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Content

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SIMULATION OF HYDRAULIC PARAMETERS IN WATER DISTRIBUTION NETWORK USING EPANET: A CASE STUDY OF SURAT CITY

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Abstract

With the increase in population, demand of water supply on the civic amenities including water supply for domestic purposes, irrigation, industry etc. has increased. Therefore, identification of sources of water supply, their conservation and optimal utilization is of utmost importance. Water demand is increasing day by day whether it is domestic, industrial and agricultural etc., but the source of water is limited. So, authorities around the world are faced with the problem to provide sufficient water from limited water source. To solve this problem, design of new or up-gradation of existing water distribution network is to be necessary. So, such type of problem can be solved manually as well as by using different technologies like LOOP 4.0, MIKENET, STANET, EPANET and WATERGEMS software. As, different areas of Surat City faced problems like water scarcity and pressure. Based on these problems, we have selected our Study area as Punagam Region of Surat City. In the present study analysis of existing Water Distribution Network is simulated using hydraulic software EPANET and to address any improvements required in existing network and the mode of operation, in order to improve the quality and quantity of water which is supplied or distributed to the consumers. Punagam area consists of five ESR (Elevated Storage Reservoir). Simulation has been carried out for hydraulic parameters like head, pressure, flow rate, hydraulic gradient and head loss. Water Distribution Network of village was designed with the EPANET software and compared with actual data. The main focus of this study is to analyze the existing water distribution network and identify deficiencies (if any) in its analysis, *implementation and its usage. At the end of the analysis it was found that the resulting pressures at all the junctions and the flows with their velocities at all pipes are adequate enough to provide water to the study area. Based on this study, necessary recommendations are made either to use large diameter pipes instead of small diameter pipes. It is also recommended to increase the discharge of that region so that it will achieve the base demand. This study helps the water supply engineers in saving time as this process is fast and less tedious.*

Keywords: *Demand, Elevated Storage Reservoir, EPANET, Flow rate, Water Distribution Network*

Introduction

Water distribution system, a hydraulic infrastructure consisting of elements such as pipes, tanks, reservoirs, pumps and valves etc., is crucial to provide water to the consumers. Effective water supply system is of paramount importance in designing a new water distribution network or in expanding the existing one. Distribution networks are an essential part of all water supply systems. Distribution system costs within any water supply scheme may be equal to or greater than 60 % of the entire cost of the project. A water distribution system is a collection of hydraulic control elements jointly connected to convey quantities of water from sources to consumers. Simulation of water distribution systems using computer technique has reached a mature stage of development. However, the optimal network design is quite complicated due to nonlinear relationship between flow and head loss and the presence of discrete variables. There is still not a convenient evaluation for the reliability of water distribution systems. Traditionally, a water distribution network design is based on the proposed street plan and the topography. The primary task for water utilities is to deliver water of the required quantity to individual customers under sufficient pressure through a distribution network. The distribution of drinking water in distribution networks is technical challenge both in quantitative and qualitative terms. It is essential that each point of the distribution network be supplied without an invariable flow of water complying with all the qualitative and quantitative parameters. The water supply in most Indian cities is only available for a few hours per day, pressure is irregular, and the water is of questionable quality. Intermittent water supply, insufficient pressure and unpredictable service impose both financial and health costs on Indian households. Leakage hotspots are assumed to exist at the model nodes identified. For this study area Punagam Zone of Surat City has been identified and the simulation of the existing network is carried out.

1.1 Aim of the Study

To assess the performance of Water Distribution Network of Punagam area of Surat City using hydraulic simulation software i.e. EPANET and to address any improvements required in existing network and the mode of operation, in order to improve the quantity and quality of water distributed to the consumers.

1.2 Objective of Study

Ÿ To study the existing water supply network of PUNAGAMarea of SURAT city.

Ÿ To collect pipe report and junction report of existing network.

Ÿ To analyze the data by using EPANET software.

Ÿ To check the discharge & pressure head in existing network.

2. Study Area

Punagam area is a part of Surat city. Punagam area is located in East zone of Surat. The population of study area is 2, 22,252. The study area covers residential area about 600.83 Ha. When the water from the distribution network reaches to the Punagam area there is sudden decrease in the pressure head due to which water related problems arises. Leakages, failure of pipes and other factors are there which affects the water distribution network. Therefore its required to analyze the existing network of the Punagam area using EPANET and compared computed result with actual result which is obtained from Surat Municipal Corporation.

The water distribution system of Punagam area i.e. WDS-E3 network systems ESR-E7, ESR-E8, ESR-E9, ESR-E9A, ESR-E10. In this present paper analysis of WDS ESR-E9, WDS ESR-E9A and WDS ESR-E10.

Figure 1 Map of Punagam Area, Surat City

3. Methodology

This includes overview of EPANET software & methodology to analyze the flow of water distribution network.

3.1 EPANET Software

EPANET is developed by the US Environmental Protection Agency. It is a computer program that performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, etc. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps. In addition to chemical species, water age and source tracing can also be simulated.

EPANET is designed to be a research tool for improving our understanding of the movement and fate of drinking water constituents within distribution systems. It can be used for many different kinds of applications in distribution systems analysis. Sampling program design, hydraulic model calibration, chlorine residual analysis, and consumer exposure assessment are some examples.

EPANET can help assess alternative management strategies for improving water quality throughout a system.

Running under windows, EPANET provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include color-coded network maps, data tables, time series graphs, and contour plots.

EPANET was developed by the water supply and water resources division (formerly the drinking water research division) of the U.S Environmental protection agency's national risk management research laboratory. It is public domain software that may be freely copied and distributed.

3.2 Hydraulic Modeling Capabilities

Full-featured and accurate hydraulic modeling is a prerequisite for doing effective water quality modeling. EPANETcontains a state-of-the-art hydraulic analysis engine that includes the following capabilities:

- Places no limit on the size of the network that can be analyzed
- Computes friction head loss using the Hazen-William, Darcy-

Weisbach or Chezy-Manning formula

- Includes minor head losses for bends, fittings, etc.
- Models constant or variable speed pumps
- Computes pumping energy and cost
- Models various types of valves including shutoff, check, pressure regulating, and flow control valves
- Allows storage tanks to have any shape (i.e., diameter can vary with height)
- Considers multiple demand categories at nodes, each with its own pattern of time variation
- Models pressure-dependent flow issuing from emitters (sprinkler heads)
- Can perform system operation on both simple tank level and timer controls and on complex rule-based controls.

EPANET's Windows user interface provides a network editor that simplifies the process of building piping network models and editing their properties. Various data reporting and visualization tools such as graphical views, tabular views, and special reports, and calibration are used to assist in interpreting the results of a network analysis (EPA, 2000).

By employing these features, EPANET can study water quality phenomena as:

- Blending water from different sources
- Age of water throughout a system
- Loss of chlorine residuals.
- Growth of disinfection by-products.
- Tracking contaminant propagation events.

3.3 Model Input Data

- In order to analyze the WDN using EPANET following input data files are needed:
- Junction report
- Pipe report

3.3.1 Junction Report

Junctions are points in the network where links join together and where water enters or leaves the network. The basic input data required for junctions are:

- Elevation above some reference (usually mean sea level)
- Water demand (rate of withdrawal from the network)
- Initial water quality.

The output results computed for junctions at all time periods of a simulation are:

- Hydraulic head (internal energy per unit weight of fluid)
- Pressure

- Water quality

Junctions can also:

- Have their demand vary with time
- Have multiple categories of demands assigned to them
- Have negative demands indicating that water is entering the network
- Be water quality sources where constituents enter the network
- Contain emitters (or sprinklers) which make the outflow rate depend on the pressure.

3.3.2 Pipe Report

Pipes are links that convey water from one point in the network to another. EPANET assumes that all pipes are full at all times. Flow direction is from the end at higher hydraulic head (internal energy per weight of water) to that at lower head.

The principal hydraulic input parameters for pipes are:

- Start and end nodes
- Diameter
- Length
- Roughness coefficient (for determining head loss)
- Status (open, closed, or contains a check valve).

Computed outputs for pipes include:

- Flow rate
- Velocity
- Head loss
- Darcy-Weisbach friction factor
- Average reaction rate (over the pipe length)
- Average water quality (over the pipe length).

The hydraulic head lost by water flowing in a pipe due to friction with the pipe walls can be computed using one of three different formulas:

- Hazen-Williams formula
- Darcy-Weisbach formula
- Chezy-Manning formula

The Hazen-Williams formula is the most commonly used head loss formula in the US. It cannot be used for liquids other than water and was originally developed for turbulent flow only. The Darcy-Weisbach formula is the most theoretically correct. It applies over all flow regimes and to all liquids. The Chezy-Manning formula is more commonly used for open channel flow. Each formula uses the following equation to compute head loss between the start and end node of the pipe:

$$\mathbf{H_L = aq^b}$$

Where,

H_L= head loss (length),

q = flow rate (volume/time),

a = resistance coefficient,

b = flow exponent.

Table 1 lists expressions for the resistance coefficient and values for the flow exponent for each of the formulae. Each formula uses a different pipe roughness coefficient that must be determined empirically. Table 2 lists general ranges of these coefficients for different types of new pipe materials. Be aware that a pipe's roughness coefficient can change considerably with age.

Table 1 - Pipe Head Loss Formula for Full Flow

Formula	Resistance coefficient (A)	Flow exponent
Hazen-Williams	$4.727^{-1.852}d^{-4.781}L$	1.852
Darcy-Weisbach	$0.0252f(\varepsilon,d,q)d^{-5}L$	2
Chezy-Manning	$4.66n^{2}d^{-5.33}L$	2
Notes: C = Hazen-Williams roughness coefficient ε = Darcy-Weisbach roughness coefficient (ft.) f = friction factor (dependent on ε, d and q) n = Manning roughness coefficient d = pipe diameter (ft.) L = pipe length (ft.) q = flow rate (cfs)		

Table 2 - Roughness Coefficient for new pipe

Material	Hazen-williams C	Darcy-Weisbach ε	Manning's n
Cast Iron	130-140	0.85	0.012-0.015
Concrete or concrete lined	120-140	1.0-10	0.012-0.017
Galvanized Iron	120	0.5	0.015-0.017
Plastic	140-150	0.005	0.011-0.015
Steel	140-150	0.15	0.015-0.017
Vitrified Clay	110		0.013-0.015

Pipes can be set open or closed at preset times or when specific conditions exist, such as when tank levels fall below or above certain set points, or when nodal pressures fall below or above certain values.

3.3.3 Steps to Analyze the Water Distribution Network

Following steps has been carried out to model a water distribution network using EPANET:

Ÿ Draw a network representation of your distribution system or import a basic description of the network placed in a text file.

Ÿ Edit the properties of the objects that make up the system. It includes editing the properties and entering of the required data in various objects like reservoir, pipes, nodes or junctions, etc.

Ÿ Describe how the system is operated

Ÿ Run a Hydraulic/Water Quality Analysis

Ÿ View the results of the analysis which can be viewed in various forms i.e. in form of tables and graphs.

Ÿ Repeat the procedure for the other distribution network i.e WDS ESR-E9A and WDS ESR-E10.

4. Results and Discussion

After collecting data of three distribution networks of Punagam area pressure, flow and velocity have been computed using EPANET and by following the methodology described outputs by EPANET are obtained. Analysis of results has been carried out and error between computed results and actual results are compared for junction as well as pipe report of three distribution networks.

