	882	500	583.9
7/23	1499.3	307	119	2,327	1,302	408	196	1,417	881	504	546.3
7/24	1499.4	219	81	2,127	1,439	419	161	1,413	880	502	582.6
7/25	1499.4	302	120	2,445	962	368	148	1,414	871	496	577.5
7/26	1499.6	295	114	2,146	1,043	376	103	1,404	872	494	572.8
7/27	1499.6	275	101	2,348	1,251	370	118	1,414	881	497	582.9
7/28	1500.0	285	102	2,436	876	374	157	1,411	879	498	577.6
7/29	1500.3	202	82	1,571	1,196	342	151	1,414	883	500	554.1
7/30	1500.4	314	126	1,780	1,087	366	138	1,305	835	483	568.8
7/31	1500.5	283	103	2,404	1,233	304	139	1,412	884	499	572.4

Daily Generation (MWh) from Major Hydropower Plants **August, 2006**											
Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
8/1	1500.6	302	120	2,487	952	373	142	1,412	881	498	545.0
8/2	1500.7	343	145	1,924	1,270	336	148	1,409	878	501	577.9
8/3	1501.0	360	140	2,634	1,166	299	147	1,420	879	500	579.5
8/4	1501.2	289	108	2,357	1,434	17	149	1,408	880	500	581.6
8/5	1501.5	160	69	2,205	1,157	340	148	1,389	878	501	558.8
8/6	1501.6	319	125	2,089	1,356	374	148	1,408	870	500	560.1
8/7	1501.6	308	119	2,549	1,320	175	148	1,320	870	408	566.8
8/8	1501.6	374	144	2,403	1,178	294	140	1,359	851	430	595.4
8/9	1501.6	288	109	2,537	1,082	335	144	1,412	880	501	577.5
8/10	1501.7	217	85	2,316	975	316	126	1,409	876	502	567.9
8/11	1501.7	319	125	2,456	1,216	365	134	1,409	876	501	584.7
8/12	1501.6	282	103	2,180	1,349	358	181	1,395	868	484	559.7
8/13	1501.7	271	94	2,510	1,353	235	166	1,373	862	464	568.3
8/14	1501.7	319	122	2,666	1,499	258	173	1,406	883	476	579.4
8/15	1501.6	358	142	2,408	1,518	319	146	1,367	880	491	590.8
8/16	1501.5	326	130	2,653	1,268	354	122	1,383	883	413	585.3
8/17	1501.4	418	173	2,620	1,523	363	146	785	529	452	547.5
8/18	1501.2	615	172	2,808	1,538	331	169	397	705	483	585.0
8/19	1501.5	136	54	2,228	1,260	368	168	1,417	888	496	560.2
8/20	1501.9	286	106	2,260	1,098	368	173	1,414	881	496	574.2
8/21	1502.0	297	115	2,465	1,145	377	151	1,380	857	485	579.1
8/22	1502.1	268	95	2,450	1,345	347	179	1,388	880	496	603.2
8/23	1502.2	424	165	2,970	1,497	348	193	277	882	496	556.4
8/24	1503.0	440	168	2,850	1,509	340	189	414	746	488	598.7
8/25	1503.8	201	81	2,063	1,190	360	189	1,417	875	491	555.1
8/26	1504.4	147	60	1,999	927	367	136	1,416	864	481	532.0
8/27	1505.0	213	71	1,972	1,154	327	151	1,417	879	495	560.4
8/28	1506.1	264	96	1,994	1,208	328	190	1,414	874	494	559.3
8/29	1506.9	258	84	1,849	1,264	370	190	1,414	874	491	567.8
8/30	1507.5	303	112	2,173	1,163	346	137	1,411	877	492	588.0
8/31	1507.9	295	104	2,136	1,309	294	172	1,415	879	490	592.8

Daily Generation (MWh) from Major Hydropower Plants **September, 2006**											
Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
9/1	1508.1	206	104	2,029	1,065	366	150	1,411	878	331	567.0
9/2	1508.4	182	58	1,899	1,439	388	180	1,297	860	0	554.6
9/3	1508.5	300	88	1,851	1,318	372	100	1,414	880	29	541.0
9/4	1508.6	305	117	1,898	1,474	365	179	1,415	881	0	552.3
9/5	1508.7	223	83	1,940	1,480	364	201	1,415	882	318	594.6
9/6	1508.9	156	49	1,859	1,480	380	208	1,416	875	497	582.0
9/7	1508.9	262	80	1,865	1,543	403	208	1,418	872	497	587.2
9/8	1509.2	213	67	2,033	1,348	363	207	1,410	876	516	567.5
9/9	1512.8	45	14	1,661	1,148	365	207	1,406	875	521	527.1
9/10	1514.3	117	43	1,739	1,218	345	113	1,414	878	526	581.7
9/11	1515.4	169	64	1,718	1,271	353	207	1,414	879	505	554.6
9/12	1518.1	160	65	1,866	1,219	389	223	1,407	876	524	583.2
9/13	1519.1	138	53	1,690	1,253	366	208	1,409	879	526	565.7
9/14	1519.7	260	94	1,835	1,371	374	208	1,419	880	524	583.8
9/15	1520.1	252	77	1,746	1,380	376	200	1,384	864	508	546.1
9/16	1520.4	158	53	1,732	1,282	394	208	1,384	882	510	572.2
9/17	1520.8	125	50	1,356	998	347	237	1,408	878	525	538.2
9/18	1521.0	156	53	1,594	1,312	376	271	1,333	875	525	571.3
9/19	1521.2	254	79	1,954	1,446	387	284	1,377	856	520	579.8
9/20	1521.4	262	90	1,821	1,473	391	283	1,414	883	527	598.5
9/21	1521.5	218	68	1,772	1,392	387	230	1,307	881	526	609.1
9/22	1521.7	147	50	1,648	1,250	401	288	1,413	880	527	549.9
9/23	1522.0	130	48	1,494	779	397	287	1,405	879	526	565.2
9/24	1522.3	148	49	1,383	1,140	347	282	1,399	876	520	598.5
9/25	1523.0	161	56	1,513	1,026	313	285	1,321	875	432	586.6
9/26	1523.4	172	57	1,770	1,235	357	242	1,403	873	472	607.9
9/27	1523.6	235	64	1,973	1,399	363	289	1,409	874	489	588.6
9/28	1523.8	255	92	1,747	1,408	372	288	1,384	869	485	588.2
9/29	1524.0	203	78	1,719	1,292	379	276	1,415	862	491	579.3
9/30	1524.2	206	87	1,597	845	296	284	1,414	878	468	503.9

Daily Generation (MWh) from Major Hydropower Plants
October, 2006

Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
10/1	1524.3	195	83	1,324	817	369	284	1,412	879	479	520.0
10/2	1524.5	115	69	1,248	722	401	284	1,415	878	507	495.2
10/3	1524.6	135	51	1,207	781	270	269	1,195	850	425	521.5
10/4	1524.8	142	56	1,342	988	290	281	1,410	880	496	539.3
10/5	1524.9	125	50	1,562	1,026	314	284	1,415	882	503	552.9
10/6	1525.0	161	66	1,678	1,180	380	285	1,411	882	458	515.6
10/7	1525.1	173	69	1,505	1,065	402	285	1,408	879	511	514.2
10/8	1525.2	242	88	1,691	1,156	401	286	1,405	880	497	585.9
10/9	1525.2	294	106	1,618	1,375	386	286	1,381	879	491	607.1
10/10	1525.3	258	98	1,692	1,382	406	285	1,339	876	491	586.0
10/11	1525.3	220	79	1,700	1,410	403	285	1,347	878	492	605.5
10/12	1525.3	278	115	1,684	1,408	416	285	1,309	879	490	598.8
10/13	1525.4	327	107	1,547	1,369	422	282	1,342	871	467	601.9
10/14	1525.4	206	77	1,650	1,311	386	285	1,339	852	496	576.0
10/15	1525.4	313	106	1,733	1,254	379	285	1,328	870	496	609.9
10/16	1525.4	302	102	1,752	1,336	409	280	1,357	876	486	582.9
10/17	1525.5	172	69	1,430	973	408	283	1,330	878	462	578.4
10/18	1525.5	260	96	1,728	1,140	340	277	1,404	865	498	584.9
10/19	1525.5	323	112	1,714	1,237	381	286	1,403	861	498	579.1
10/20	1525.5	341	116	1,769	1,237	410	217	1,248	812	475	587.0
10/21	1525.5	239	100	1,528	1,098	368	194	1,142	862	480	609.4
10/22	1525.6	215	90	1,654	872	360	184	1,122	845	477	595.6
10/23	1525.6	204	84	1,557	934	307	193	1,269	861	503	603.2
10/24	1525.7	183	77	1,253	839	341	177	1,411	842	503	547.4
10/25	1525.7	179	75	1,300	992	324	191	1,420	862	424	569.3
10/26	1525.7	185	76	1,653	1,140	345	202	1,422	863	500	573.2
10/27	1525.8	171	70	1,846	1,077	398	191	1,417	860	483	585.4
10/28	1525.8	97	34	1,697	758	384	192	1,415	855	486	560.7
10/29	1525.9	120	45	1,472	943	338	192	1,421	856	481	576.1
10/30	1525.9	131	51	1,722	1,000	324	190	1,421	854	476	589.0
10/31	1525.9	131	51	1,992	906	341	184	1,407	846	468	588.1

Daily Generation (MWh) from Major Hydropower Plants **November, 2006**											
Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
11/1	1526.0	198	81	1,817	1,224	368	137	1,410	835	467	590.2
11/2	1526.0	221	94	1,872	1,306	411	191	1,404	828	462	594.8
11/3	1526.0	226	94	2,013	1,266	376	159	1,387	828	455	607.8
11/4	1526.0	130	50	1,973	925	377	289	1,388	811	448	574.8
11/5	1526.0	188	73	2,023	1,130	361	283	1,395	789	442	606.1
11/6	1526.0	231	88	2,187	1,217	341	286	1,386	787	435	620.2
11/7	1526.0	196	78	2,100	1,163	383	285	1,389	766	420	610.5
11/8	1526.0	249	104	2,001	1,328	365	288	1,364	750	407	607.4
11/9	1526.0	309	115	2,069	1,263	417	286	1,362	730	430	620.9
11/10	1526.0	232	96	2,192	1,261	433	284	1,384	732	455	622.1
11/11	1526.0	204	80	2,017	1,010	399	287	1,369	731	456	581.9
11/12	1526.0	253	89	2,014	1,213	327	287	1,380	704	449	615.4
11/13	1526.0	308	108	2,097	1,277	356	283	1,382	686	445	617.6
11/14	1525.9	264	95	2,179	1,229	404	283	1,385	669	439	623.2
11/15	1525.9	280	110	2,249	1,342	350	285	1,370	662	405	632.2
11/16	1525.9	347	122	2,514	1,014	371	287	1,271	645	416	621.2
11/17	1525.5	309	103	2,388	1,051	382	288	1,347	640	424	617.5
11/18	1525.5	189	78	2,021	1,129	389	287	1,328	633	402	601.5
11/19	1525.4	390	113	2,011	1,161	377	286	1,316	839	403	621.1
11/20	1525.4	329	127	2,113	1,297	374	288	1,326	628	403	616.8
11/21	1525.3	405	157	2,401	1,049	417	287	1,291	612	403	629.8
11/22	1525.3	338	140	2,344	1,253	449	286	1,268	600	386	635.5
11/23	1525.2	389	141	2,502	1,277	431	276	1,242	578	358	620.6
11/24	1525.2	345	150	2,262	1,299	449	278	1,211	565	358	640.8
11/25	1525.1	261	99	2,361	1,068	444	284	1,197	563	338	603.1
11/26	1525.1	293	116	2,525	1,189	425	282	1,173	562	337	626.5
11/27	1525.0	417	162	2,394	1,267	411	285	1,169	560	346	631.3
11/28	1524.9	469	193	1,932	1,123	426	280	1,161	555	350	641.2
11/29	1524.8	444	172	2,317	1,272	430	276	1,148	557	348	639.1
11/30	1524.7	510	205	2,135	1,116	422	259	1,131	550	345	640.8

Daily Generation (MWh) from Major Hydropower Plants **December, 2006**											
Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
12/1	1524.6	509	199	2,142	1,149	403	263	1,099	547	343	638.8
12/2	1524.5	387	158	2,207	1,148	405	258	1,084	549	326	599.8
12/3	1524.4	503	204	2,037	1,126	404	249	1,081	543	340	616.5
12/4	1524.2	507	199	2,115	1,075	422	257	1,072	542	346	629.5
12/5	1524.1	450	170	1,911	1,109	309	255	1,062	533	351	639.6
12/6	1524.0	518	201	1,982	1,082	395	248	1,050	536	331	643.2
12/7	1523.9	474	192	2,030	1,139	384	243	1,038	530	336	645.7
12/8	1523.8	545	209	2,039	1,090	398	243	1,029	533	328	643.9
12/9	1523.6	519	209	2,003	1,168	334	235	1,009	530	321	620.9
12/10	1523.5	478	191	1,760	1,004	287	235	1,001	534	319	642.8
12/11	1523.5	416	153	1,922	1,014	392	274	1,052	523	334	631.3
12/12	1523.4	467	179	2,143	1,280	423	289	1,032	524	337	640.4
12/13	1523.3	474	193	1,907	893	403	284	996	532	326	625.7
12/14	1523.2	565	237	1,879	1,182	393	285	984	534	341	642.8
12/15	1523.1	489	191	1,843	975	403	279	973	533	320	641.1
12/16	1523.0	329	126	1,861	1,049	385	274	962	532	342	636.1
12/17	1522.9	448	177	1,986	1,038	380	269	946	527	313	645.6
12/18	1522.8	515	220	2,097	1,072	389	263	938	531	340	638.5
12/19	1522.7	561	249	1,903	1,024	356	261	932	524	320	643.6
12/20	1522.5	479	201	1,729	979	380	255	921	522	313	645.2
12/21	1522.4	641	265	1,953	1,001	352	248	915	524	338	648.4
12/22	1522.2	533	222	1,798	1,057	348	246	908	510	311	639.0
12/23	1522.1	500	212	1,667	958	372	243	894	506	318	618.8
12/24	1521.9	658	291	2,119	963	393	238	879	494	304	644.3
12/25	1521.7	636	266	1,705	789	350	235	865	488	304	641.9
12/26	1521.5	748	326	1,581	928	376	233	856	491	321	635.9
12/27	1521.3	612	263	1,684	1,007	373	230	848	481	314	637.6
12/28	1521.1	689	291	1,744	1,007	403	229	848	482	301	644.1
12/29	1520.8	794	336	1,694	939	381	230	842	474	286	644.6
12/30	1520.7	521	212	1,771	949	377	225	837	480	302	645.1
12/31	1520.5	722	315	1,560	979	393	220	827	475	329	648.0