4.1 WDS ESR-E9

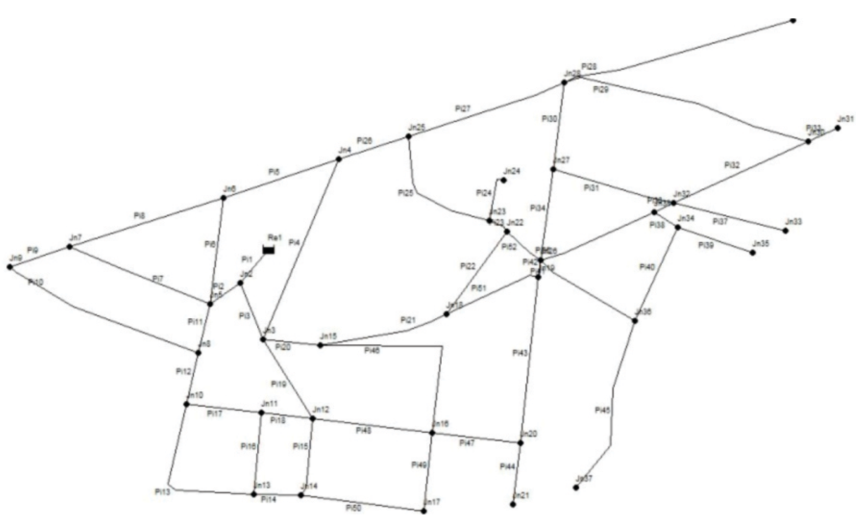

Figure 2 Network Daigram of WDS ESR-E9

4.1.1 Junction Report (WDS ESR-E9)

It includes 38 junctions. The result obtained using EPANET software for WDS ESR-E9 is calculated. The error between actual pressure and the pressure computed using EPANET software is also compared.

Following are some finding of above study:

- The pressure is computed using Hazen-William approach.
- For WDS ESR-E9 jn-2, jn-4, jn-5, jn-6, jn-7, jn-9, jn-12, jn-13, jn-14, jn-16, jn-23, jn-25, jn-26, jn-28, jn-29, jn-30, jn-31 junction gives negative pressure.
- There is fluctuation in the pressure head

4.1.2 Pipe Report (WDS ESR-E9)

Pipe report of WDS ESR-E9 includes 52 pipes. The result obtained using EPANET software for WDS ESR-E9 is presented. The error between actual flow and flow computed using EPANET software is compared. The error between actual head loss & head loss computed EPANET software is also compared.

Following are some of the findings of above study:

- The flow computed using EPANET is nearly equal to the actual flow.
- The velocity computed using EPANETis nearly equal to the actual velocity.
- The head loss computed using EPANETis nearly equal to the actual head loss.

4.2 WDS ESR-E9A

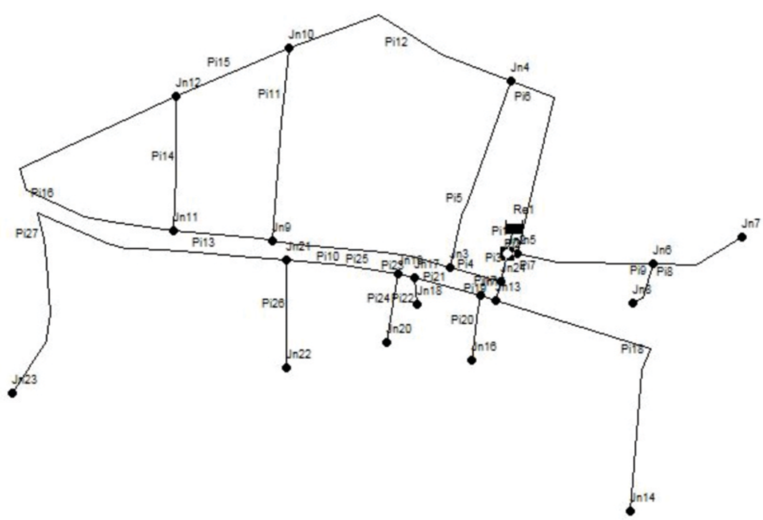

Figure 3 Network Daigram of WDS ESR-E9A

4.2.1 Junction Report (WDS ESR-E9A)

It includes 24 junctions. The result obtained using EPANET software for WDS ESR-E9A is calculated. The error between actual pressure and the pressure computed using EPANET software is also compared.

Following are some finding of above study:

Ÿ The pressure is computed using Hazen-William approach.

Ÿ For WDS ESR-E9A jn-5, jn-6, jn-7, jn-8, jn-10, jn-11, jn-15, jn-19, jn-23 junction gives negative pressure.

Ÿ There is fluctuation in the pressure head.

4.2.2 Pipe Report (WDS ESR-E9A)

Pipe report of WDS ESR-E9A includes 27 pipes. The result obtained using EPANET software for WDS ESR-E9A is presented. The error between actual flow and flow computed using EPANET software is compared. The error between actual head loss & head loss computed EPANET software is also compared.

Following are some of the findings of above study:

Ÿ The flow computed using EPANET is nearly equal to the actual flow.

Ÿ The velocity computed using EPANETis nearly equal to the actual velocity.

Ÿ The head loss computed using EPANETis nearly equal to the actual head loss.

4.3 WDS ESR-E10

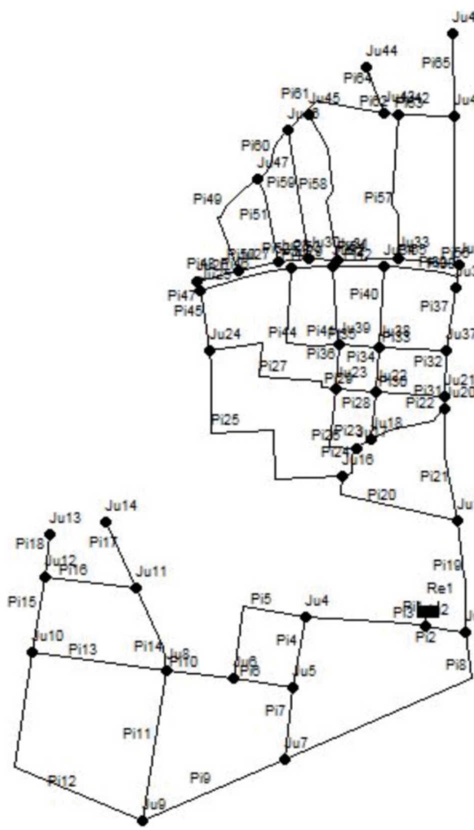

Figure 4 Network Daigram of WDS ESR-E10

4.3.1 Junction Report (WDS ESR-E10)

It includes 47 junctions. The result obtained using EPANET software for WDS ESR-E10 is calculated. The error between actual pressure and the pressure computed using EPANET software is also compared.

Following are some finding of above study:

- The pressure is computed using Hazen-William approach.
- For WDS-ESR-E10 jn-2, jn-3, jn-4, jn-5, jn-6, jn-7, jn-8, jn-9, jn-11, jn-12, jn-15, jn-17, jn-18, jn-20, jn-21, jn-22, jn-27, jn-30, jn-31, jn-32, jn-33, jn-34, jn-35, jn-36, jn-37, jn-41 junction gives negative pressure.
- There is fluctuation in the pressure head.

4.3.2 Pipe Report (WDS ESR-E10)

Pipe report of WDS ESR-E10 includes 65 pipes. The result obtained using EPANET software for WDS ESR-E10 is presented. The error between actual flow and flow computed using EPANET software is compared. The error between actual head loss & head loss computed EPANET software is also compared.

Following are some of the findings of above study:

- The flow computed using EPANET is nearly equal to the actual flow.
- The velocity computed using EPANETis nearly equal to the actual velocity.
- The head loss computed using EPANETis nearly equal to the actual head loss.

Conclusion

- The main focused of this study is to analyze the water distribution network and identify deficiencies (if any) in its analysis, implementation and its usage.
- At the end of the analysis it was found that the resulting pressures at all the junctions and the flows with their velocities at all pipes are adequate enough to provide water to the study area.
- It was observed that the pipes connected to the tanks as distribution pipes to the other pipes have smaller diameters.
- This study would help the water supply engineers in saving time as it this process is fast and less tedious
- Comparison of these results indicates that the simulated model seems to be reasonably close to actual network.
- Discharge should be increased to achieve the base demand.

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URBAN STORM WATER MANAGEMENT FOR STES CAMPUS

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Abstract

In many developed and developing countries, the celerity of urbanization is on its peak level. This rapid urbanization is a well set platform for many researchers, planners, builders & engineers to study the consequences of urbanization with the help of urban hydrology and hydrological sciences. Among all the ill-effects of urbanization, increase in impervious area; is the concept which attracts the researchers and planners, which causes flooding and reduction in ground water recharge. Urban storm water management is one of the methods: which is implied to control the effects of flooding due to excess rainfall on urban areas. In the present case, a storm water drainage system is to be designed for the campus of Sinhgad Technical Education Society, Vadgaon (Bk.) Pune. The area is rapidly developing and is having probability of more development in the future. The existing drainage system was designed during the period of 1990-1995 and exhibits poor maintenance and inappropriate execution during rainy season. Stagnant water and surplus flow over the terrain can be seen which affects the working of the institution. Thus there is a dire need of new and advanced drainage system for the proposed area. So, a new drainage system is to be designed, keeping in mind the lacunae's of the existing drainage system, which will be completely based on the present meteorological, hydrological and geographical conditions which are having extreme changes as compared to the previous design. StormCAD software is used as a designing tool, which helps in providing the most efficient drainage network for the present study. StormCAD is stand alone software which designs the network as per the requirements and results in the most efficient and appropriate drainage system design.

Introduction

Storm **water** is the water that runs off surfaces due to heavy rains across the land instead of seeping into the ground. In general, storm water is the rain that washes off driveways, parking lots, roads, yards, rooftops and other hard surfaces. The imperviousness of the surfaces is the main reason for storm runoff.

India is a developing nation with agricultural background. It is in its finest phase of urbanization and infrastructure development and is supposed to be highly developing in all the sectors. With a high pace of urbanization, the infrastructure here is also going to provide a platform for various engineers, researchers, builders etc. Water is the most important and central need for any developing and developed country. And a country like India, where agriculture is the major source of income of about more than half of its population, the available water is mostly used for this purpose.

Rainfall observed across the country is uneven and shows drastic variations in diverse conditions throughout the rainy season. The lone and key source of water at this point is from rainfall. About 60-70% of rain water flows of in the form of surface runoff, which can be collected and utilized for several different purposes, proving the surface run-off water as a major resource and not a waste. Urbanization is a main cause for increase in imperviousness of any region, which gives rise to surface runoff. Saving, disposing, collecting and employing the storm water results in human well being, protection from health hazards, flood control and proper functioning of the locality.

Effective management of storm water runoff is the basic necessity which will help in satisfying the above objectives. Absence of storm water management will result in pollution, sedimentation, local flooding and stream channel instability, declination in the level of ground water table. These effects can be reduced to some extent by having a well designed drainage network (also termed as storm sewer network). A properly designed storm drainage network will contribute in appropriate disposal of storm runoff, which will help in reducing the stagnant water bodies' formation in the locality due to inappropriate execution of drainage network.

Many of such above mentioned factors were duly observed during rainfall season in the locality of Sinhgad Technical Education Society campus, Vadgaon (Bk.). Surplus flow of storm water in the form of runoff, overflowing of some of the drainage channels, formation of stagnant water bodies were the prime observations during rainy seasons, which affects the working of the institute. So, it was decided to design a new and effective storm water drainage system, keeping in mind the lacunae's of the existing drainage system, which was showing improper execution.

The existing drainage system was designed during the period of 1990-1995, compared to which the present hydrological, geographical and meteorological conditions are drastically changed. So, a need of storm water management for the current situation is the necessity, which is considered in this study.