Daily Generation (MWh) from Major Hydropower Plants **January, 2007**											
Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
1/1	1520.2	823	358	1,675	819	389	212	813	464	290	645.1
1/2	1520.0	656	278	1,559	937	369	212	805	460	301	646.2
1/3	1519.7	802	351	1,630	936	373	209	794	463	316	642.0
1/4	1519.5	665	292	1,732	798	378	207	747	463	317	641.3
1/5	1519.2	829	376	1,598	905	371	205	775	452	315	644.9
1/6	1519.0	781	325	1,540	864	363	199	768	443	304	644.8
1/7	1518.7	681	291	1,575	841	362	199	764	422	285	631.7
1/8	1518.4	885	393	1,521	819	347	199	760	431	290	642.5
1/9	1518.2	784	345	1,594	922	353	197	764	446	152	632.3
1/10	1517.8	950	417	1,364	914	346	194	764	448	0	635.6
1/11	1517.4	1,029	458	1,435	881	337	197	744	427	48	645.4
1/12	1517.2	812	356	1,553	857	339	197	741	418	237	620.1
1/13	1516.9	1,029	458	1,435	881	337	195	744	427	228	645.4
1/14	1516.6	781	334	1,391	880	314	194	731	413	241	637.6
1/15	1516.3	787	341	1,315	826	329	195	731	417	256	641.2
1/16	1516.1	720	270	1,244	729	336	194	718	418	237	542.1
1/17	1515.8	771	369	1,495	868	322	152	707	413	235	638.0
1/18	1515.4	1,127	503	1,391	819	307	194	707	415	235	642.9
1/19	1514.9	1,075	496	1,443	876	310	191	706	419	235	636.8
1/20	1514.5	1,096	475	1,311	682	312	192	699	413	235	640.1
1/21	1514.0	1,173	541	1,266	832	342	185	689	409	235	643.5
1/22	1513.6	1,126	493	1,306	816	305	183	688	412	235	627.8
1/23	1513.1	1,050	468	1,391	791	315	186	688	410	236	633.3
1/24	1512.6	1,130	506	1,322	748	313	186	678	405	235	627.8
1/25	1512.2	1,076	482	1,282	820	311	183	676	408	235	623.3
1/26	1511.8	937	397	1,390	755	316	180	672	407	237	637.1
1/27	1511.5	698	304	1,263	758	391	174	658	405	238	635.3
1/28	1511.2	691	307	1,351	758	304	170	651	403	238	600.5
1/29	1511.0	601	253	1,366	698	245	172	642	397	236	574.4
1/30	1510.9	431	168	1,186	796	272	166	639	402	235	580.9
1/31	1510.7	396	148	1,496	739	297	165	671	397	235	580.0

Daily Generation (MWh) from Major Hydropower Plants **February, 2007**											
Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
2/1	1510.5	552	249	1,359	719	292	161	680	392	219	591.9
2/2	1510.3	595	286	1,312	782	292	163	664	390	214	568.6
2/3	1510.1	558	219	1,253	656	279	163	661	386	218	572.2
2/4	1509.9	389	148	1,327	770	282	164	677	414	229	579.2
2/5	1509.9	273	113	1,376	775	303	170	906	413	265	581.0
2/6	1509.7	447	227	1,300	821	303	172	723	407	220	603.4
2/7	1509.6	115	43	1,087	1,052	296	190	886	420	246	566.8
2/8	1509.6	281	104	1,396	793	313	181	769	395	247	578.3
2/9	1509.5	376	157	1,513	785	288	178	719	388	234	602.1
2/10	1509.3	668	246	1,202	854	297	172	708	393	227	622.4
2/11	1508.9	662	342	1,252	706	280	214	765	412	224	587.1
2/12	1508.7	682	310	1,139	792	292	224	820	409	220	609.4
2/13	1508.4	735	335	1,373	778	276	206	705	386	227	606.6
2/14	1508.6	340	73	1,289	901	299	209	825	355	177	598.2
2/15	1508.5	483	185	2,156	1,208	167	194	871	407	240	607.5
2/16	1508.4	458	182	1,694	833	293	185	779	406	241	568.6
2/17	1508.3	409	187	1,489	1,000	291	197	787	409	249	588.2
2/18	1508.2	400	157	1,597	734	314	215	711	391	226	580.5
2/19	1508.1	453	179	1,477	863	272	215	702	391	212	580.8
2/20	1508.0	333	150	1,532	759	281	204	692	393	218	574.9
2/21	1507.8	487	221	1,302	808	282	203	704	402	230	599.7
2/22	1507.6	492	211	1,568	878	291	198	701	408	234	599.0
2/23	1507.5	421	179	1,510	821	290	192	696	409	241	535.6
2/24	1507.4	386	164	1,419	775	282	189	695	402	224	572.7
2/25	1507.2	456	213	1,763	838	393	189	698	406	221	575.8
2/26	1507.1	319	141	1,494	756	280	202	763	413	228	599.8
2/27	1507.0	323	135	1,497	826	323	192	703	399	180	568.9
2/28	1507.0	137	57	1,278	796	314	194	677	408	221	529.6

Daily Generation (MWh) from Major Hydropower Plants

March, 2007

Date	Kulekhani Reservoir Level (masl)	NEA					IPP				PEAK (MW)
		Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
3/1	1507.0	262	110	1,713	879	321	234	714	420	238	554.7
3/2	1507.0	179	76	1,676	977	335	245	729	401	214	580.6
3/3	1507.0	254	107	1,582	648	305	204	692	402	235	547.7
3/4	1507.0	158	72	1,340	884	325	198	684	421	249	516.7
3/5	1507.0	90	38	1,448	898	321	193	677	430	244	580.4
3/6	1507.0	100	42	1,526	609	315	188	682	435	241	570.0
3/7	1507.0	99	45	1,572	953	319	184	668	425	240	583.4
3/8	1507.0	138	62	1,338	945	318	179	654	427	244	581.5
3/9	1506.8	489	214	1,274	731	310	178	642	422	242	555.9
3/10	1506.7	348	144	1,448	395	306	158	639	416	234	538.2
3/11	1506.6	397	162	1,463	0	301	172	627	410	223	593.5
3/12	1506.4	492	208	1,450	0	308	177	628	411	236	575.6
3/13	1506.3	367	155	1,638	0	323	274	651	407	235	587.1
3/14	1506.2	391	100	1,872	0	330	287	655	399	222	582.6
3/15	1506.1	379	157	1,786	0	332	259	651	383	228	597.8
3/16	1505.8	668	291	1,623	0	317	213	630	388	222	602.7
3/17	1505.5	781	336	1,481	0	284	196	606	395	226	585.6
3/18	1505.3	515	213	1,522	165	300	197	610	434	250	597.4
3/19	1505.1	372	155	1,335	1,009	320	195	638	469	275	567.2
3/20	1505.1	256	107	1,795	824	357	188	655	485	307	606.2
3/21	1505.0	308	127	1,750	833	350	183	765	503	293	623.5
3/22	1504.8	622	117	1,420	1,011	365	164	809	478	279	622.0
3/23	1504.4	591	155	2,008	930	340	165	680	478	318	611.6
3/24	1504.2	419	159	1,663	872	361	161	711	506	351	591.7
3/25	1504.1	330	124	1,850	840	404	159	739	511	356	612.1
3/26	1504.0	264	104	1,974	988	390	154	721	514	343	596.6
3/27	1503.9	358	157	1,857	956	375	153	691	503	336	601.7
3/28	1503.7	390	166	1,842	896	365	154	706	519	378	614.6
3/29	1503.6	570	309	2,096	1,057	393	156	732	520	427	616.1
3/30	1503.4	505	143	2,314	1,133	413	151	723	505	436	604.6
3/31	1503.2	408	147	1,926	1,034	429	147	702	499	445	589.3

Daily Generation (MWh) from Major Hydropower Plants **April, 2007**											
Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
4/1	1503.0	352	140	2,447	1,191	427	138	718	519	448	609.9
4/2	1502.9	336	126	2,600	1,415	443	130	754	517	482	589.1
4/3	1502.6	470	192	2,382	1,220	396	130	705	515	463	600.4
4/4	1502.3	629	270	2,414	1,087	426	126	689	516	466	596.6
4/5	1502.0	521	161	2,369	1,276	426	124	687	518	488	597.5
4/6	1501.7	502	213	2,468	1,064	423	123	654	518	487	601.9
4/7	1501.5	510	146	2,138	1,060	452	121	711	502	494	544.2
4/8	1501.3	630	185	2,347	1,167	417	118	801	517	493	597.0
4/9	1500.9	516	315	2,261	1,233	393	110	678	520	499	594.4
4/10	1500.5	878	273	2,244	1,090	370	107	643	516	466	599.1
4/11	1500.1	564	301	1,838	982	354	101	807	503	429	574.6
4/12	1499.7	740	260	2,341	1,226	315	119	728	501	433	594.9
4/13	1499.0	1,029	230	2,178	968	340	111	653	509	440	598.0
4/14	1498.7	763	137	1,868	811	355	116	639	517	467	566.9
4/15	1498.5	419	172	2,419	1,177	352	115	607	526	480	579.0
4/16	1498.1	625	269	2,240	1,092	386	117	712	529	492	592.0
4/17	1497.8	547	220	2,513	1,118	397	127	725	528	498	581.3
4/18	1497.5	552	224	2,286	1,242	380	247	670	511	478	585.2
4/19	1497.2	575	237	2,280	1,114	352	166	648	540	467	605.1
4/20	1496.9	497	171	2,718	1,192	362	239	649	540	442	592.1
4/21	1496.8	274	106	1,713	1,025	349	188	673	540	475	513.7
4/22	1496.8	182	55	1,507	896	362	134	948	516	486	557.0
4/23	1496.7	386	153	1,487	1,168	390	122	784	517	424	591.4
4/24	1496.5	378	148	1,309	972	360	102	759	517	424	514.6
4/25	1496.3	404	170	1,508	1,075	360	94	863	517	418	577.8
4/26	1496.1	460	188	2,041	1,314	369	150	797	520	422	577.4
4/27	1495.5	766	320	2,010	995	352	106	707	527	397	608.5
4/28	1495.2	536	201	2,060	1,138	321	97	724	536	391	562.1
4/29	1495.0	332	126	1,792	907	333	93	711	570	428	563.5
4/30	1494.6	709	271	2,143	1,001	264	72	665	465	403	596.7

Daily Generation (MWh) from Major Hydropower Plants **May, 2007**											
Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
5/1	1494.5	225	74	2,286	1,231	276	87	697	566	444	563.3
5/2	1494.3	486	197	1,911	1,009	343	83	801	600	477	584.2
5/3	1493.9	609	234	2,219	1,127	377	78	781	581	477	584.9
5/4	1493.4	706	276	2,355	1,231	353	74	754	622	476	583.3
5/5	1493.1	518	229	2,247	1,056	369	71	864	636	502	546.0
5/6	1492.9	397	166	2,385	1,280	208	79	895	643	511	539.8
5/7	1492.8	311	123	2,314	1,434	323	93	1,167	659	509	573.2
5/8	1492.6	459	184	2,751	1,373	328	80	896	639	500	582.1
5/9	1492.2	636	274	1,950	1,246	376	72	913	651	143	587.3
5/10	1492.0	438	165	2,599	1,364	387	94	909	641	234	587.6
5/11	1491.8	312	137	1,550	860	395	85	967	636	442	580.0
5/12	1491.9	169	73	1,496	920	344	91	1,061	644	447	554.4
5/13	1491.7	429	130	2,686	1,187	399	94	1,000	597	422	605.5
5/14	1491.3	618	44	2,176	1,302	386	78	892	580	417	612.0
5/15	1490.9	771	248	1,973	1,120	385	62	839	528	472	615.2
5/16	1490.5	473	194	2,431	1,250	376	76	912	697	495	586.0
5/17	1490.2	509	210	2,527	1,519	393	95	821	722	481	610.0
5/18	1490.3	346	141	2,274	1,424	366	97	1,058	798	480	582.2
5/19	1489.9	186	50	2,172	1,316	392	100	817	776	436	570.1
5/20	1489.8	363	142	2,146	1,471	349	44	1,403	810	400	584.4
5/21	1489.4	519	189	2,047	1,549	398	0	1,405	812	436	606.0
5/22	1489.0	523	210	1,902	1,566	344	0	1,360	794	436	600.0
5/23	1488.6	580	246	2,157	1,626	311	0	1,189	809	496	535.2
5/24	1488.1	580	247	2,138	1,600	334	0	1,068	812	492	581.2
5/25	1487.6	497	202	2,229	1,428	378	0	959	813	494	595.9
5/26	1487.2	522	205	2,702	0	413	0	894	815	474	541.6
5/27	1486.8	412	174	2,066	435	403	0	845	813	490	551.8
5/28	1486.6	301	119	2,056	1,387	395	0	817	809	501	593.4
5/29	1486.4	197	61	2,445	1,418	380	0	814	804	497	592.4
5/30	1486.5	89	29	1,935	1,331	420	0	809	753	505	571.9
5/31	1486.4	165	69	2,411	1,369	425	0	582	708	507	597.0