StormCAD which runs as a Stand-Alone application, which is flexible enough to work with any other application is used as a designing tool. It can analyze gradually varied flow, unlimited number of storm events, automated design for pipes and inlets, curved pipe inlets etc. and provides the most efficient drainage system design.

Literature survey

The Rational method is the oldest and widely used method for storm water drainage design which was first introduced by the Irish engineer Mulvaney (1850), the American Kuichling (1889),and the British Lloyd-Davis (1906). While the American Rational method uses the runoff coefficient according to rainfall characteristics and total land area. The British Lloyd-Davis formula is $Q=iA_{DCIA}$, instead of $Q=CiA$, where i is the rainfall intensity, A_{DCIA} is the drainage area as Directly Connected Impervious Area (DCIA), C is the runoff coefficient, A is the entire drainage area.

The runoff coefficient of the Rational Method is assumed to be directly proportional to the total imperviousness (Environmental 1983; Schueler 1944; Joint 1998). Schueler (1994) studied the consequences of imperviousness for some components of the urban water environment, such as runoff, stream shapes, water quality, stream warming, and stream biodiversity. He pointed out that transportation-related imperviousness often exerts a greater impact on the hydrology than the rooftop-related imperviousness. He also presented a relationship between urban land use and imperviousness (Schueler 1995).

The city of Olympia (1994) analyzed 11 residential, multifamily, and commercial sites to understand urban imperviousness. It was found that about 63-70% of the total impervious area consists of transportation related impervious surfaces, which are mainly roads, driveways, grasslands and parking lots.

Using accurate and large scale planimetric data, Prisloe et al. (2000) created a very accurate GIS database of impervious surfaces features that included buildings, roads, driveways, sidewalks and other constructed impervious landscape features in four towns of Connecticut. They compared the results with traditional land-use and/or zoning based imperviousness data, which were originally developed by analysis of satellite imagery for the same area in 1999 (Civco and Hurd 1999). They found two different relations between the actual imperviousness based on planimetreic data and the predicted imperviousness based satellite imagery. In residential areas, the predicted imperviousness of rural/suburban areas is about 2-3% larger than the actual imperviousness, but the predicted imperviousness of the urban area is about 6% smaller than the actual one.

Hoffman and Crawford (2001) developed a detailed GIS and database tools to predict flooding of individual parcels using the SWMM in the combined sewer system of Portland, Oreg. They developed very accurate coverage's of impervious surfaces that consists of single family residential (SFR) and commercial buildings and parking lots. In general, 80% of SFR and 100% of commercial buildings and parking lots are directly connected to the combined sewer system through laterals in their model. About 6% of SFR and 100% of streets are considered as DCIA, and runoff from those areas flows to storm inlets.

Storm CAD

StormCAD helps Civil Engineers to design and analyze the storm drainage network. It is an extremely powerful and easy to use program which is capable of importing data from other civil engineering software's/ programming tools. It is a comprehensive modeling tool for the design and analysis of storm sewer systems. It provides calculations for catchment runoff, gutters, inlets, junctions, pipe networks, and outfalls. It is having an intuitive interface which makes design and analysis of storm sewer systems simple. Most important benefits of StormCAD are: robust capacity, GVF analysis, analysis of unlimited number of storm events, automated design for pipes and inlets, curved pipe alignment etc.

Study Area

The study area is currently having an old design of drainage system. As the built up area is increasing with variation in rainfall, a new storm drainage system is required. The study area selected for the design of urban storm sewer is the Sinhgad Technical Education Society campus, Vadgaon (Bk), which is a part of southern Pune city. The area lies on the 18°27'57" N Latitude and 73°49'30" East Longitude. The land use mainly comprises of residential buildings, office buildings, roads, educational buildings, some commercial buildings etc. The land is having a natural slope towards the Vadgaon town, situated near Sinhgad Road. Rainfall pattern observed is as adequate rainfall for long duration & high intensity rainfall for short duration, which is mainly observed between June & September. Average annual rainfall is around 600mm-700mm & Temperature varies from 37°C-38°C during summer to 10° C to 11° C during winter.

Data Availability

The demand of data for the design of an urban storm drainage network is extensive. It consists of land use cover, rainfall data for a sufficient period (say 20 to 30 years) and topographic details. The land use cover of the area can be achieved by detailed study of the area. The rainfall data was made available from Indian Meteorological Department, Pune. The topographic details were made available by conducting a detail survey of the campus to know the elevations at every point, using Total Station.

Methodology and Results

For the present case, Storm CAD is being used for the design of storm drainage network. Circular concrete sewers are used for the entire design, as they provide more efficiency and durability as compared to other ones (like rectangular). The methodology is simply based on the hydraulic and hydrologic principles. Calculations are performed by analyzing the flow conditions, from inlets to final outlet (i.e. outfall). The road elevations and cross slopes are taken as per the survey conducted for this project.

The first step in any storm water drainage system design is to divide the entire area into sub-units. For each of this area, two important parameters required are: -

(I) Hydrological Parameters: - time of concentration of rainfall, rainfall intensity and most important rational coefficient (i.e. runoff coefficient C).

(ii) Hydraulic Parameters: - slope, area, drainage length, etc.

The rational coefficient C can be determined by comparing the land use pattern of the study area with the standard land use characteristics available in the various engineering hydrology literature, for which standard C values have been calculated. If the area includes different types of land use cover than the average value of C is being worked out by using the well known "Weighted Area Method". For Example: - If the area is having different land use patterns covering an area of A1, A2, A3……A_n with runoff coefficients C1, C2, C3……C_n, the average C for the area will be: -

$$C_{avg} = \frac{C_1 * A_1 + C_2 * A_2 + C_3 * A_3 + \dots\dots + C_n * A_n}{A_1 + A_2 + A_3 + \dots\dots + A_n}$$

The return period for storm drainage design usually varies between 2 to 5 years. For the present case, return period is taken as 3 years. Rainfall intensity i for each sub unit can be calculated from the rainfall data available from the Indian Meteorological Department, Pune. All these determined values are then used in the rational formula, i.e. Q=CiA to find the discharge from each of the sub unit. The pipe diameter is then determined by using the Manning's Equation which is:

$V=1/n *A*R^{2/3}*S^{1/2}$

where,

n = Manning's Roughness Coefficient for the pipe material

S = Slope of the pipe line

Q= Discharge in m^3/s and

A= Area of the pipe in m^2.

A paradigm network was analyzed for the study purpose, using StormCAD: -

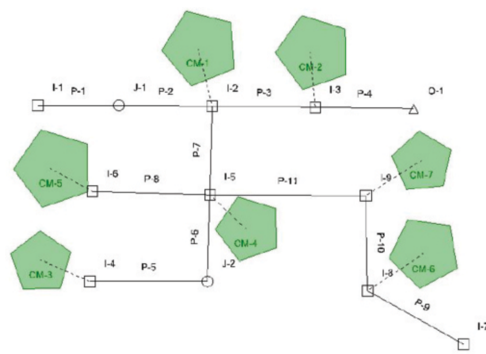

Fig 1: - Network Before Analysis

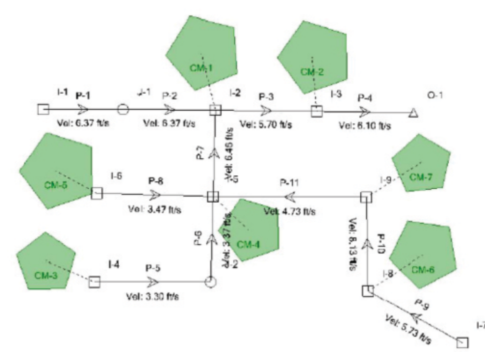

Fig 2: - Network After Analysis

It can be seen that the velocity through each of the conduit is worked out. With the help of above methodology and study the storm drainage network for the actual study area can be designed.

Conclusion

In the present study a new and efficient urban storm drainage network is to be design for a part of STES Campus. By comparing the land use pattern of the study area of 1990-1995 to the present situation of 2015, the impervious area is highly increased in the past 20-25 years. The selected area is rapidly urbanizing and will come out with many problems of storm water disposal in the future. Thus it is necessary to design a new and efficient storm water drainage system to take care of excess storm water to fulfill the present and future requirements. The design with the use of StormCAD, a designing tool, will help us in providing the most efficient design as per the present topographical, geographical and meteorological conditions. The required diameter of the sewers, as per the design range will be provided. Wherever necessary manholes, and junctions shall be provided. The value of rational coefficient should be worked out more accurately, to attain the best results. StormCAD can also provide the cost and economy study for the storm drainage network. If enough data is available, this technique can be applied to a larger part or for the entire city also.

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TURBULENCE OVER A ROUGH BED USING DOUBLE AVERAGED NAVIER-STOKES EQUATION

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Abstract

The paper describes an experimental study carried out in a laboratory flume to investigate the turbulence structure over cube mounted rough bed. The cubes were made of wood and were positioned at the channel bed with different relative spacing. The spacing was chosen such that roughness types under different spacing generally classified; isolated roughness flow, wake interference flow and skimming flow. The three-dimensional velocity field was measured by an Acoustic Doppler Velocimeter (ADV). The study particularly focuses on the changes induced in the spatially averaged mean velocity profile, turbulent intensity and Reynolds shear stress. The spatially averaged mean velocity profile show two distinct regions; the linear or exponential distribution below the roughness tops and logarithmic profile above the roughness top. Near the bottom, the form-induced stress changes significantly and then either decreasing or switching from making a negative to a positive with the roughness spacing.

Keywords: *Roughness; Velocity; Spacing; ADV; Double-average*

Introduction

The vast majority of environmental fluid flows are classified as hydraulically rough. This roughness may be both physical and biological. In essence this means that the influence of the bed roughness and local non-uniformity in the near-bed flow on the global flow characteristics cannot be ignored. These flows are typically described by the Reynolds averaged Navier-Stokes equations, which deal with time-averaged variables and involve no spatial averaging. Such an approach, however, is often inconvenient, due do the complex boundary conditions that lead to the high flow heterogeneity. The key drawbacks of the Reynolds averaged Navier-Stokes equations based approaches in relation to rough-bed flows have been discussed in Nikora et al. (2007a). It has been argued that to resolve the problem theoretically and conceptually, time averaging of the hydrodynamic equations should be supplemented by volume averaging or area averaging in the plane parallel to the mean bed surface. The double-averaging procedure gives new momentum for fluid, which are averaged in both time and space domains. Although the idea of spatial averaging has been used by many researchers (Smith and McLean, 1977; Wilson and Shaw 1977; Raupach et al. 1991 and others), the full development of this approach to use in hydraulics was fully made by Nikora et al. (2001, 2007a), who developed new continuity and momentum equations for rough-bed open channel flows by double-averaging the Navier-Stokes equations. The main advantages of this methodology, i.e. (a) self-consistency; (b) refined definitions for roughed flows such as flow uniformity, and the bed shear stress; (*c*) explicit accounting for the viscous drag, form drag and form-induced drag; e) scaling considerations and parameterizations based on double-averaged variables; and (f) partitioning of the roughness parameters and flow properties. These advantages use of the double averaged hydrodynamic equations helps in guiding and developing numerical models, designing laboratory field and experimental data analysis associated with rough-bed flows.

The spatial heterogeneity can contribute significantly to momentum transfer in the roughness layer it is not sufficient to consider Reynolds and viscous shearing alone; instead these should be complemented by the form-induced stress. In double-averaging terms, fluid stress therefore comprises three distinct components (Nikora et al. 2001):

$$\frac{\tau}{\rho} = \nu \left\langle \frac{\partial \bar{u}_i}{\partial x_j} \right\rangle - \left\langle \overline{u'_i u'_j} \right\rangle - \left\langle \tilde{u}_i \tilde{u}_j \right\rangle \qquad (1)$$

where $\nu \partial \langle \partial \bar{u} \rangle_i / \partial x_j$ is the viscous stress, which generally consider to be negligible, compared to spatially averaged Reynolds stress ($-\langle\overline{u'w'}\rangle$) in rough turbulent flows, except in the immediate vicinity of the wall where velocity gradients are large.

The main objective of the present study is to investigate the effect of the superposition of surface waves of different frequencies on the flow over artificial rough bed using spatial averaging method. At first the double-averaged Navier-Stokes equations are presented as theoretical frame work for the whole analysis. The deviations of double average mean velocity, turbulent intensity, Reynolds shear stress, form induced stress and total stress over rough bed are investigated. This study also addresses the effect on turbulence properties due change in relative roughness spacing in context of double averaging. The spacing is chosen such that the three roughness types, isolated roughness flow (L/h = 9), wake interference flow (L/h = 5) and skimming flow (L/h = 3) are investigated. The investigators are affirmative that the quantitative knowledge generated in the present study will be useful for numerical modeling or laboratory investigations, environmental hydraulics of rough bed flows, and in understanding sediment pickup, grain-sorting and transportation under coastal environment in the cubical rough bed dominated regions.

2. Experimental program and Data analysis

Experiments were conducted in a laboratory flume at the Fluid Mechanics and Hydraulic Laboratory (FMHL) of Indian Institute of Engineering Science and Technology (IIEST), Shibpur, India. A specially designed flat bed open-channel flume of dimension 18.3 m long, 0.90 m wide and 0.90 m deep with a constant slope of 0.00025 was used for the experiments. The flat surface of the flume was finished with plane net cement, which was somewhat frictionless. Water was re-circulated into the flume by a vertical turbine pump of 0.30 m/s^{3}capacity from a sump of dimension 30 m long, 3 m wide and 2 m deep. The discharge into the flume was controlled by a control valve. The discharge valve was gradually opened to achieve desired discharge and the tail gate was operated simultaneously to ensure the desired flow depth (H). The water flow after the discharge from pump passed through series of wire-meshes placed at the upstream end of the flume to ensure the smooth and vortex free uniform flow through the experimental channel.

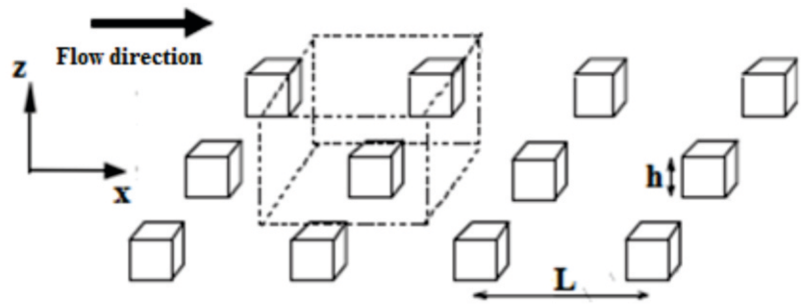

Figure 1. Cube arrangement in the experiment and calculation of domain

To perform experiments, the rough bed was established using large number of artificial water resistance wooden cube of dimension 0.025 m width × 0.025 m length × 0.025 m height. The cubes were polished to eliminate any excess material and to smoothen the exterior surface. The cubes were positioned along the flume (Fig. 1) using a special quality of adhesive (water resistance). Three different values of pitch length *L* (centre to centre spacing between two consecutive cubes) were chosen to achieve isolated roughness flow, wake interference flow and skimming flow (Perry et al. 1969). The cubical roughnesses were equally spaced along lateral and longitudinal direction so that the flow pattern repeats along the flume bed (Fig. 1). In this paper we consider the time-averaged flow variables do not change in time and the spatially-averaged flow variables change neither in the longitudinal direction nor in the lateral direction. Consequently, the spatially averaged bottom-normal and lateral velocities are equal to zero. Two different tests were carried out in the present study: (1) test over the flat surface, and (2) tests over the cubical roughness mounted on the flat surface at varying spacing. All experiments were conducted at location x = 12 m from the flume entrance where the flow was verified to be fully developed. The velocity data were collected at the rate of 40 Hz for 5 min duration for each measuring point with lowest point in each profile being 0.42 cm above the flat surface. Although it would have been preferable to sample even closer to the flat surface, initial test at closer distance gave noise with poor signal correlations. The increase in noise was likely due to echo effects from flat surfaced and high velocity gradients within the sampling volume (Lacey and Rennie 2012). In the present study, the raw ADV data were processed to remove these noises. These noises were removed by a phase space threshold despiking method described by Goring and Nikora (2002) and implemented in the Win-ADV software (Wahl 2000). To remove such effects, the velocity data collected by ADV were analysed systematically for all selected verticals. Assessment of the quality of individual time series velocity data is based on visual inspection of plotted raw velocity time series. The ADV velocity data were cleaned by removing all communication errors, low signal-to-noise ratio data (< 15 dB) and low correlation samples (< 70%). This was performed by Win-ADV software resulted in the removal of approximately 2% of all collected raw velocity time series. Such excluded signals were replaced by data using a cubic polynomial interpolation method. The effects of large noise were removed by minimizing the possible aliasing effect near the Nyquist frequency (herein f_n= 20 Hz). The highest measurement point from the bed was about 14 cm above the flume bed for each experiment.

The mean flow depth H was kept constant at 20 cm for all tests. All experiments were performed at single turbulent flow of flow rate (Q) = 0.053 m^3/s, at Reynolds number $(\mathrm{Re} = Uh/\mathrm{n})$ 58800, and Froude number $(Fr = U/\sqrt{gh})$ = 0.21, where U = 29.4 cm/s is the depth averaged velocity, v is the kinematic viscosity of water, and g is the acceleration due to gravity. All tests were fully rough turbulent flow of $h/\lambda > 70$, where λ $(=\upsilon/\ u)_S$ is the viscous length scale.

Table 1 Hydrodynamic Conditions

Reynolds number, $R_e(=Uh/\nu)$	58800
Depth average stream-wise velocity, *U* (cm/sec)	29.4
Mean flow depth, *H* (cm)	20
Froude number, $Fr(=U/\sqrt{gh})$	0.21
Friction velocity, $u_s\left(=\sqrt{\tau_0/\rho}\right)$ (cm/sec)	2.0 (*L/h* = 3), 2.1 (*L/h* = 5) & 1.5 (*L/h* = 9)

3. Results and Discussions

3.1 Experiment over the flat surface

In turbulent flow, the time-averaged mean velocities ($\overline{u},\overline{w}$), the root-mean-square velocity components ($\sigma u,\sigma w$) and the time-averaged Reynolds shear stress at each measuring point were computed by standard procedure (Nezu and Rodi 1986, Debnath et al 2012). For ease of comparison we have normalized the time averaged mean velocity, turbulent intensities and Reynolds shear stress by the friction velocity ($u*$) and are given by

$$\hat{u}=\overline{u}/u_*,\hat{w}=\overline{w}/u_*\ I_u=\sigma_u/u_*,I_w=\sigma_w/u_*\ \text{and}\ uw^+=\overline{u'w'}/u_*^2 \qquad (2)$$

The normalized mean velocity components $\hat{u}=\overline{u}/u_*,\hat{w}=\overline{w}/u_*$ stream-wise turbulence intensity (I_u), bottom-normal turbulence intensity (I_w) and Reynolds shear stress (uw^+) over the flat surface are plotted all together in Figs. 3(a-d). The friction velocity (u_* = 1.47 cm/sec) is calculated from the universal velocity logarithmic law (Schlichting 1960) over the plane bed surface as

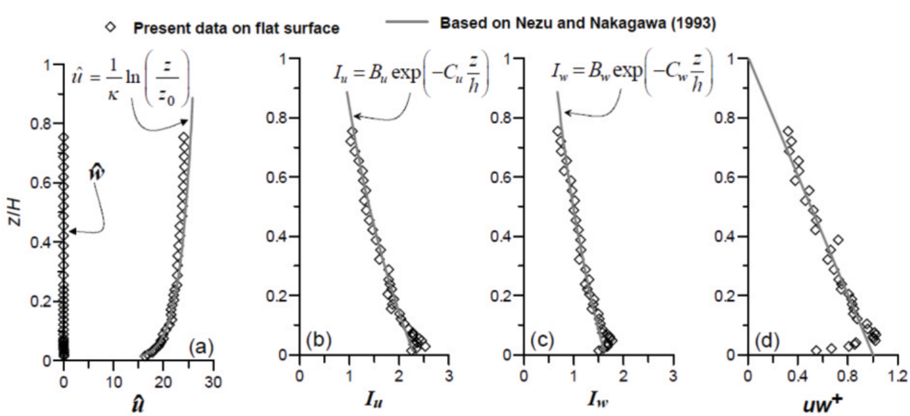

Figure 2. Normalized profile of (a) Stream-wise and bottom-normal mean velocities ($\hat{u}$ and $\hat{w}$), (b) Stream-wise turbulence intensity (uI), (c) Bottom-normal turbulence intensity (wI) (d) Reynolds shear stress (uw^{+}), based on present measurements on flat surface. Here the parameters B_u = 2.5, B_w = 1.61, C_u = 1.01 and C_w = 0.98.

$$\hat{u} = 1/\kappa \ln(z/z_0) \qquad (3)$$

where k (= 0.4) is the von Karman constant and z_0 (= 0.0014 cm) is the equivalent bed roughness with coefficient of regression R^2 0.96. It is also observed that the vertical and the lateral mean velocity components are zero throughout the depth of the flow (Fig. 2a). The Reynolds shear stress component (Fig. 2d) exhibits the same features as seen in the turbulent intensity profile. It observed that Reynolds shear stress increases and reaches a maximum value at a distance close to the bed and then decreases towards the free surface. The results on flat surface are in good agreement with Nezu and Rodi (1986) and Nezu and Nakagawa (1993).

3.2 Experiment over cube mounted bed

3.2.1 Mean velocity

Our results support a composite profile consisting of two theoretical distributions: the linear or exponential distribution below the roughness tops (z), and logarithmic law above the roughness top. The Fig. 3 displays the double-averaged velocity profile $\langle\hat{u}\rangle = (\langle\overline{u}\rangle / u_s)$ normalized by spatially averaged friction velocity $\left(= \sqrt{\langle\tau_0\rangle/\rho}\right)$ against normalized vertical distance (z-d)/h, where d is the zero-plane displacement. The methodology adopted for estimation of the zero plane displacement is shown later. The value of u_s are determined from the spatially averaged bottom shear stress(τ_0). The spatially averaged bottom shear stress is determined by extrapolating the spatially averaged Reynolds shear stress profile up to the roughness top.

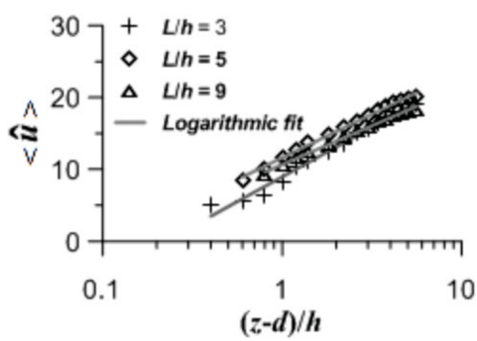

Figure3. Logarithmic representation of above roughness top against (z-d)/h for (a) L/h = 3, (b) L/h = 5 and (c) L/h = 9

Figs.3a-c presents results for relative roughness spacing (L/h) = 3, 5 and 9 respectively. The vertical distribution of ˆ***u*áñ** for flow over cubic roughness follow the logarithmic behavior above roughness tops with coefficient of regression R^2 0.95 for all the relative spacing. This result seems to confirm the findings of Coleman et al. (2007).