Daily Generation (MWh) from Major Hydropower Plants **June, 2007**											
Date	NEA						IPP				PEAK (MW)
	Kulekhani Reservoir Level (masl)	Kulekhani 1	Kulekhani 2	Kaligandaki A	Marsyangdi	Trishuli	Jhimruk	Khimti	Bhotekoshi	Chilime	
6/1	1486.2	260	96	1,824	1,172	432	0	751	751	490	578.2
6/2	1486.1	122	40	2,108	1,325	439	0	737	770	511	567.7
6/3	1486.0	208	75	1,886	1,165	424	0	775	740	475	580.0
6/4	1485.9	131	38	2,164	1,243	379	0	964	804	502	589.5
6/5	1486.0	82	24	2,327	1,511	422	0	1,018	811	505	563.5
6/6	1486.8	90	38	2,248	1,310	420	0	1,346	808	505	595.5
6/7	1487.3	122	53	2,323	1,568	422	0	1,167	812	506	556.5
6/8	1487.4	189	63	2,467	1,568	422	0	1,407	807	505	561.4
6/9	1487.6	91	34	2,367	1,418	427	0	1,306	808	114	539.3
6/10	1487.8	109	41	2,090	1,438	423	0	1,404	812	504	582.6
6/11	1487.9	182	69	1,928	1,303	435	0	1,396	796	499	570.1
6/12	1487.9	190	67	2,250	1,424	445	36	1,363	775	488	574.3
6/13	1488.1	198	69	2,251	1,466	375	62	1,366	797	489	586.8
6/14	1488.2	175	55	2,144	1,457	424	80	1,399	798	500	596.4
6/15	1489.2	58	18	2,119	1,167	419	77	1,413	871	511	589.6
6/16	1489.6	105	44	2,193	1,095	415	96	1,286	844	498	562.7
6/17	1489.8	173	58	2,123	1,340	416	119	1,382	847	497	576.3
6/18	1489.8	265	105	2,371	1,389	347	104	1,412	869	495	603.3
6/19	1490.9	199	73	2,384	1,468	329	101	1,418	876	505	596.7
6/20	1491.3	177	61	2,388	1,427	400	97	1,415	870	506	604.4
6/21	1491.4	184	56	2,772	1,527	420	95	950	871	299	583.4
6/22	1491.6	107	42	2,515	1,400	440	94	988	861	235	577.8
6/23	1491.8	146	54	2,108	1,257	411	96	1,414	879	253	580.0
6/24	1491.9	112	40	2,343	1,496	406	97	1,419	879	261	581.3
6/25	1492.1	109	45	2,295	1,508	428	80	1,418	875	259	594.6
6/26	1492.2	177	63	2,364	1,532	451	105	1,420	871	461	597.6
6/27	1492.4	141	59	2,091	1,436	411	102	1,370	875	520	599.3
6/28	1492.5	143	51	2,247	1,554	451	101	1,417	873	519	611.5
6/29	1492.8	98	37	2,290	1,246	377	104	1,421	873	511	593.2
6/30	1493.0	130	45	2,030	1,334	411	103	1,399	872	502	580.4

APPENDIX D

VISUAL BASIC APPLICATION USED IN EXCEL MODEL

D.1 RESERVOIR LEVEL OPTIMIZATION OF PROR RESERVOIRS

```
Private Sub ResOpti_Click()

For i = 0 To 1
Z = 3166 + i * 24 'target cell
a = 3150  ' range variable
b = a + i * 24
c = b + 24

Set r1 = Range(Cells(b, 13), Cells(c, 13))

SolverReset
    SolverOk SetCell:=Cells(Z, 21), MaxMinVal:=1, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear" '#1 maximum
     SolverAdd CellRef:=Range(r1), Relation:=1, FormulaText:="1"
    SolverAdd CellRef:=Range(r1), Relation:=3, FormulaText:="0.5"
     SolverOk SetCell:=Cells(Z, 21), MaxMinVal:=1, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear"
    SolverAdd CellRef:=Cells(Z, 21), Relation:=3, FormulaText:="0"
    SolverOk SetCell:=Cells(Z, 21), MaxMinVal:=1, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear"
     SolverOk SetCell:=Cells(Z, 21), MaxMinVal:=1, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear"

    SolverSolve UserFinish:=True
    SolverFinish KeepFinal:=1

Next i
End Sub
```

D.2 MINIMISATION OF SPILLING FROM PROR RESERVOIRS

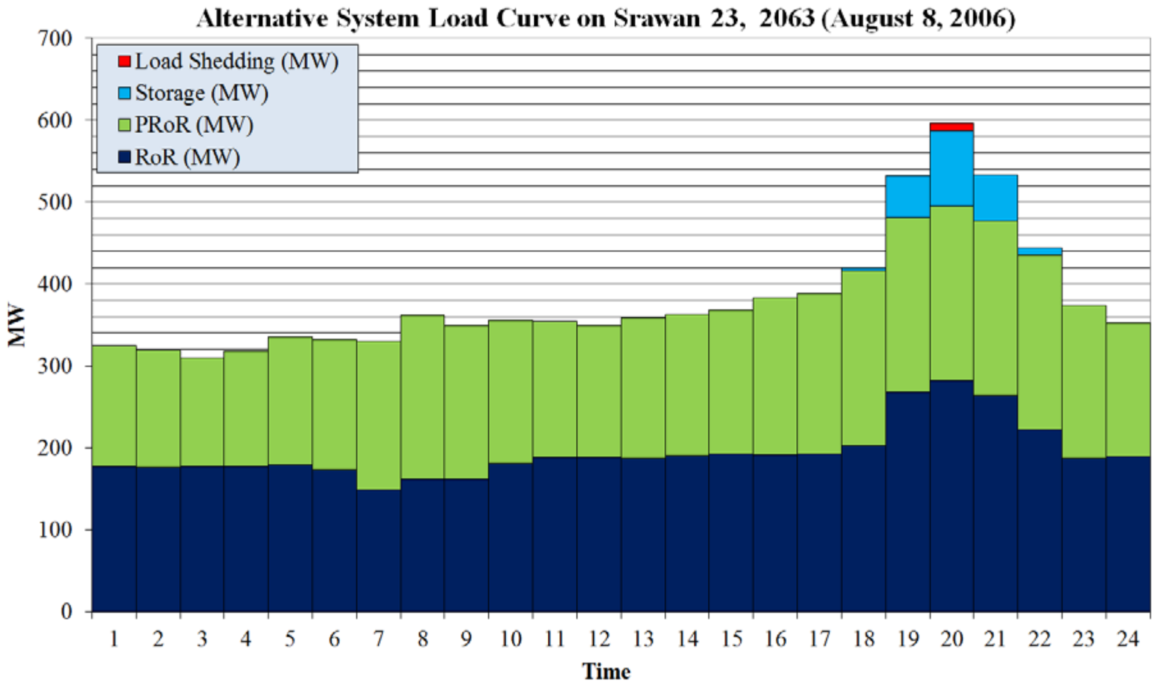
```
Private Sub Spill_Click()

For i = 0 To 5
Z = 3174 + i * 24 'target cell
a = 3150  ' range variable
b = a + i * 24
c = b + 24

Set r1 = Range(Cells(b, 13), Cells(c, 13))

SolverReset
    SolverOk SetCell:=Cells(Z, 24), MaxMinVal:=2, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear" '#2 minimum
    SolverAdd CellRef:=Range(r1), Relation:=1, FormulaText:="1"
    SolverAdd CellRef:=Range(r1), Relation:=3, FormulaText:="0.5"
     SolverOk SetCell:=Cells(Z, 24), MaxMinVal:=1, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear"
    SolverAdd CellRef:=Cells(Z, 24), Relation:=3, FormulaText:="0"
    SolverOk SetCell:=Cells(Z, 24), MaxMinVal:=2, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear"
     SolverOk SetCell:=Cells(Z, 24), MaxMinVal:=2, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear"

    SolverSolve UserFinish:=True
    SolverFinish KeepFinal:=1

Next i
End Sub
```

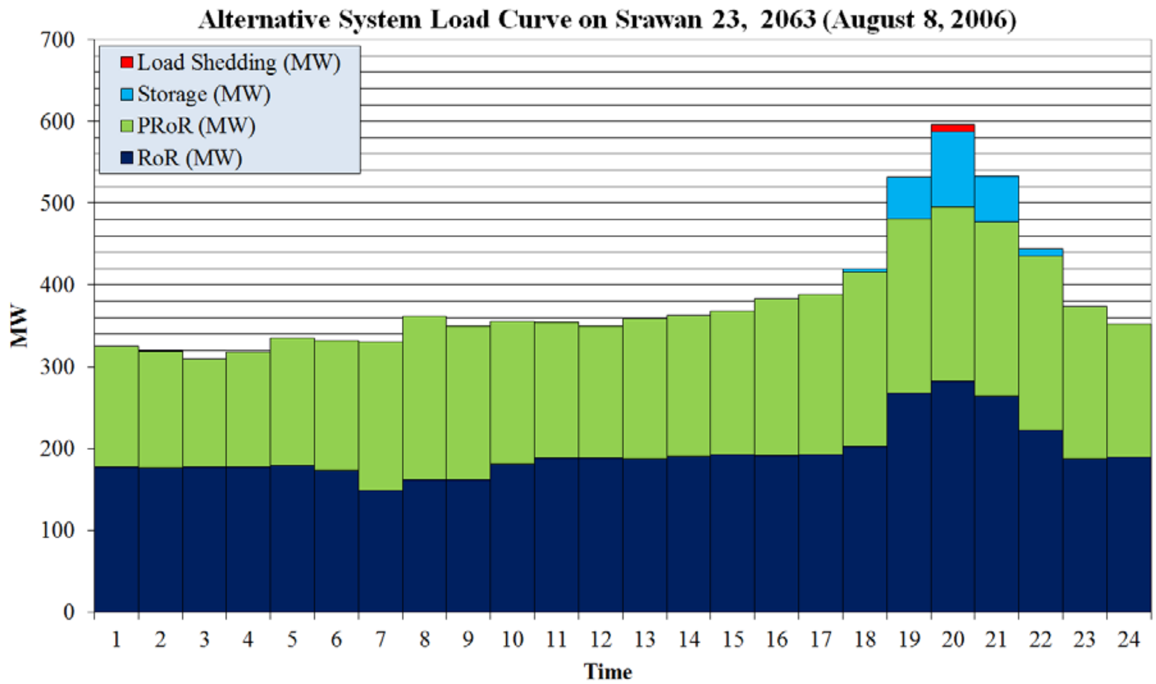
D.3 MINIMISATION OF LOAD SHEDDING

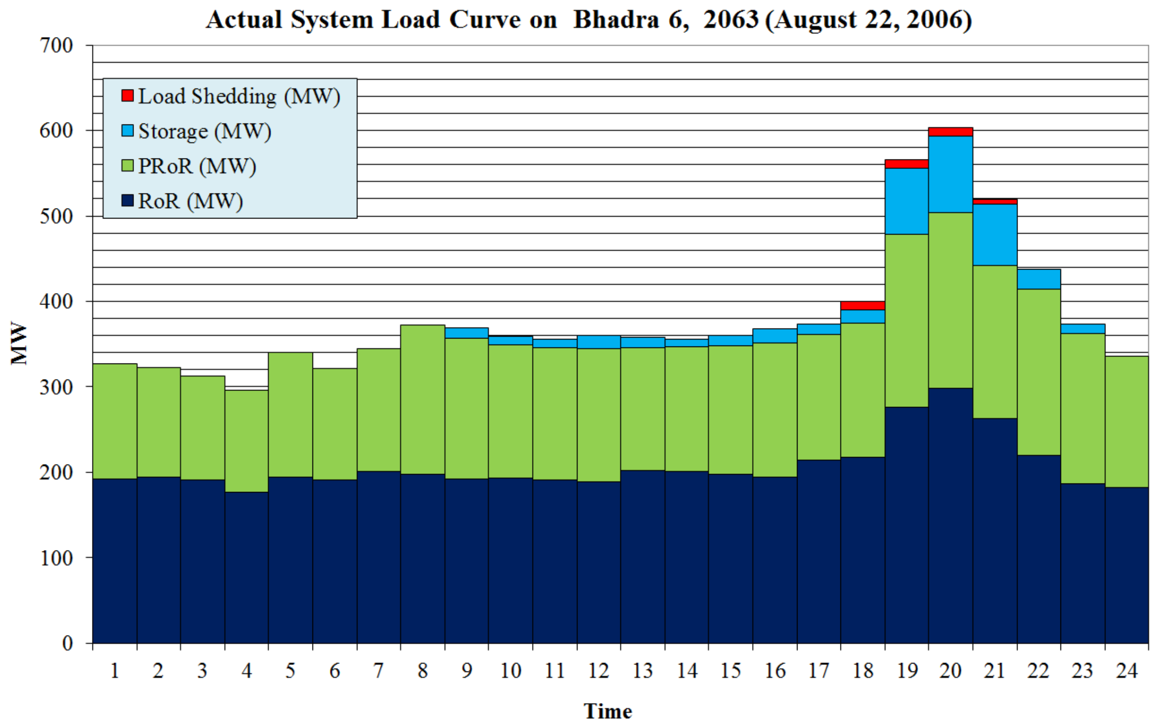
```
Private Sub LoadSheding_Click()

For i = 0 To 5
Z = 3174 + i * 24 'target cell
a = 3150 ' range variable
b = a + i * 24
c = b + 24

Set r1 = Range(Cells(b, 13), Cells(c - 1, 13))

SolverReset
    SolverOk SetCell:=Cells(Z, 25), MaxMinVal:=2, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear" '#2 minimum
    SolverAdd CellRef:=Range(r1), Relation:=1, FormulaText:="1"
    SolverAdd CellRef:=Range(r1), Relation:=3, FormulaText:="0.2"
    SolverOk SetCell:=Cells(Z, 25), MaxMinVal:=1, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear"
   SolverAdd CellRef:=Cells(Z, 25), Relation:=3, FormulaText:="0" ' to make non negative
   SolverAdd CellRef:=Cells(Z - 9, 21), Relation:=2, FormulaText:="524" ' 2 is equal
   SolverOk SetCell:=Cells(Z, 25), MaxMinVal:=2, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear"
   SolverOk SetCell:=Cells(Z, 25), MaxMinVal:=2, ValueOf:=0, ByChange:= _
        r1, Engine:=1, EngineDesc:="GRG Nonlinear"

   SolverSolve UserFinish:=True
   SolverFinish KeepFinal:=1

Next i
End Sub
```