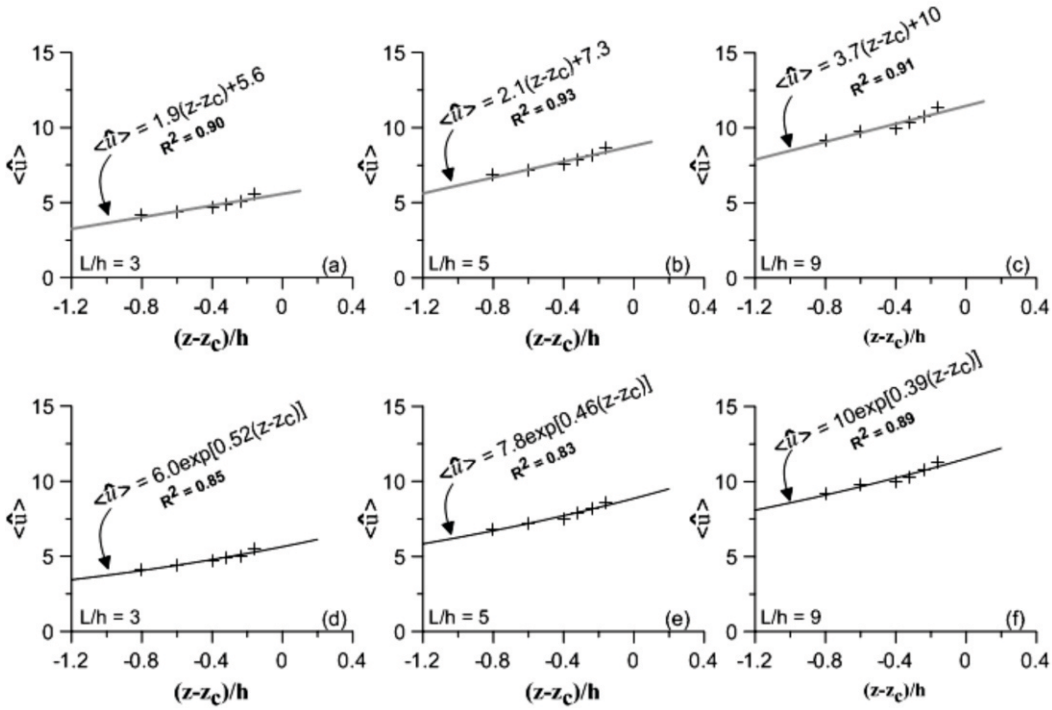

Figure 4. Linear (a-c) and exponential (d-f) double average velocity distribution below the roughness top for relative roughness spacing's L/h = 3, L/h = 5 and L/h = 9.

Fig. 4 shows the linear and exponential distributions below the roughness top for different relative roughness spacing (L/h = 3, 5 and 9). For comparison of model suitability, both plots (linear and exponential) included the same number of data points up to 5 mm above the roughness tops ($0 < (z\text{-}z)/h$ 0.2).

Below the roughness layer or roughness crest (z) the linear distribution described the data (coefficient of regression $((R^2)_{L/h\,=\,3,\,5\text{ and }9}$ 0.90, 0.93 and 0.91) better than the exponential distribution $((R^3)_{L/h\,=\,3,\,5\text{ and }9}$ 0.85, 0.83 and 0.89). Nikora et al. (2002) propose that the double averaged velocity profiles can take several forms (constant, exponential, linear or a combination of these) dependent on flow conditions and roughness geometry.

Coleman et al. (2006) found linear profile and linear to slightly exponential form for fluvial sand waves of $L/h = 20$ and $L/h = 10$ respectively. Coleman et al. (2007) found an exponential profile (below the roughness crest) for the closely spaced ribs ($L/h < 10$), and a linear profile for the widely spaced ribs (L/h 10), with the profile transitioning between the two for increasing L/h. The present results shows good fit to linear profile below the roughness tops compared to exponential for all the three spacing. This is in contradictory to the observation by Bose and Dey (2007); they reported a second-degree polynomial distribution below the roughness top. However the present results are in conformity with Raupach et al. (1991) with R^2 0.85 for closer or dense roughness spacing.

3.2.2 Zero-plane displacement height (d)

In order to find the zero-plane displacement height we used the method suggested by Nikora et al. (2002). They calculated d for various types of roughness and flow conditions and the results showed that the position of the zero-plane goes up with increasing roughness density. We can write the law of wall for flow field in its general form with zero-plane displacement (d) as follow:

$$\frac{\langle \overline{u} \rangle}{u_s} = \frac{1}{k} \ln \frac{z - d}{z_0} \tag{4}$$

where ($\overline{u}$) the spatially averaged velocity, k is the von Karman constant, Z_0 is the roughness length, and u_s $\left(= \sqrt{\langle \tau_0 \rangle / \rho}\right)$ is the friction velocity.

The zero-plane position is evaluated by the method based on the inverse of the velocity profile derivative suggested in Nikora et al. (2002). Differentiating equation (4) with respect to z we get

$$\frac{d\langle \overline{u} \rangle}{dz} = \frac{u_s}{k\left(z - d\right)} \tag{5}$$

After rearranging equation (5), we can write

$$\left(\frac{d\langle \overline{u} \rangle}{dz}\right)^{-1} = \frac{k}{u_s} z - \frac{k}{u_s} d = a_1 z - b_1 \tag{6}$$

where a_1 and b_1 are the coefficients of linear fit to the inverse velocity gradient. Then, the displacement height is evaluated as $d = b_1/a_1$ and it is adjusted to give $k = 0.4$

The left hand side of the equation (6) represents the ratio of the mixing length l_e to the friction velocity u_s i.e., $\left(d\langle\overline{u}\rangle/dz\right)^{-1} = l_e(z)/u_s$ Therefore, plots of spatially averaged inverse velocity gradient may be interpreted as vertical distributions of the mixing length (l_e) normalized with shear velocity. The distributions of double average velocities were smoothed by using a two-point running mean in the vertical before calculations of velocity derivatives. Fig. 5 provides the plot of $\left(d\langle\overline{u}\rangle/dz\right)^{-1}$ for different L/h against z.

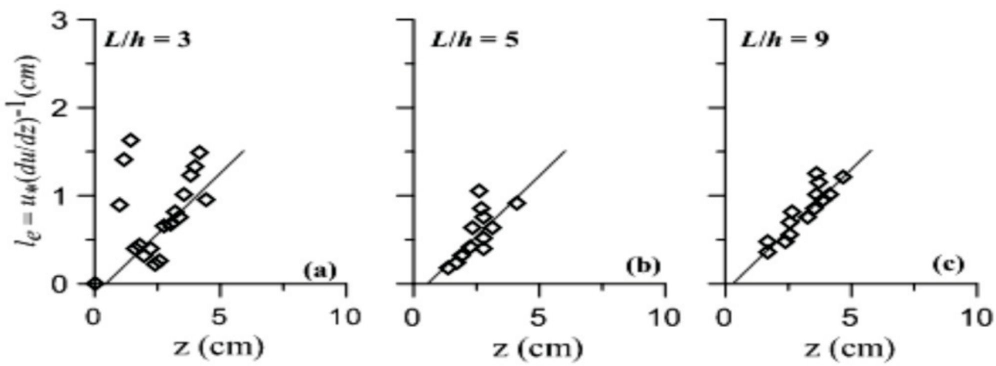

Figure 5. Determination of the zero-plane height d (using Eq. 6) for (a) $L/h = 3$, (b) $L/h = 5$ and (c) $L/h = 9$. The solid line represents the linear fit to the straight line part of the profile.

The variations in zero-plane displacement (d) with L/h for only current and combined current and wave with frequency $f = 1$Hz and $f = 2$ Hz is shown in Fig.6. The value of d decreases with increasing relative roughness spacing (L/h) which is consistent with the findings of Coleman et al (2007) for $L/h = 3$ and 5. However the present results of d differ from the findings of Coleman et al. (2007) for $L/h > 5$. This is probably because the positions of d depend on roughness density, geometry and energy of large eddies (Manes et al. 2007).

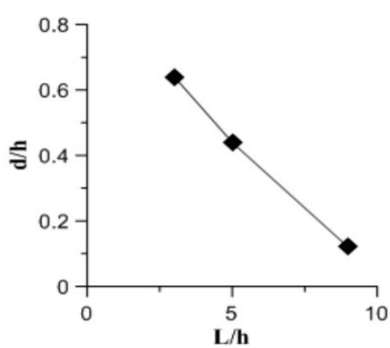

Figure 6. Variation of non-dimensional zero-plane displacement d/h against L/h (using Eq. 6)

3.2.3 Spatially averaged total shear stresses

The spatially averaged total shear stress was obtained by same procedure as discuss in Manes et al. (2007) i.e., by the addition of the spatially averaged Reynolds shear stress $\langle \overline{u'w'} \rangle$, the form-induced shear-stresses $\langle \tilde{u}\tilde{w} \rangle$, and the viscous shear stresses $\upsilon\partial\langle \overline{u} \rangle/\partial z$ The spatially averaged total shear stres $\langle \tau_t \rangle$ is balanced by the gravity and has to have a linear profile (Mannes et al. 2007). Therefore to check the linearity of the total shear stress we have performed the analysis above and below the roughness top. All vertical distribution of shear stress components (τ) $(=\langle \tau_t \rangle, \langle \overline{u'w'} \rangle, \langle \tilde{u}\tilde{w} \rangle$, and $\upsilon\partial\langle \overline{u} \rangle/\partial z)$ are plotted in Fig. 13 against z/h for relative spacing $L/h = 3$ (Fig.7a), $L/h = 5$ (Fig.7b) and $L/h = 9$ (Fig.7c).

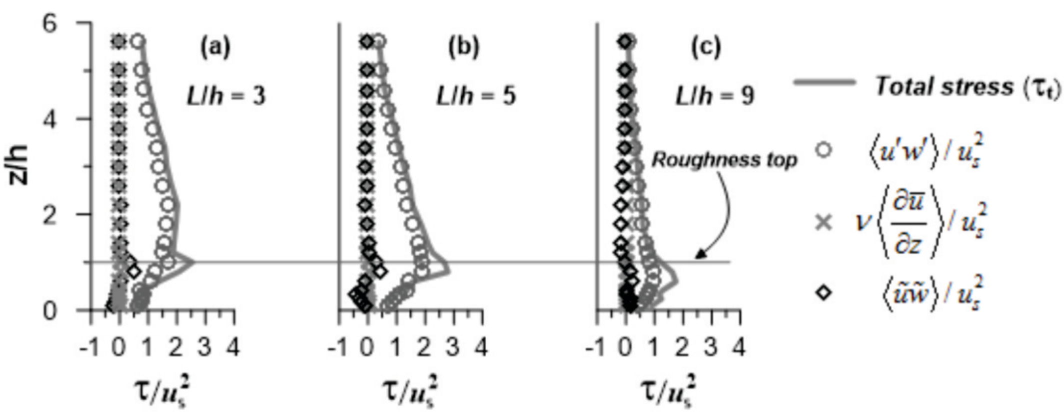

Figure 7. Normalized vertical distribution of shear stress components against z/h for (a) $L/h = 3$, (b) $L/h = 5$ and (c) $L/h = 9$

In all experiments the spatially-averaged Reynolds shear stress profile is linear above the roughness layer ($z/h > 1.2$) as observed for time averaged profiles over plane bed (Nezu and Nakagawa 1993). Fig. 7 shows that the double averaged Reynolds shear stress is the main dominating flow stress throughout the flow depth except below the roughness top probably due to decrease in turbulence level below the roughness top. The changes in peak position of spatially averaged

Reynolds shear stress strongly depend on relative roughness spacing. The form-induced stress arises from the correlations between point-to-point spatial deviations in time-averaged velocity components, the form-induced velocities $\tilde{u}$ and $\tilde{w}$. It therefore depends on both the spatial coherence and magnitude of spatial variance in the time-averaged flow. Form-induced stresses are essentially zero above the roughness layer, and increase to a positive peak below the roughness top (z/h = 0.8), this peak decreasing for increasing roughness spacing. Below the roughness top where the mean flow is highly non-homogeneous, the decrease of $\overline{u'w'}$ is compensated for by the appearance of $\langle \tilde{u}\tilde{w} \rangle$ and by the viscous shear stress $\upsilon \partial \langle \overline{u} \rangle / \partial z$ which have approximately same magnitude above the roughness tops for all the conditions. Below the roughness top the $\langle \tilde{u}\tilde{w} \rangle$ and $\overline{u'w'}$ have comparable magnitude, whereas the viscous stress becomes negligible throughout depth for all conditions. Above the roughness top, the spatially averaged total shear stress $\langle \tau_t \rangle$ reasonably follows the expected linear shear stress profile.