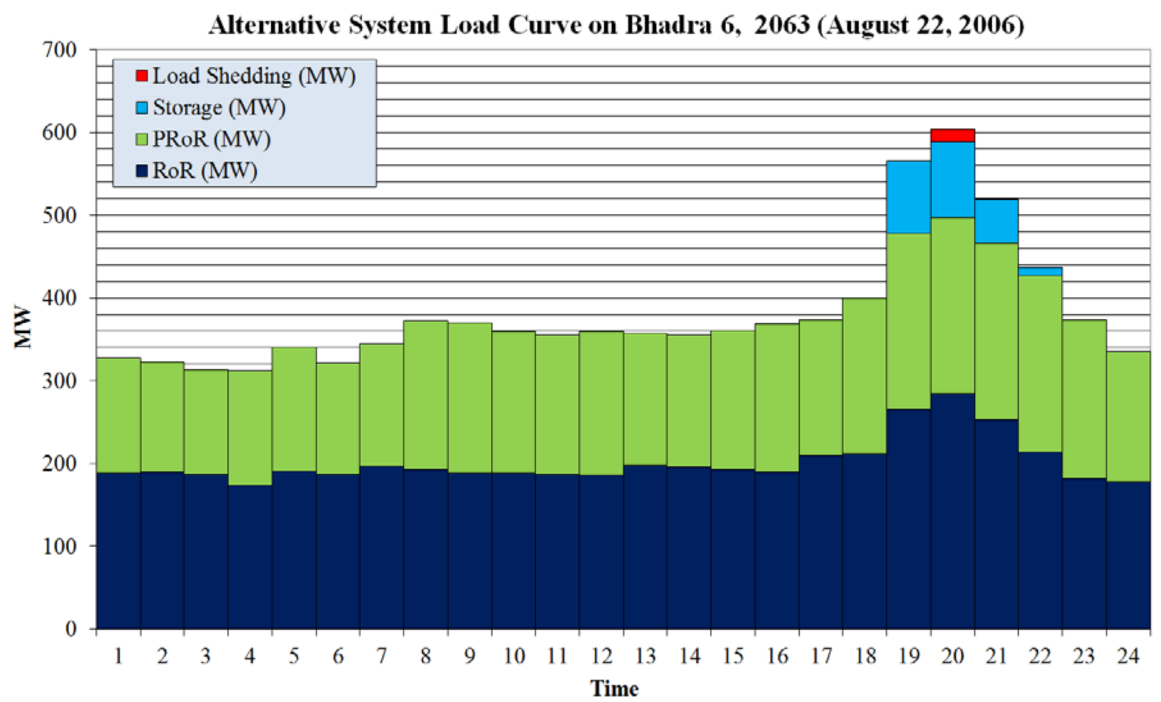
APPENDIX E

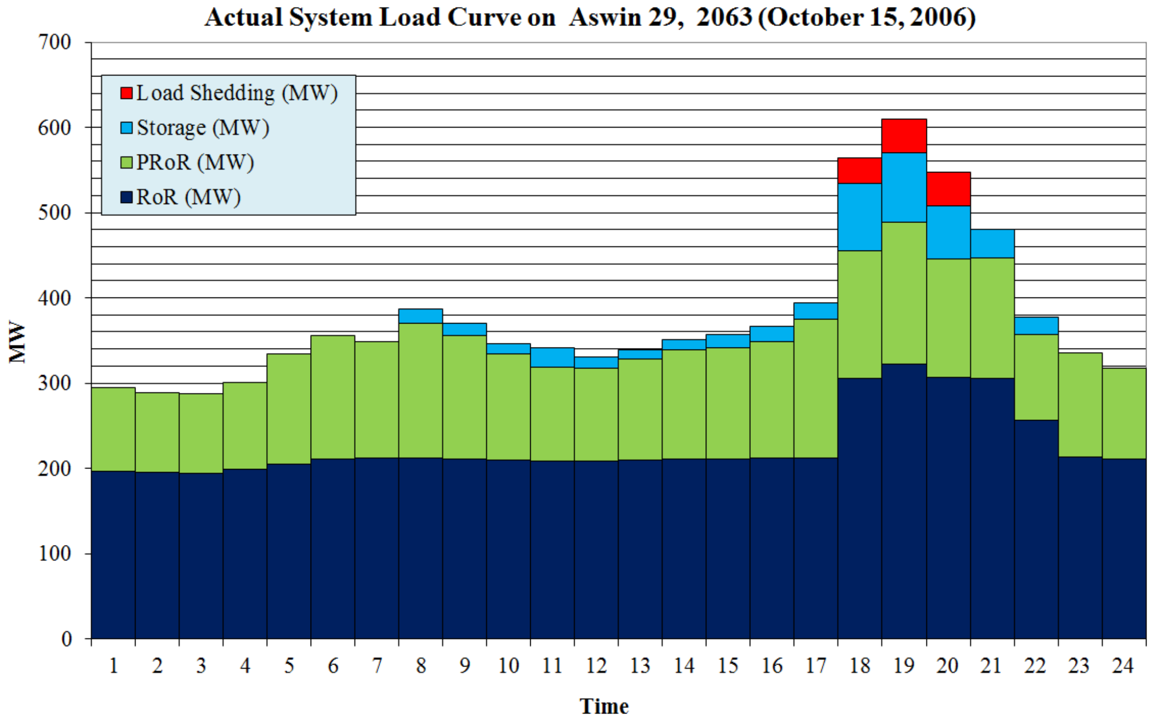
ACTUAL AND ALTERNATIVE SYSTEM LOAD CURVES

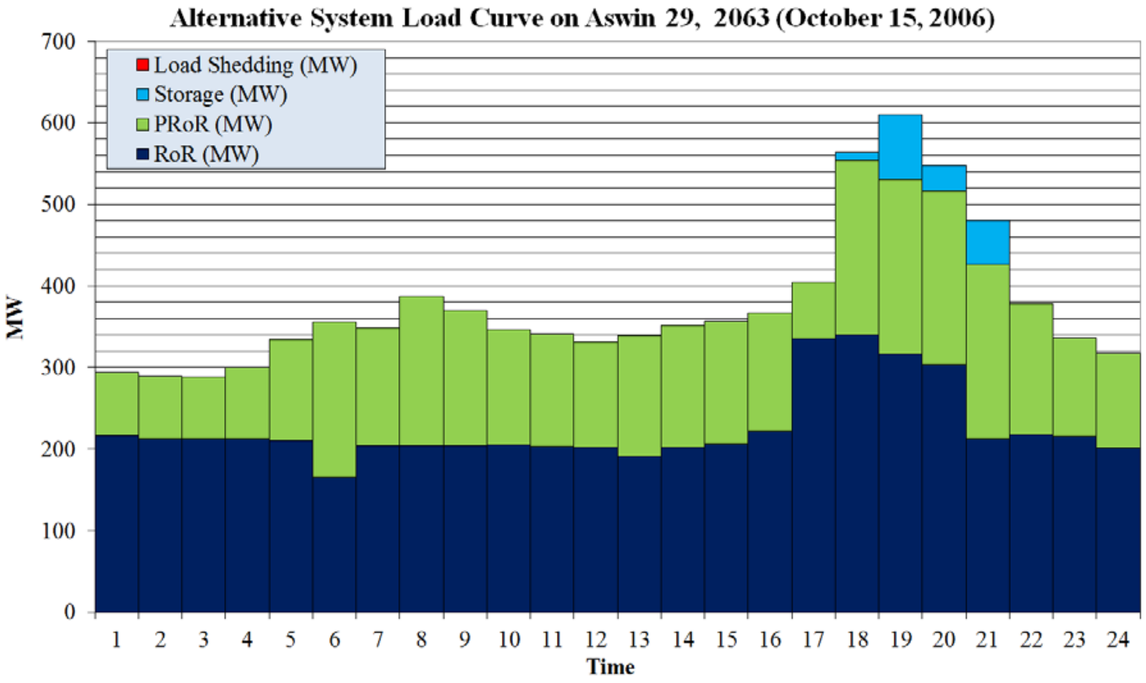


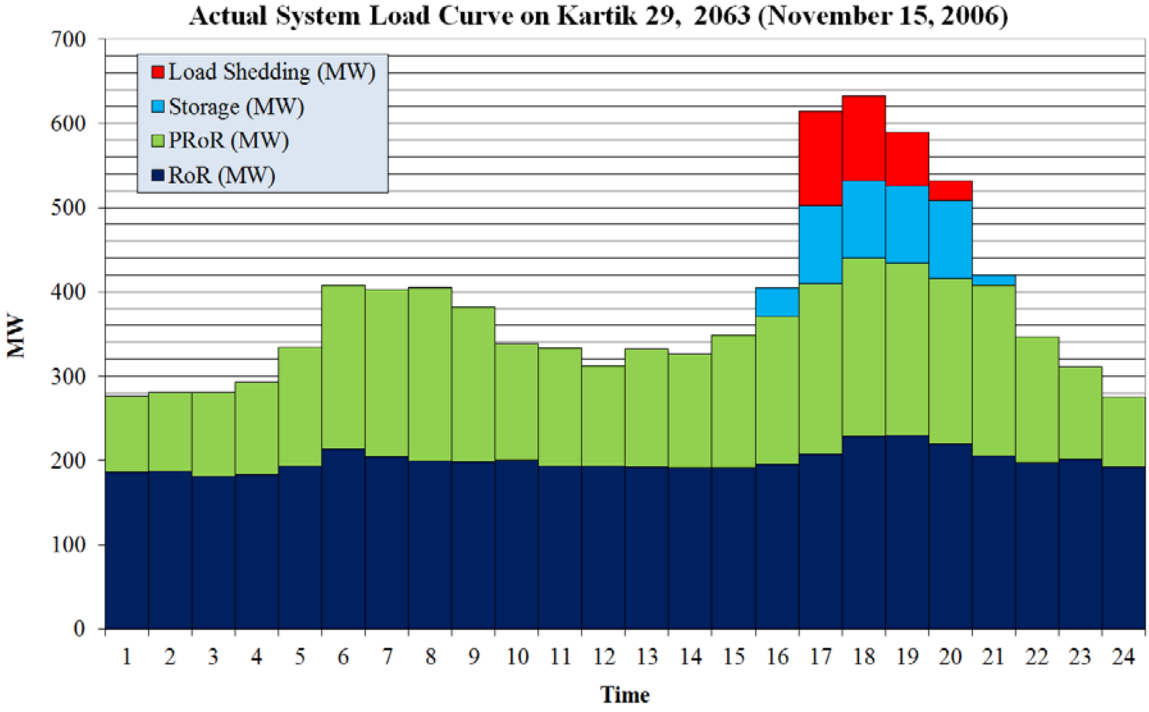


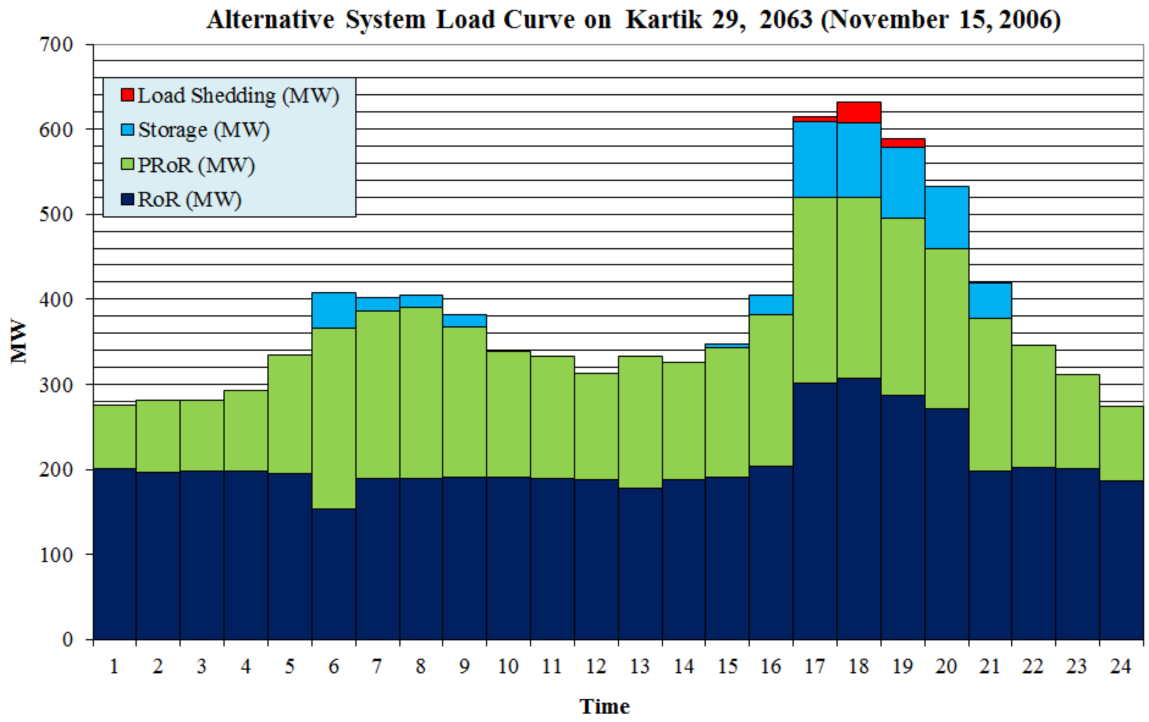

Actual System Load Curve on Bhadra 6, 2063 (August 22, 2006)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time