Conclusions

The purpose of the present study was to ascertain the effect of double-averaged parameters on the flow over cubical roughness with different relative spacing. An ADV was used to collect the velocity data over a single roughness wavelength. Three different values of pitch length (L) were use to achieve isolated roughness flow, wake interference flow and skimming flow. The cubical roughnesses were equally spaced along lateral and longitudinal direction so that the flow pattern repeats along the flume bed and it was assumed that, all averaged quantities do not vary in the flow direction. The effect of roughness spacing on the spatially averaged mean velocities and total stress containing Reynolds shear stress, form-induced stress and viscous stress in an open channel flow has been presented in this paper as a function of relative roughness spacing. The finding reveals that stream-wise double averaged profiles were support a composite profile consisting of two theoretical distributions: the linear or exponential distribution below the roughness tops and logarithmic law above the roughness top. The form-induced stress which is calculated from spatially averaged velocity components. Near the bottom the form-induced stress changes significantly and then either decreasing or switching from making a negative to a positive with the roughness spacing. Above the roughness top, the spatially averaged total shear stress reasonably follows the expected linear shear stress profile for all the experimental conditions.

This study could provide a better understanding of turbulence over a cubical roughness with different relative spacing. It is of future interest to improve our understanding of turbulence with widely varying roughness spacing. Detailed investigations are important to formulate a better modeling of three-dimensional flow structure which will help the researcher to understand the mean flow and turbulent statistics in the field of hydraulic.

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MORPHOLOGICAL STUDY OF RIVER MANDAKINI USING GEOSPATIAL TOOLS

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Abstract

Continuous heavy rains from 16^{th} to 17^{th} June, 2013 triggered flooding of Mandakini River in Kedarnath of Rudraprayag district (Uttarakhand) which caused huge devastation to the settlement. Since large changes in the geomorphology of river had taken place, hence there was a need to identify the critical locations where there are major changes in the morphology. The present study highlights the geomorphological study of River Mandakini from Kedarnath to Rudraprayag. An integrated approach based on remote sensing and GIS was employed using LISS IV, III and Landsat data procured from NRSC and downloaded from USGS websites respectively. The morphology of the river was extracted by applying NDWI and image enhancement techniques. Using the Cartosat/LISS-III/LIS-IV data, at every km of the river, lateral shift of right and left bank, bed width of the river and bed elevation have been determined. The study deals with the quantitative analysis of lateral shifting of bank and erosion- deposition pattern for pre and post 2013 flood at Kedarnath using satellite data of Nov, 2011 and Sep, 2013, respectively. Analysis of the lateral shifting of river indicates that about 5.75 km^2 area has been eroded.A relationship between the river bed slope and bedwidth of river has been developed which indicates that the bedwidth of the river varies inversely to the bed slope. The study shall be useful for planning the river protection work, area to be inhabited, fixing optimum width of the river etc.

Keywords:*Geomorphology, Remote sensing, GIS, NDWI, Channel shift, Erosion and Deposition.*

Introduction

Indian rivers have dynamic geomorphology features, especially during heavy rains. Geomorphologists analyse the origin and systematic description of landforms, and can offer the impact of human activities on those landforms. Interest in hilly rivers and associated ecosystems is not restricted to geomorphologists. Mountain environments are significant to society as a whole, as they offer a wealth of mineral assets, represent the source for major river systems that provide water for consumptive utilization and transportation, offer a vast range of recreational opportunities, and can serve as indicators of a changing climate.

For instance, most mountain rivers steep channel slopes and highly resistant river bank which generally result in high turbulent flow. "River morphology and related aquatic ecosystems in mountain streams are significantly inclined by tectonic activity, glaciation and lithology which results in high spatial variability. Mountain Rivers change their nature at vary spatial and temporal level as compared to rivers in general and thus geomorphology study of mountain streams is both important and noteworthy (Wohl 2000).

River bank erosion is a natural geomorphic process which happens in all stream as modifications of channel size and shape are made to carry the discharge and sediment provided from the drainage basin. The sediments deposited and eroded in the river have a tremendous effect on river cross sectional area, gradient, intensity of water flow and its discharge. Therefore, due to geomorphological change, there is overflow in river which causes flood in the neighbourhoods. With the remote sensing- GIS integrated approach, geomorphic mapping of the river for the pre and post monsoon images can be easily done. Data supplied by the optical and radar satellites can be employed to invoke maps of geomorphological changes and flood inundation in a short period of time which are cost effective. Radar images can be used in all type of weather conditions as they can penetrate clouds, they are quite beneficial in mapping flood and are ideal for flood monitoring, especially in complex hydraulic conditions.

Strahler in 1964 stated that geomorphometric study of a basin offers the quantitative description of the drainage basin and plays a very important role in understanding the basin characteristics. Bali et al. (2012) stated that morphometric assessment helps in predicting the approximate behavior of the watershed, if appropriately combined with geology and geomorphology.

Clarke in 1996 stated that watershed development depends on various factors such as fluvial geomorphology, surrounding area geology and structural component, type of soil and their characteristics and on vegetation cover through which the river flows. He termed geo-morphometry as "measurement and mathematical analysis of earth's surface." Recently, the risk of natural disasters has increased in the area as a result of increasing anthropogenic activities. This trend is likely to increase in future as the activities like pilgrimage, tourism, etc. will increase. The natural flow paths of the channels get obstructed due to the construction of man-made structures that results in deviation of the flow from its natural course. Apprehending the tendency of increasing urbanization due to increase in the number of pilgrims, tourists and other developmental activities in the area, selection of safe land-use locations would be a formidable task to accomplish.

The continuous and heavy rains on 16th and 17th June, 2013 triggered flooding of Mandakini River in Kedarnath of Rudraprayag district of Uttarakhand, caused huge devastation of vegetation as well as surrounding habitat. There was large amount of changes in the geomorphology of the river which needs to be studied and checks all the critical locations where there is a drastic amount of migration. The objectives of this paper to estimate lateral shifting of the banks of the river and also erosion area along the length of the river from Rudraprayag to Kedarnath. Finally identify critical and vulnerable reaches- locations where shifting is maximum.

Normalized Difference Water Index (NDWI)

NDWI shows the moisture content in vegetation and soil. The spectral reflectance of surface water bodies differ from land surfaces. In NIR band, surface water has low reflectance as compared to land surface due to strong absorption. NDWI is useful to assess water content in a normalized way.

$$\mathrm{NDWI} = \frac{\mathrm{GREEN} - \mathrm{NIR}}{\mathrm{GREEN} + \mathrm{NIR}} \qquad (1)$$

The NDWI result is dimensionless and varies between -1 to +1, depending on the leaf water content. Positive NDWI values indicate water. The index increases with moisture content in dry soil to surface water.

Study Area

The reach of Mandakini River from Kedarnath to Rudraprayag has been considered for the assessment of geomorphological change (Fig. 1). The study area lies in between 30° 44' 4.81” N to 30°17' 3.89” N latitudes and 79 ° 40' 82” E to 78° 58' 52.11” E longitudes from Kedarnath to Rudraprayag in the Mandakini catchment of Rudraprayag district situated at Uttarakhand state. It is located in the snow cover area of Himalayan region at the height of approx. 3,583 m above sea level. Mandakini River originates from the Chorabari Glacier (3895 m) near Chorabari Lake and joins Saraswati River which originates from companion glacier at Kedarnath, passing through Rambara and Gaurikund and joins Alaknanda at Rudraprayag. Kedarnath is one of the ancient and famouspilgrims place situated in Uttarakhand, India.

Figure 1 Location Map of Study Area

Data Acquisition

For the present study Survey of India Toposheets has been used, viz. 53 J/14, 53J/15, 53N/2 and 53N/3. Table 1 shows the satellite data used for fulfilling present study objectives.

Table 1 Satellite data used

Data sets	Source	Year	Resolution
Resourcesat -1 : LISS III	http://bhuvan.nrsc.gov.in/	2011	23.5 m
Resourcesat -2 : LISS IV	http://bhuvan.nrsc.gov.in/	2011,2013	5.8 m
Cartosat-1 DEM	http://bhuvan.nrsc.gov.in/	2008-2012	10 m
Landsat ETM+	http://earthexplorer.usgs.gov/	2014	30 m
IKONOS data	Google Earth	2014,2010	4 m

Methodology

To accomplish the objectives of the present study, the methodology is broadly divided in three steps as given below:

a. Visual interpretation of satellite data
b. Stream profile analysis
c. Image processing of satellite data

1.1Pre-processing of input image

Geo-referencing of satellite data is necessary before initiating the analysis. For this firstly we manage all the toposheet which come under the study area and simultaneously did geo referencing of all the toposheets using ERDAS 14, followed by mosaicing and clipping out all the toposheets for acquiring the desired study area. Feather algorithm has been used for mosaicing to get the unified boundaries between different images. The toposheets are precisely ortho-rectified with the following parameters:

Projection Type: Universal Transverse Mercator (UTM)
Spheroid Name: WGS 84
Datum Name: WGS 84
Zone: 44

Now, this toposheet data is taken as a reference for georeferencing the satellite data individually. For each image 12 GCPs (ground control points) have been taken and georeferencing has been carried out using third-order degree polynomial with nearest neighbourhood resampling technique. RMS (root mean square) error has been kept below 0.5 pixel size.

Various non-linear enhancement techniques, such as histogram equalization and Gaussian stretch on satellite data have been used accordingly for the proper visualization of data. Histogram equalization will increase the contrast in the heavily populated range of the histogram while reduce the contrast at the sparsely populated image of the histogram while Gaussian stretch increase contrast at the tail ends of the histogram.

NDWI (normalized difference water index) image for data has been prepared by making a model using model maker in ERDAS 14. The model has been prepared using Eq.(1).

1.2Delineation of river bank line.

The right and left river bank lines have been identified and delineated for the satellite image by creating shape file with feature types as polyline, having same projection as that of satellite data in Arc catalog box in Arc GIS 10. The areas with recent soil deposits have higher moisture compared to

other areas adjacent to river bank.
This shows that either the river was flowing through that area in the recent past, or that area was submerged in water, when there was high flow in the river. These areas are also taken as a part of river. Thus NDWI image was very useful in identifying the bank line. The length of both right and left banks of river for both the years 2011 and 2013 respectively have been computed using Arc map. We have taken right bank of year 2011 as a reference baseline for identifying the eroded and deposited area. As the river from Kedarnath to Rudraprayag along the bank line is of approximately 75 km, so the baseline was divided into 15 chainage at an interval of 5km. each. The lateral distance is calculated at each chainage individually for right and left bank of river. A graph has been plotted between lateral distance v/s chainage, where Rudraprayag is taken as source and Kedarnath as end point, showing the lateral shifting of the bank lines of the river from 2011 to 2013 depicting the post effect of flood.