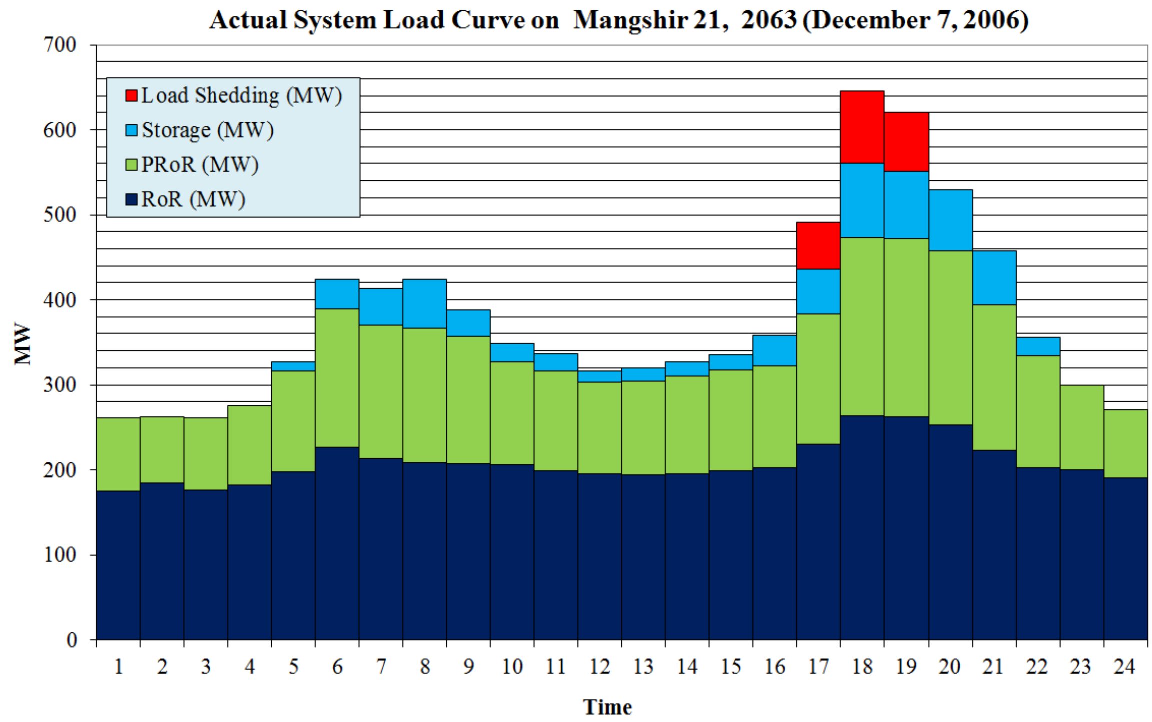

Alternative System Load Curve on Bhadra 6, 2063 (August 22, 2006)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Actual System Load Curve on Aswin 29, 2063 (October 15, 2006)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Alternative System Load Curve on Aswin 29, 2063 (October 15, 2006)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Actual System Load Curve on Kartik 29, 2063 (November 15, 2006)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
Time
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24


Alternative System Load Curve on Kartik 29, 2063 (November 15, 2006)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
Time
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24


Actual System Load Curve on Mangshir 21, 2063 (December 7, 2006)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
Time
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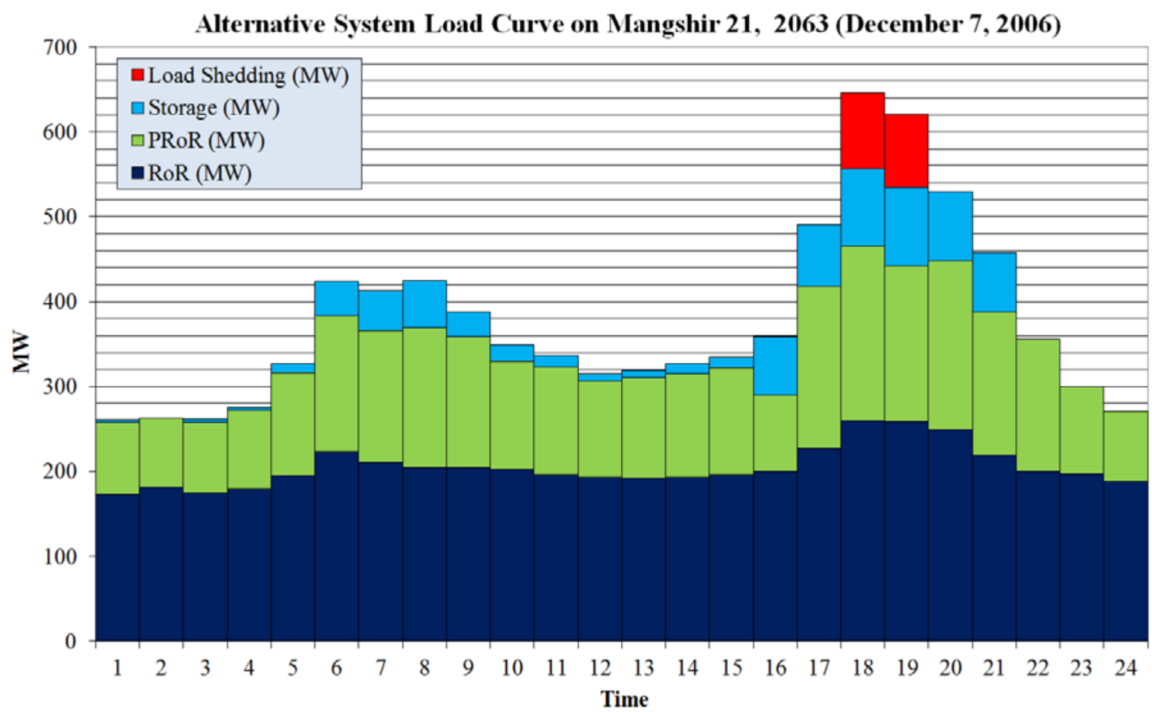
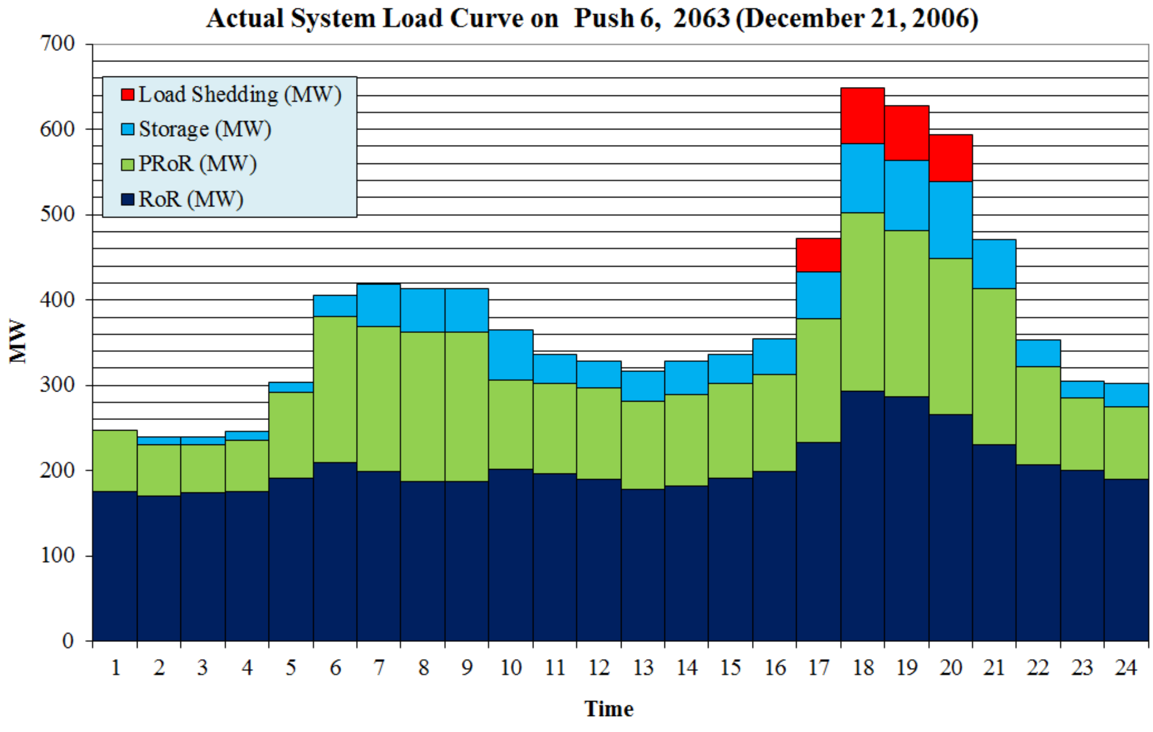
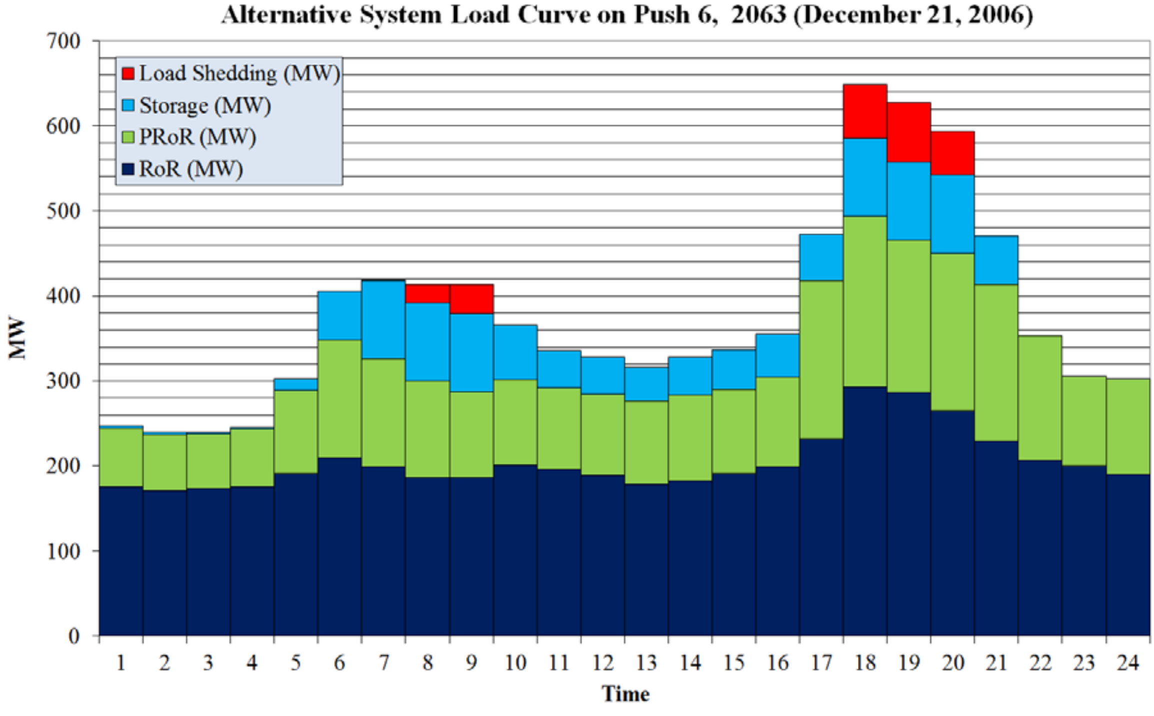
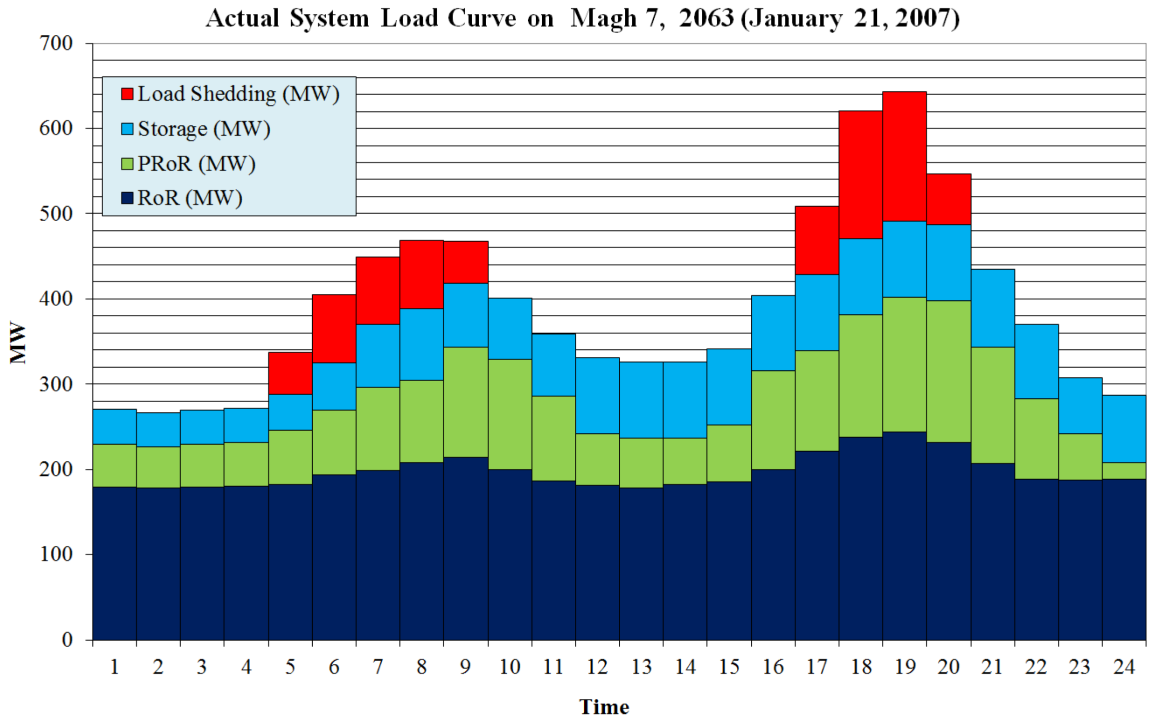
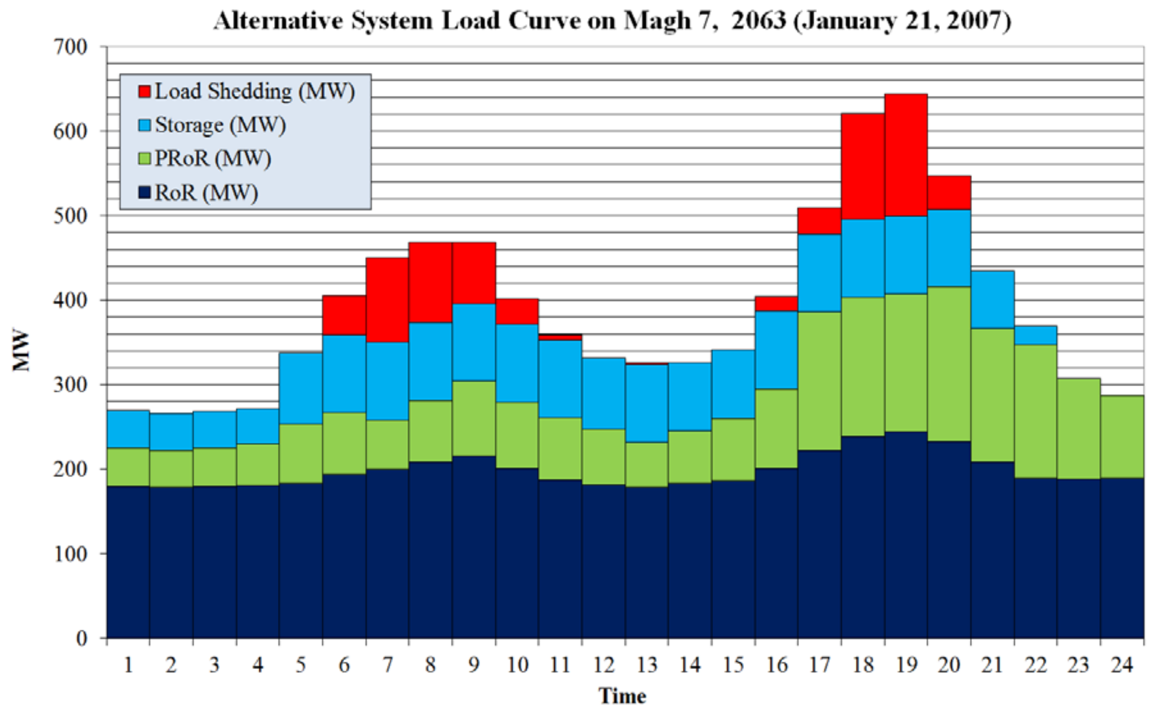
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24