The areas which are now under water but initially covered by land are represented as eroded areas and marked by a shapefile with feature type as polygon. And the areas changed from water to land are marked as deposited areas. For each reach, erosion and deposition area have been computed through area estimation with the shifting bank-lines in study period. Graphs have been plotted to show the total erosion and deposition occurred individually for right and left bank as well in totality for a river.

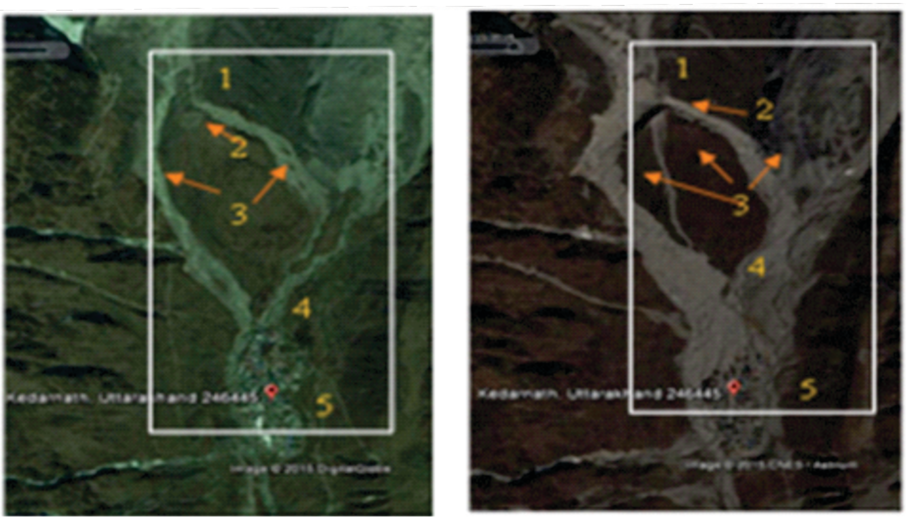
1.3Stream Profile Analysis

CARTOSATDEM (Digital elevation model) of study area for year 2008-2012 procured from NRSC website http://bhuvan.nrsc.gov.in is not projected globally. So firstly the DEM procured were projected to UTM projection, Zone 44, using Project Raster under data management in Arc Map 10. A mosaic of DEM has been prepared to cover the study area and through Extract by mask (Spatial Analyst), the study area was extracted. It extracts the cells of a raster that correspond to the areas defined by a mask.

With the help of 3D Analyst tool using interpolate line and profile graph, the relationship between chainage and elevation has been computed. Through this relationship, relation between width of the river and slope has also been computed.

Results and Discussions

From Google Earth imageries, through image interpretation we analyzed the changes in morphology of Mandakini River near Kedarnath. The processed images of year 2011 and 2013 are shown in Figure 2a and 2b.



Nov, 2011 Kedarnath Sep, 2013 Kedarnath

Figure 2 Google Earth imageries of Kedarnath area of year 2011 and 2013.

From the above figures, one can recognize five locations where change can be identified as mentioned below:

Location [1]: It is observed that in the year 2011, water comes from glacier in a single stream while in 2013, post flood, the channel has now become broader.

Location [2]: Large amount of debris which lies on the path was carried away by the flow of water after the flood of year 2013.

Location [3]: Water which was initially moving in two streams is now moving in three streams i.e. an overall a new channel is formed.

Location [4]: Water used to move in a thin channel while post flood large amount of water has moved into the area from all directions

Location [5]: Kedarnath settlement was completely devastated.

Satellite Data Analysis.

The output of the NDWI is shown in Figure 3. After the application of NDWI on the image, the river is clearly identified and can now be easily extracted from the image.

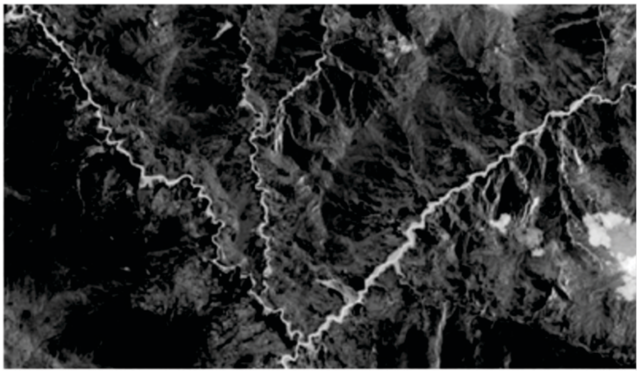
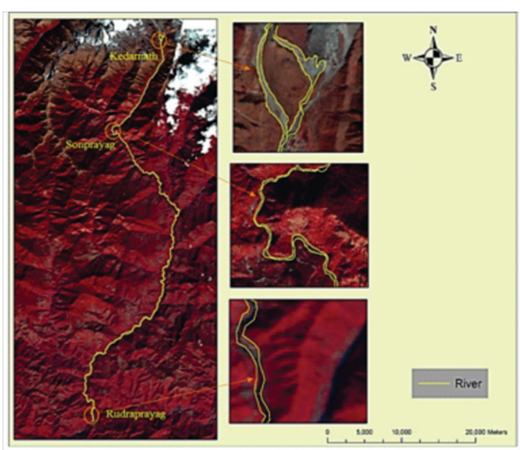


Figure 3 NDWI Output

Figures 4 and 5 shows delineated Mandakini River 0f year 2011 and 2013 after the preprocessing of the satellite data.

By integrating the river bank lines of year 2011 and 2013, it is observed that Mandakini River has shifted its bank line extremely causing severe damage to vegetation and surrounded habitat. There is also a formation of new stream just above the Kedarnath. Some parts of the roads have been washed away by water.

As the river is shifted from its baseline, we have calculated the quantitative amount of lateral shift with respect to right as well as left bank with base year 2011. The river is divided into 75 chainage as shown in Figure 6, and at each chainage lateral shift is being computed.



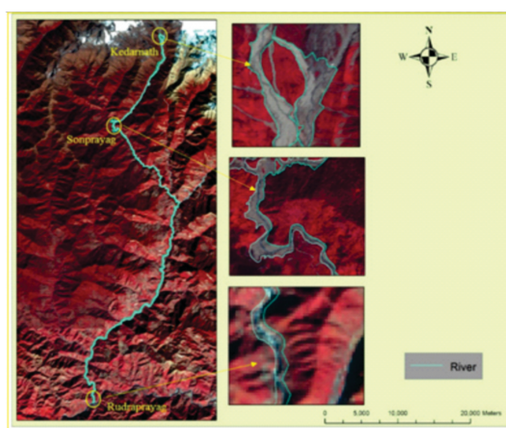


Figure 4 Delineated 2011 Mandakini River**Figure 5** Delineated 2013 Mandakini River

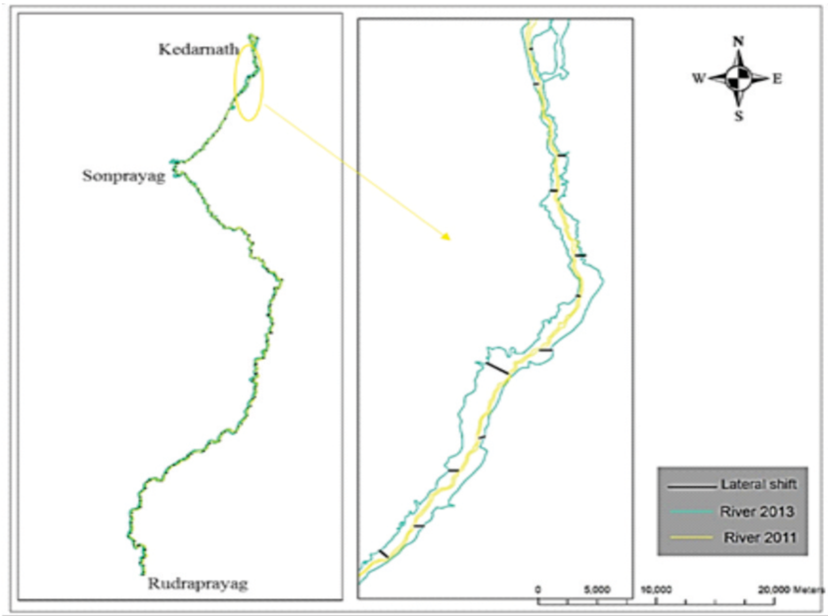


Figure 6 Lateral shifting of right and left banks.

The shifting configuration of right bank of year 2013 with reference to base year 2011 is graphically represented in Figure7. The positive values indicates shifting of right bank in outward direction of 2011 right bank, while negative values show shifting in inward direction. It is observed that mostly the river is shifted outwardly and maximum shift is in 12th reach of approx. 358 m.

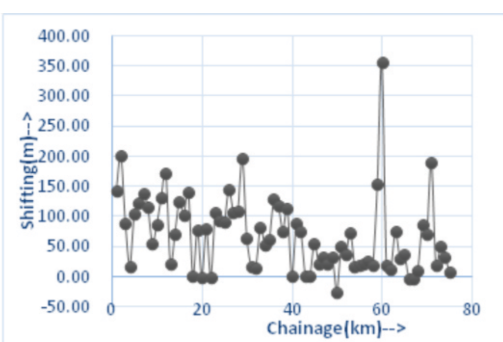


Figure 7 Graphical representation of right bank shift at each chainage

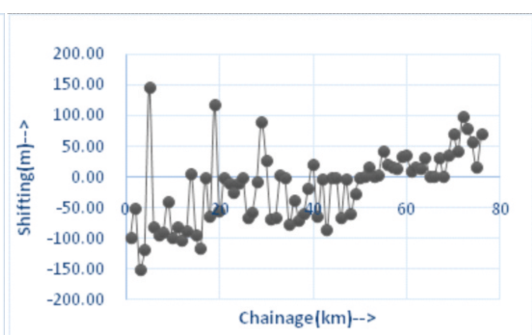


Figure 8 Graphical representation of left bank shift at each chainage

The shifting configuration of left bank of year 2013 with reference to base year 2011 is graphically represented in Figure 8. The positive values indicates that there shifting of left bank in outward direction of 2011 right bank, while negative values show shifting in inward direction. It is observed that initially from Rudraprayag there is no fixed direction of shifting but as it moves upstream shifting becomes constant in outward direction i.e. the channel has widen itself . At fifth chainage, there is a maximum shift of 150 m.

Figure 9 shows the erosion and deposition in the Mandakini River. It is observed that bank erosion is more prominent in the right bank. There is hardly any deposition in right bank while there is deposition in left bank especially near Rudraprayag.

Figures 10 and 11 depict graphical representation of reach wise total erosion/deposition in both banks respectively of the river.