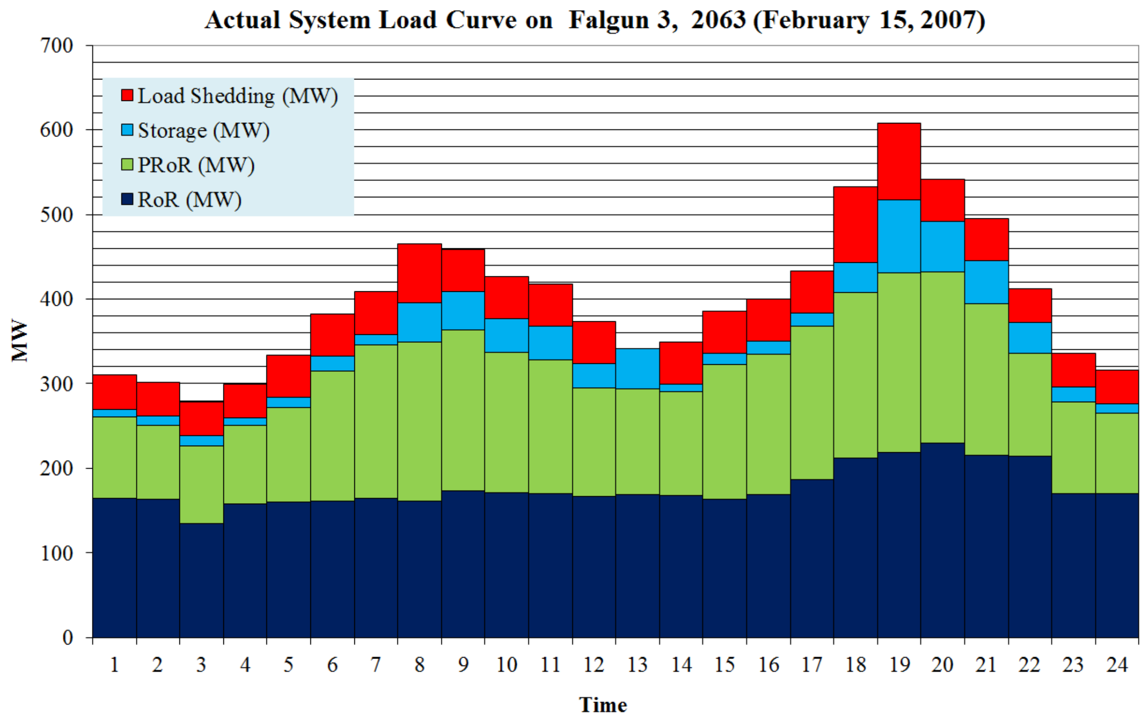

Alternative System Load Curve on Mangshir 21, 2063 (December 7, 2006)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
Time
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24

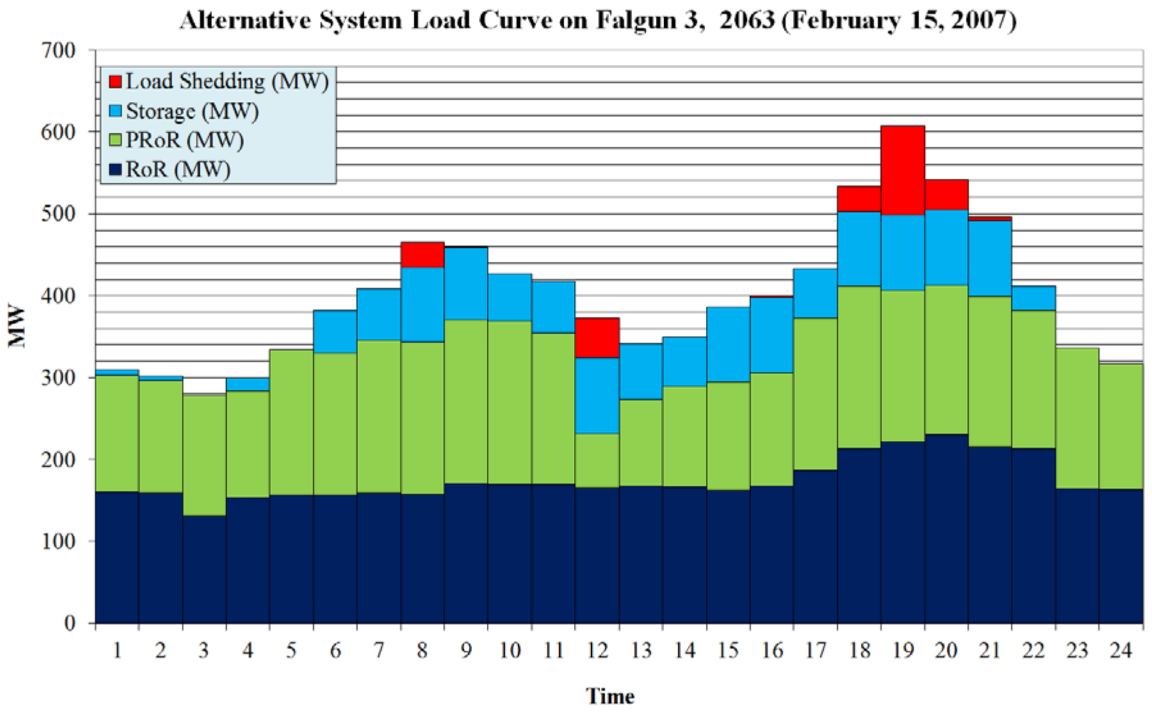

Actual System Load Curve on Push 6, 2063 (December 21, 2006)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time

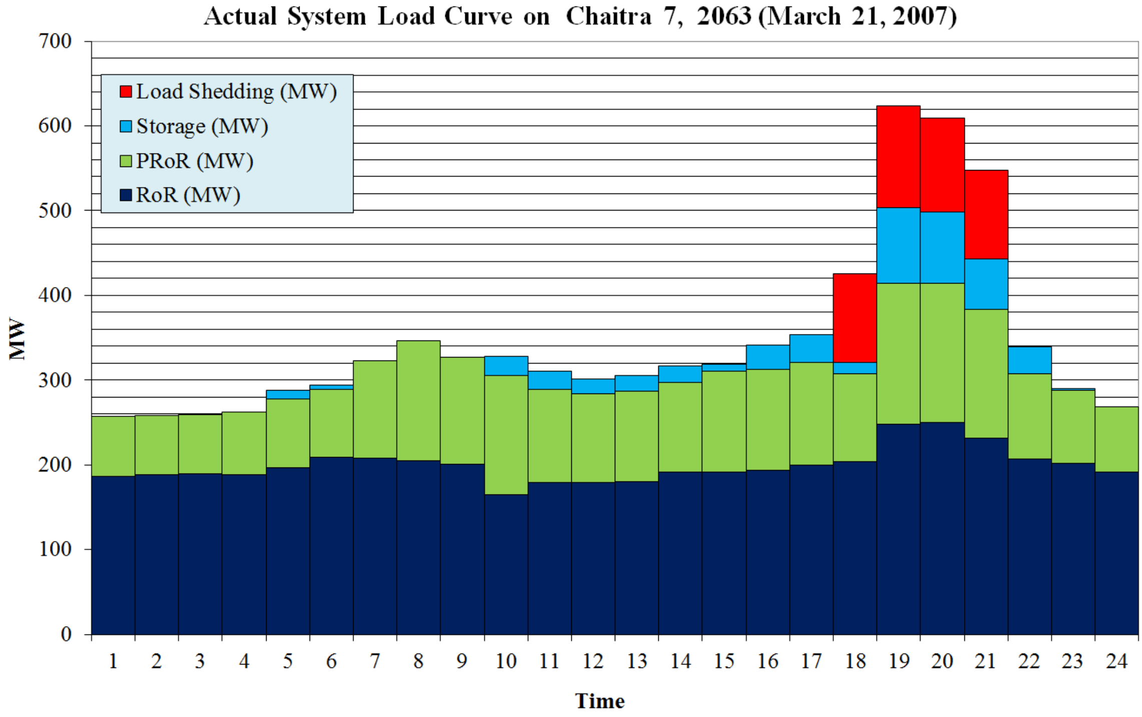

Alternative System Load Curve on Push 6, 2063 (December 21, 2006)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Actual System Load Curve on Magh 7, 2063 (January 21, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Alternative System Load Curve on Magh 7, 2063 (January 21, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Actual System Load Curve on Falgun 3, 2063 (February 15, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Alternative System Load Curve on Falgun 3, 2063 (February 15, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Actual System Load Curve on Chaitra 7, 2063 (March 21, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
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100
200
300
400
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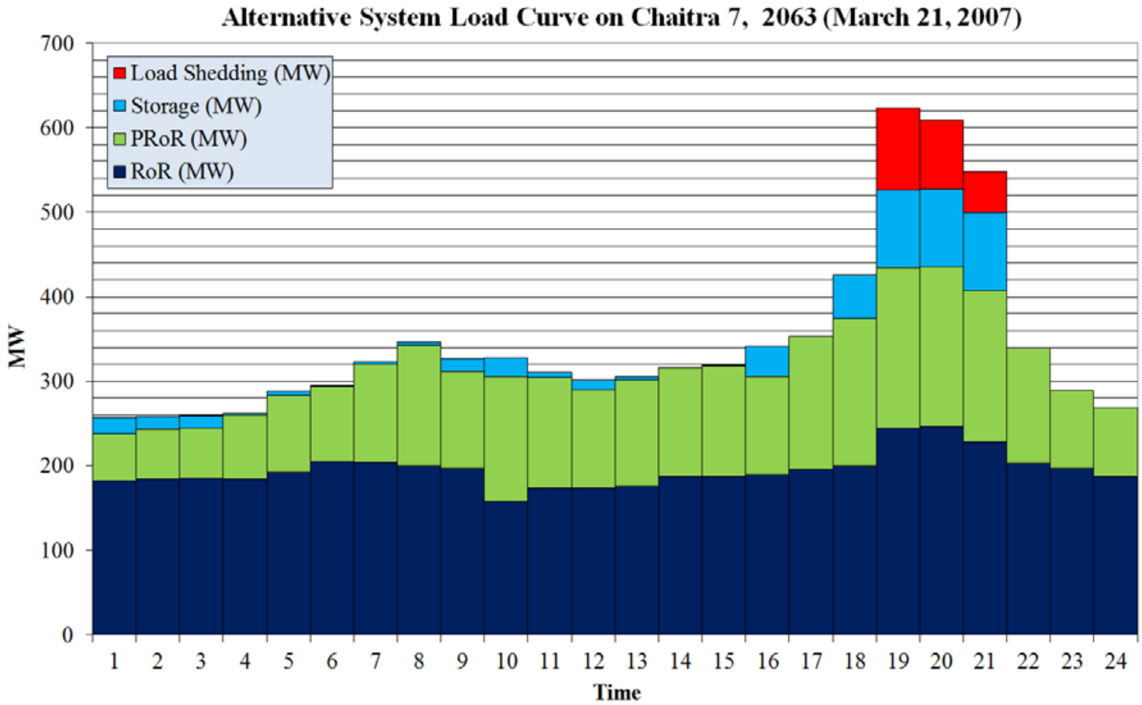
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Alternative System Load Curve on Chaitra 7, 2063 (March 21, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
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100
200
300
400
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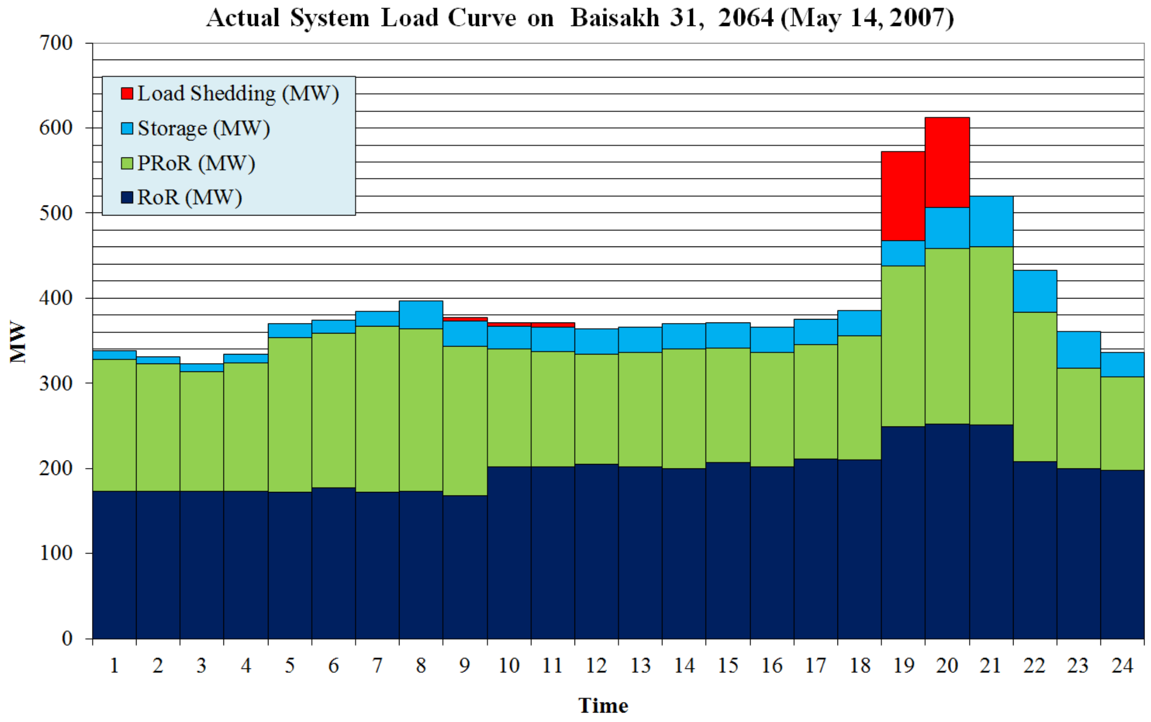
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Actual System Load Curve on Baisakh 31, 2064 (May 14, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
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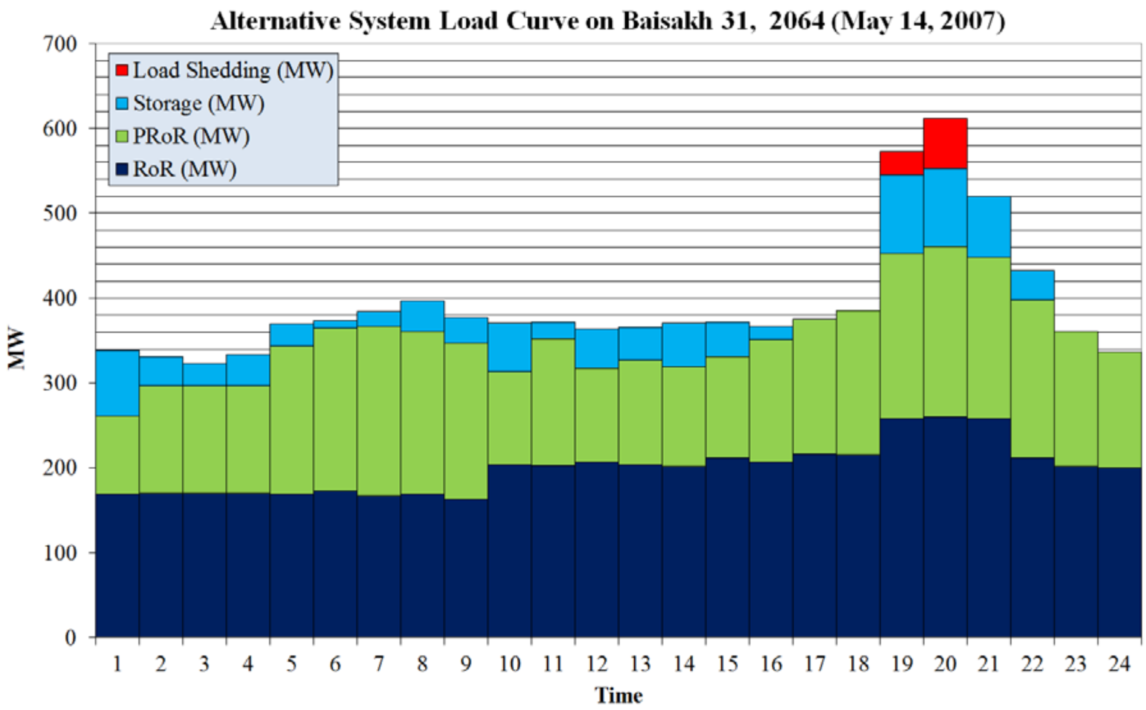
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Alternative System Load Curve on Baisakh 31, 2064 (May 14, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
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100
200
300
400
500
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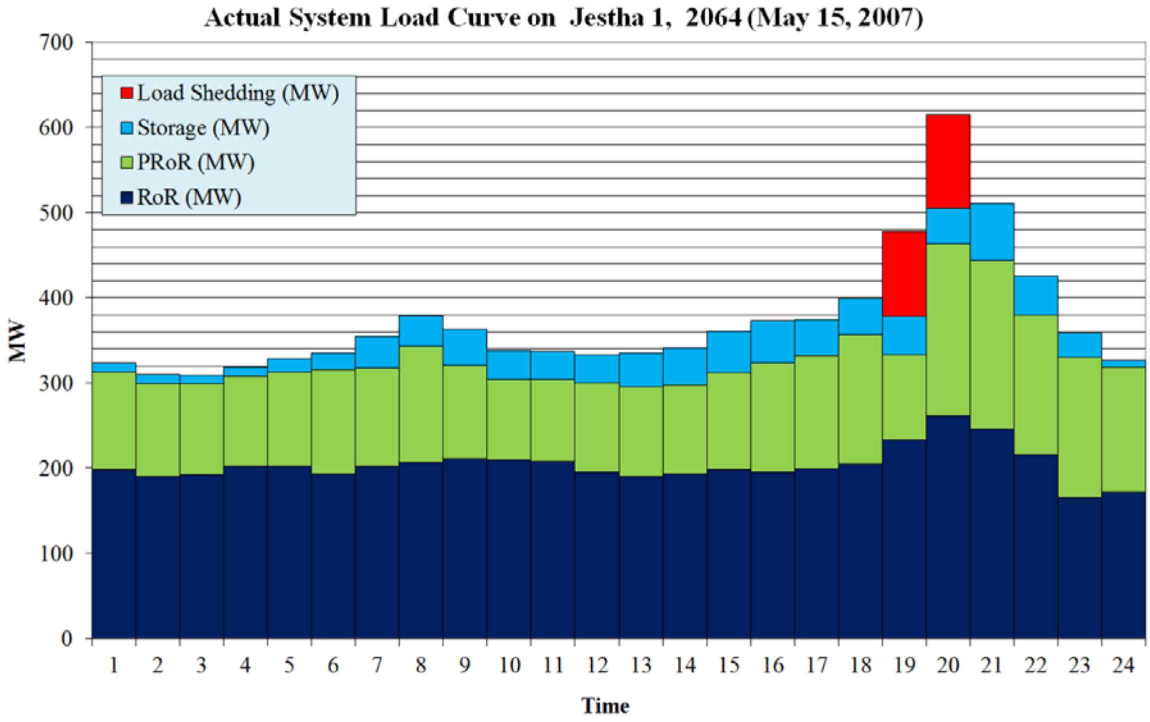
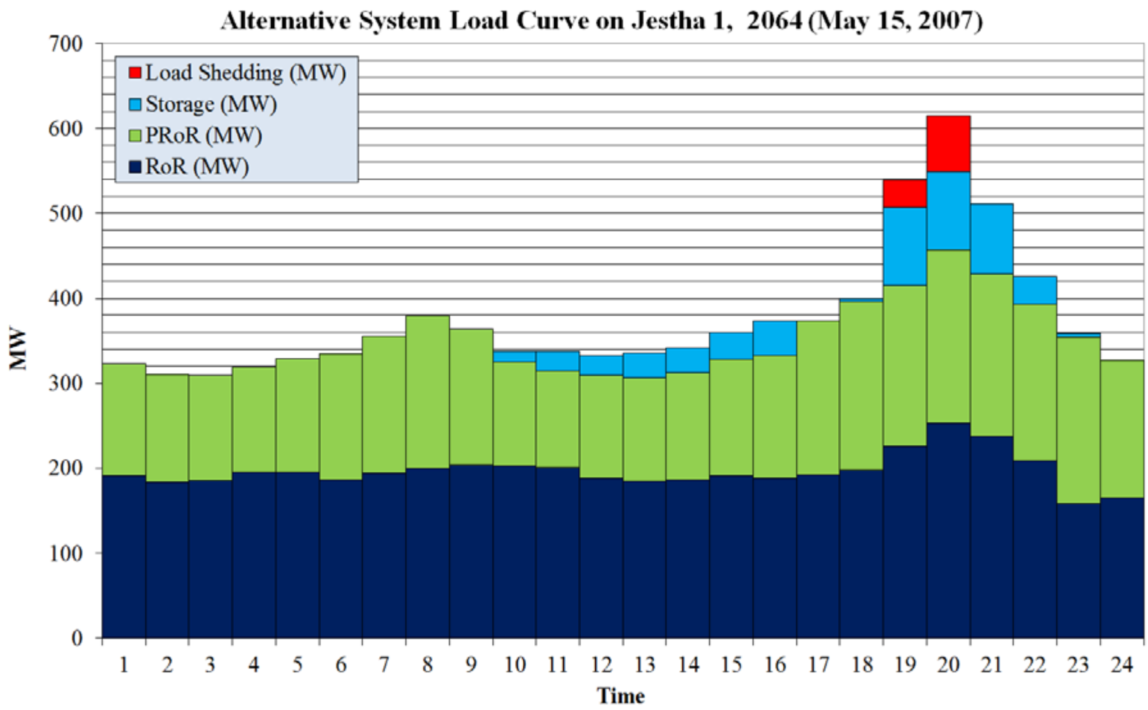
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time


Actual System Load Curve on Jestha 1, 2064 (May 15, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
Time
0
100
200
300
400
500
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24


Alternative System Load Curve on Jestha 1, 2064 (May 15, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
Time
0
100
200
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400
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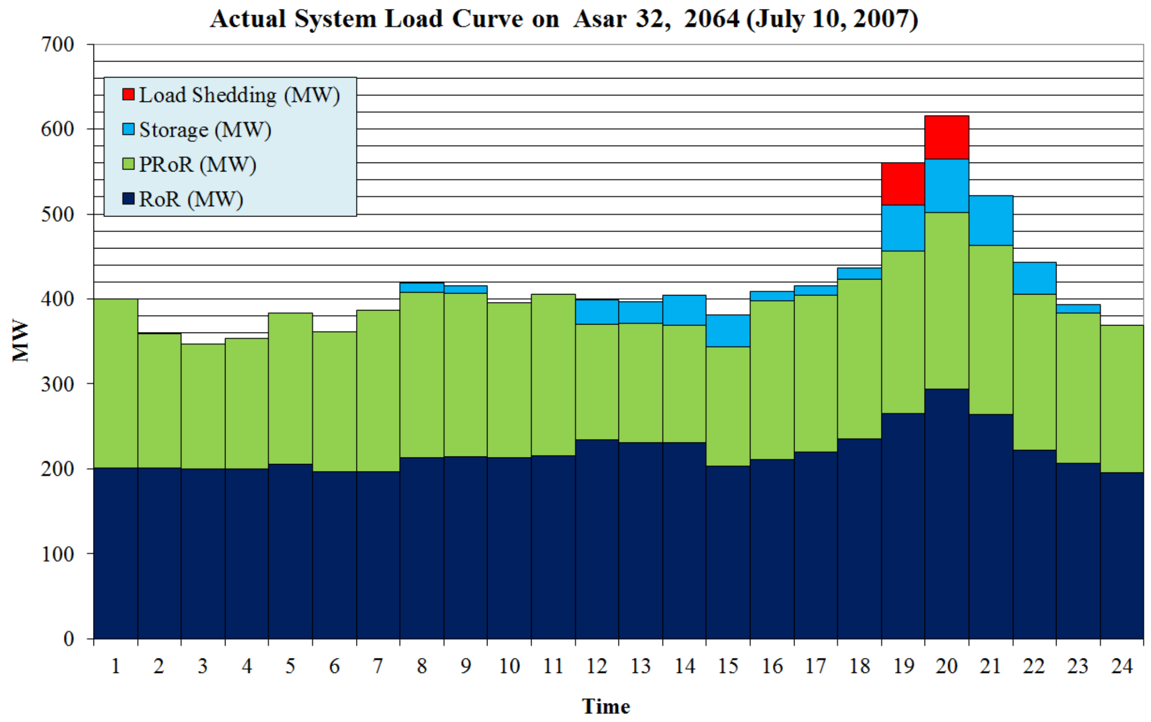
600
700
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24


Actual System Load Curve on Asar 32, 2064 (July 10, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
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2
3
4
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16
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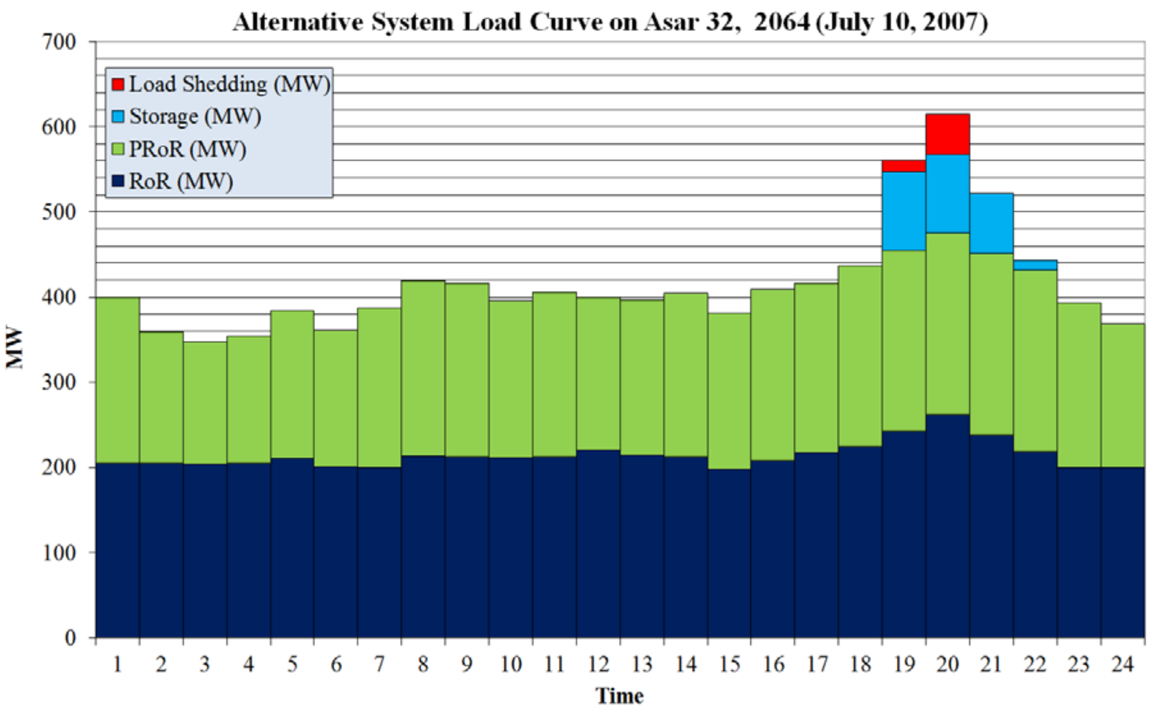
22
23
24
Time


Alternative System Load Curve on Asar 32, 2064 (July 10, 2007)
Load Shedding (MW)
Storage (MW)
PRoR (MW)
RoR (MW)
MW
0
100
200
300
400
500
600
700
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23
24
Time

APPENDIX F

UNIT RATE, QUANTITY AND COST ESTIMATE

F.1 UNIT RATE

Unit Rate				
Exchange rate (USD to NRs) = 80				
Underground Work				
S. No	**Item**	**Unit**	**Rate**	
			NRs	**USD**
1	Reinforcement	kg	92	1.15
2	Excavation	m^3	4,000	50.00
3	Concrete	m^3	14,000	175.00
4	Shotcrete	m^3	25,000	312.50
5	Reinforced shotcrete	m^3	35,000	437.50
6	Rock bolt	2 m	2,000	25.00
7	Rock bolt	3 m	3,200	40.00
8	Formwork	m^2	1,500	18.75
External work				
9	C25 Concrete	m^3	10,500	131.25
10	C35 Concrete	m^3	13,500	168.75
11	Reinforcement	kg	92	1.15
12	Rock excavation	m^3	1,200	15.00
13	Excavation	m^3	176	2.20
14	Formwork	m^2	750	9.38
15	Steel pipe	kg	150	1.88

F.2 QUANTITY AND COST ESTIMATE

F.2.1 FLUSHING SYSTEM

Existing Tunnel Option

S.No.	Description	Unit	Quantity	Rate		Total
				NRs	USD	USD
1	**Intake**					
1.1	**Excavation**	m^3	6,300	176	2.20	13,860
1.2	**Concrete**					
1.2.1	slab	m^3	210	10,500	131.25	27,563
1.2.2	Side wall 1	m^3	300	10,500	131.25	39,375
1.2.3	Side wall 2	m^3	300	10,500	131.25	39,375
1.3	**Reinforcement**					
1.3.1	slab	kg	20,606	92	1.15	23,697
1.3.2	Side wall 1	kg	29,438	92	1.15	33,853
1.3.3	Side wall 2	kg	29,438	92	1.15	33,853
	Sub Total					**211,576**
2	**Access to gate operation shaft**					
2.1	**Excavation**	m^3	2,310	1,200	15.00	34,650
2.2	**2 m Rock bolts**	Nos	280	2,000	25.00	7,000
	Sub Total					**41,650**
3	**Vertical shaft for gate operation**					
3.1	**Excavation**	m^3	1,730	4,000	50.00	86,490
3.2	**Concrete**					
3.2.1	Side walls	m^3	162	14,000	175.00	28,350
3.2.2	Side wall 1	m^3	81	14,000	175.00	14,175
3.2.3	Side wall 2	m^3	81	14,000	175.00	14,175
3.3	**Reinforcement**					
3.3.1	Side walls	kg	15,896	92	1.15	18,281
3.3.2	Side wall 1	kg	7,948	92	1.15	9,140
3.3.3	Side wall 2	kg	7,948	92	1.15	9,140
	Sub Total					**179,751**

S.No.	Description	Unit	Quantity	Rate		Total
				NRs	USD	USD
4	**Horizontal tunnel**					
4.1	**Excavation**	m^3	4,191	4,000	50.00	209,560
4.2	**Concrete**					
4.2.1	Base Slab	m^3	242	14,000	175.00	42,315
4.2.2	Side wall 1	m^3	242	14,000	175.00	42,315
4.2.3	Side wall 2	m^3	242	14,000	175.00	42,315
4.3	**Reinforcement**					
4.3.1	slab	kg	23,727	92	1.15	27,286
4.3.2	Side wall 1	kg	23,727	92	1.15	27,286
4.3.3	Side wall 2	kg	23,727	92	1.15	27,286
	Sub Total					**418,362**
5	**Vertical shaft**					
5.1	**Excavation**	m^3	1,612	4,000	50.00	80,600
5.2	**Concrete**					
5.2.1	Side walls	m^3	78	14,000	175.00	13,650
5.2.2	Side wall 1	m^3	78	14,000	175.00	13,650
5.2.3	Side wall 2	m^3	78	14,000	175.00	13,650
5.3	**Reinforcement**					
5.3.1	Side walls	kg	7,654	92	1.15	8,802
5.3.2	Side wall 1	kg	7,654	92	1.15	8,802
5.3.3	Side wall 2	kg	7,654	92	1.15	8,802
5.4	**Grouting**			3,000,000	37,500	37,500
	Sub Total					**185,455**
6	**Concrete plug**					
6.1	**Concrete**	m^3	211	14,000	175.00	36,969
6.2	**Reinforcement**	kg	1,016	92	1.15	1,168
	Sub Total					**38,137**
7	**Treatment for abrasive resistivity of the surface**			32,250,000	403,125	403,125
8	**Radial gate**			30,100,000	376,250	376,250
9	**Stop log**			14,700,000	183,750	183,750
10	**Ladders and Fencing**			2,500,000	31,250	31,250
	Total					**2,069,306**
11	**Unforeseen/miscellaneous**			20 % of total		413,861
	Total cost of flushing system (Existing tunnel)					**2,483,168**

New Tunnel Option

S.No.	Description	Unit	Quantity	Rate		Total
				NRs	USD	USD
1	**Intake**					
1.1	**Excavation**	m^3	6,300	176.00	2.20	13,860
1.2	**Concrete**					
1.2.1	slab	m^3	210	10,500	131.25	27,563
1.2.2	Side wall 1	m^3	300	10,500	131.25	39,375
1.2.3	Side wall 2	m^3	300	10,500	131.25	39,375
1.3	**Reinforcement**					
1.3.1	slab	kg	20,606	92	1.15	23,697
1.3.2	Side wall 1	kg	29,438	92	1.15	33,853
1.3.3	Side wall 2	kg	29,438	92	1.15	33,853
	Sub Total					**211,576**
2	**Access to gate operation shaft**					
2.1	**Excavation**	m^3	1,041	4,000	50.00	52,063
2.2	**2 m Rock bolts**	Nos	510	2,000	25.00	12,750
2.3	**Shotcrete**	m^3	108	25,000	312.50	33,867
	Sub Total					**98,680**
3	**Vertical shaft for gate operation**					
3.1	**Excavation**	m^3	2,806	4,000	50.00	140,306
3.2	**Concrete**					
3.2.1	Side walls	m^3	263	14,000	175.00	45,990
3.2.2	Side wall 1	m^3	131	14,000	175.00	22,995
3.2.3	Side wall 2	m^3	131	14,000	175.00	22,995
3.3	**Reinforcement**					
3.3.1	Side walls	kg	25,787	92	1.15	29,655
3.3.2	Side wall 1	kg	12,894	92	1.15	14,828
3.3.3	Side wall 2	kg	12,894	92	1.15	14,828
	Sub Total					**291,597**

S.No.	Description	Unit	Quantity	Rate NRs	Rate USD	Total USD
4	**Tunnel**					
4.1	**Excavation**	m^3	5,628	4,000	50.00	281,400
4.2	**Concrete**					
4.2.1	Base Slab	m^3	704	14,000	175.00	123,113
4.2.2	Side wall 1	m^3	1,089	14,000	175.00	190,531
4.2.3	Side wall 2	m^3	1,089	14,000	175.00	190,531
4.3	**Reinforcement**					
4.3.1	slab	kg	69,031	92	1.15	79,386
4.3.2	Side wall 1	kg	106,834	92	1.15	122,859
4.3.3	Side wall 2	kg	106,834	92	1.15	122,859
	Sub Total					**1,110,678**
5	**Treatment for abrasive resistivity of the surface**			50,232,000	627,900	627,900
6	**Radial gate**			30,100,000	376,250	376,250
7	**Stop log**			14,700,000	183,750	183,750
8	**Ladders and Fencing**			2,500,000	31,250	31,250
	Total					**2,931,680**
9	**Unforeseen/miscellaneous**	20 % of total				586,336
	Total cost of flushing system (New tunnel)					**3,518,016**

F.2.2 HSRS

Existing Tunnel Option

S.No.	Description	Unit	Quantity	Rate		Total
				NRs	USD	USD
1	**Intake**					
1.1	**Excavation**	m^3	120	176.00	2.20	264
1.2	**Concrete**					
1.2.1	slab	m^3	6	10,500	131.25	788
1.2.2	Side wall 1	m^3	1	10,500	131.25	164
1.2.3	Side wall 2	m^3	1	10,500	131.25	164
1.3	**Reinforcement**					
1.3.1	slab	kg	589	92	1.15	677
1.3.2	Side wall 1	kg	123	92	1.15	141
1.3.3	Side wall 2	kg	123	92	1.15	141
	Sub Total					**2,339**
2	**Inclined shaft**					
2.1	**Excavation**	m^3	332	4,000	50.00	16,610
2.2	**Shotcrete**	m^3	63	14,000	175.00	10,953
2.3	**Reinforcement**	kg	6,141	92	1.15	7,062
	Sub Total					**34,625**
3	**Concrete plug**					
3.1	**Concrete**	m^3	211	14,000	175.00	36,969
3.2	**Reinforcement**	kg	1,016	92	1.15	1,168
	Sub Total					**38,137**
4	**Treatment for abrasive resistivity of the surface**			42,750,000	534,375	534,375
5	**Hydro-mechanical parts**			3,500,000	43,750	43,750
6	**Steel pipe**	kg	32,182	150	1.88	60,341
7	**Four suction heads, outlets sections, valves**			8,000,000	100,000	100,000
8	**High quality suction hose pipe**	m	400	25,000	312.50	125,000
9	**Flexible high quality pipe**		6,000	15,000	187.50	1,125,000
10	**Floaters for pipe suspension, Rafts to handle the suction head**			4,000,000	50,000	50,000
11	**Grouting**			2,800,000	35,000	35,000
	Total					**2,148,566**
12	**Unforeseen/miscellaneous**	20 % of total				429,713
	Total cost of HSRS (Existing tunnel)					**2,578,280**

New Tunnel Option

S.No.	Description	Unit	Quantity	Rate		Total
				NRs	USD	USD
1	**Intake**					
1.1	**Excavation**	m^3	120	176.00	2.20	264
1.2	**Concrete**					
1.2.1	slab	m^3	6	10,500	131.25	788
1.2.2	Side wall 1	m^3	1	10,500	131.25	164
1.2.3	Side wall 2	m^3	1	10,500	131.25	164
1.3	**Reinforcement**					
1.3.1	slab	kg	589	92	1.15	677
1.3.2	Side wall 1	kg	123	92	1.15	141
1.3.3	Side wall 2	kg	123	92	1.15	141
	Sub Total					**2,339**
2	**Inclined shaft**					
2.1	**Excavation**	m^3	332	4,000.00	50.00	16,610
2.2	**Shotcrete**	m^3	63	14,000	175.00	10,953
2.3	**Reinforcement**	kg	6,141	92	1.15	7,062
	Sub Total					**34,625**
3	**Concrete plug**					
3.1	**Concrete**	m^3	36	14,000	175.00	6,256
3.2	**Reinforcement**	kg	172	92	1.15	198
	Sub Total					**6,454**
4	**Flushing tunnel**					
4.1	**Excavation**	m^3	3,168	4,000	50.00	158,400
4.2	**Concrete**					
4.2.1	Base Slab	m^3	528	14,000	175.00	92,400
4.2.2	Side wall 1	m^3	594	14,000	175.00	103,950
4.2.3	Side wall 2	m^3	594	14,000	175.00	103,950
4.3	**Reinforcement**					
4.3.1	Base Slab	kg	51,810	92	1.15	59,582
4.3.2	Side wall 1	kg	58,286	92	1.15	67,029
4.3.3	Side wall 2	kg	58,286	92	1.15	67,029
	Sub Total					**652,340**

S.No.	Description	Unit	Quantity	Rate		Total
				NRs	USD	USD
5	**Treatment for abrasive resistivity of the surface**			28,050,000	350,625	350,625
6	**Hydro-mechanical parts**			3,500,000	43,750	43,750
7	**Steel Pipe**	kg	32,182	150	1.88	60,341
8	**Four suction heads, outlets sections, valves**			8,000,000	100,000	100,000
9	**High quality suction hose pipe**	m	400	25,000	312.50	125,000
10	**Flexible high quality pipe**	m	6,000	15,000	187.50	1,125,000
11	**Floaters for pipe suspension, Rafts to handle the suction head**			4,000,000		50,000
12	**Grouting**			2,800,000		35,000
	Total					**2,585,473**
13	**Unforeseen/miscellaneous**	20 % of total				517,095
	Total cost of HSRS (New tunnel)					**3,102,568**