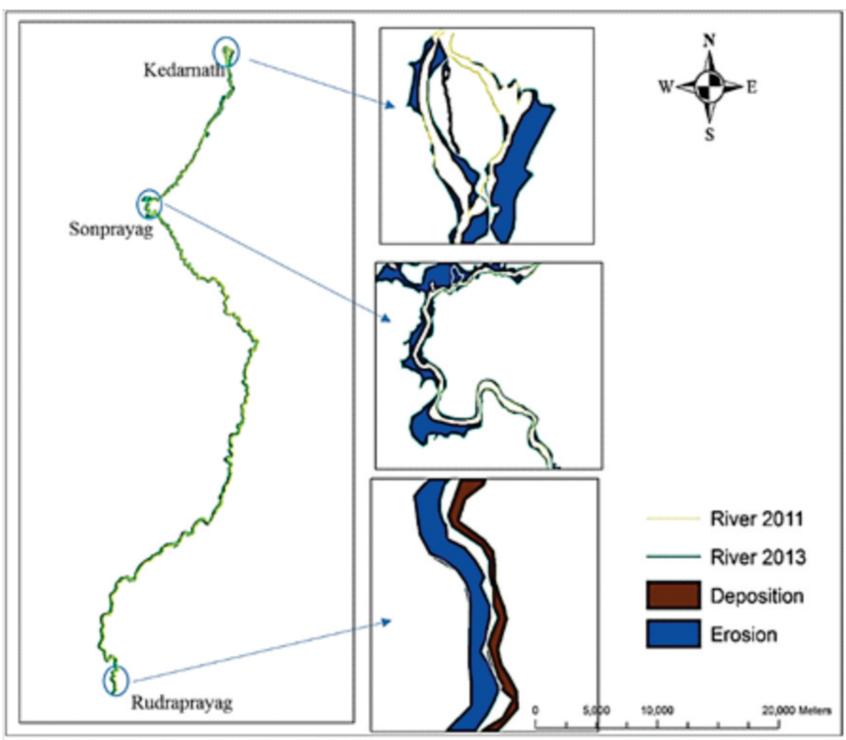


Figure 9 Erosion and deposition of Mandakini River

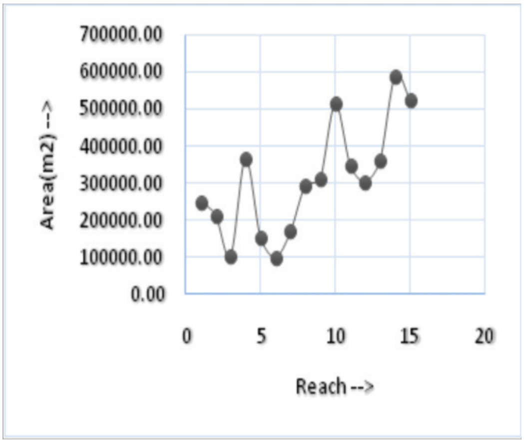


Figure 10 Erosion and Deposition within right bank

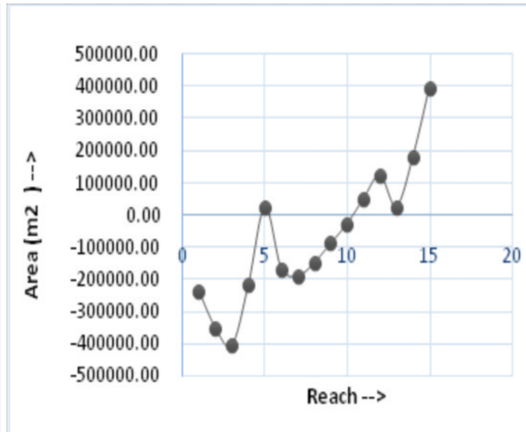


Figure 11 Erosion and Deposition within left bank

From the graphical representation, it is observed that there is hardly any deposition while bank erosion is prominent over whole right bank. Kedarnath's reach exhibits huge erosion in both the banks. Near downstream of the river that is Rudraprayag and surrounding area, the whole river is migrated towards right and thus there is great amount of sediment deposited here.

The total bank erosion and deposition occurred during 2011- 2013 in Mandakini river from Kedarnath to Rudraprayag come out to be:

Total eroded area:	Total deposited area:
Right bank: 4.63 km^2.	Right bank: 0.03 km^2.
Left bank: 1.10 km^2.	Left bank: 2.11 km^2
New channel: 9562.53 m^2.	Total: 2.14 km^2.
Total: 5.75 km^2.	

Stream Profile Analysis

Figure 12 shows graphical representation between elevation and chainage. It is observed that from Rudraprayag to Kedarnath, as we go upstream, the elevation with respect to sea level increases and thus increases the slope which is shown in Figure 13.

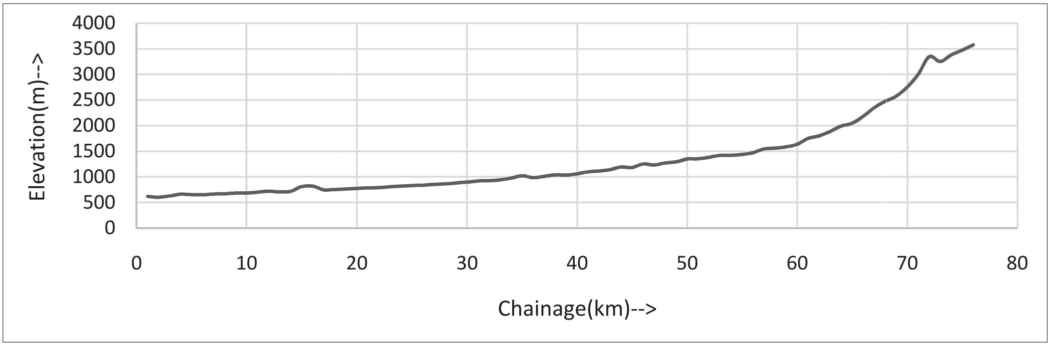


Figure 12 Variation of bed level with chainage

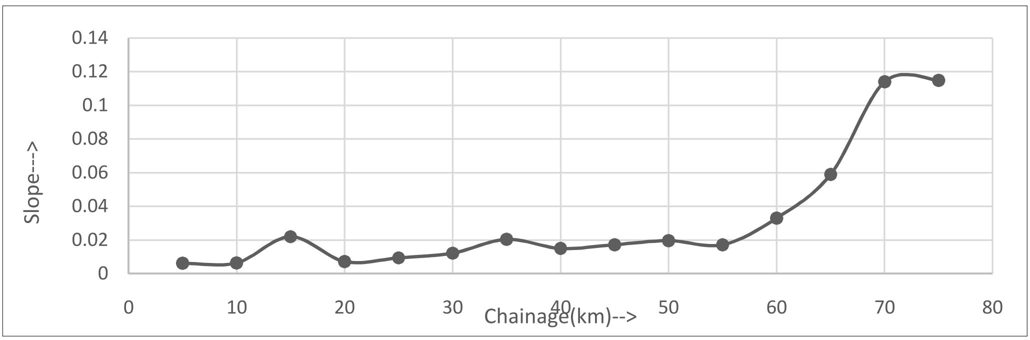


Figure 13 Variation of bed slope of the river with chainage

As slope and bandwidth of the river are function of discharge Q. And here discharge will remain constant from Kedarnath to Rudraprayag as there is no other source of water except River Mandakini, so width will be dependent upon slope. As we move from Kedarnath to Rudraprayag width increases which is shown in Figure 14.

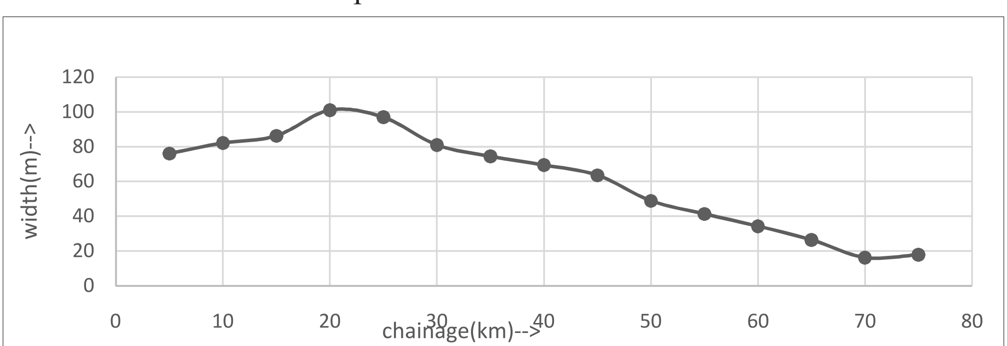


Figure 14 Variation of bed width of the river along the chainage

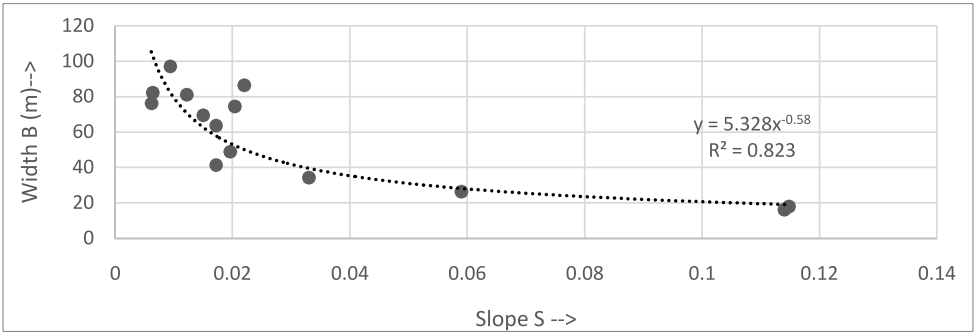


Figure 15 Variation of bed width with bed slope of the river.

Through the graphical representation between slope and top width of the river as given in Figure 15, a relation between slope and width has been developed as given below

$$B = 5.33\,2\; s^{-0.587} \qquad (2)$$

This shows the behavior of the river where width of the river decreases with increase in the slope.

Conclusion

The present study reveals that after the devastating impact of kedarnath 2013 flood, the geomorphology of Mandakini River has been changed significantly. The total eroded area is 5.75 km^2, which shows that the rate of erosion is prominent in the entire Mandakini River especially on the right bank. There are diminutive traces of sediment deposition on right bank. While there is sediment settlement near Rudraprayag and its vicinity after the flood. Through the analysis, the most critical locations were identified such as Kedarnath, Gaurikund, and Rambara etc. Starting with Kedarnath, where both right as well as left banks of Mandakini River has broadened its path and there are formations of new channels. Kedarnath valley and settlements which are sited on the banks of river Mandakini from Kedarnath to Sonprayag were damaged because of flood and some towns were completely washed away which includes Gaurikund and Rambara. It also demonstrates that the roads and footpath between Gaurikund and Kedarnath have been completely washed away. The second critical location was Sonprayag where the analysis demonstrates that erosion occurred to a large extent. Rudraprayag was identified as third critical location where Mandakini

River has migrated its channel towards the right.

Overall this study demonstrates that the utility and application of remote sensing data for the study of geomorphology plays a major role.It permits for the fast assessment of large as well as unreachable areas and for the monitoring of changes in the river channel, bank erosion and deposition that would be difficult to do using field study alone This study has further proved how the GIS approach has been useful in analysis of geo-spatial database and simplification of channel position mapping and its quantitative analysis.

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2. Survey paper (giving an original, detailed and critical view of a research problem or an area to which the author has made a contribution visible through his self-citation);
3. Short or preliminary communication (original management paper of full format but of a smaller extent or of a preliminary character);
4. Scientific critique or forum (discussion on a particular scientific topic, based exclusively on management argumentation) and commentaries.

   Exceptionally, in particular areas, a scientific paper in the Journal can be in a form of a monograph or a critical edition of scientific data (historical, archival, lexicographic, bibliographic, data survey, etc.) which were unknown or hardly accessible for scientific research.

Professional articles:

1. Professional paper (contribution offering experience useful for improvement of professional practice but not necessarily based on scientific methods);
2. Informative contribution (editorial, commentary, etc.);
3. Review (of a book, software, case study, scientific event, etc.)

Language

The article should be in English. The grammar and style of the article should be of good quality. The systematized text should be without abbreviations (except standard ones).

All measurements must be in SI units. The sequence of formulae is denoted in Arabic numerals in parentheses on the right-hand side.

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An abstract is a concise informative presentation of the article content for fast and accurate Evaluation of its relevance. It is both in the Editorial Office's and the author's best interest for an abstract to contain terms often used for indexing and article search. The abstract describes the purpose of the study and the methods, outlines the findings and state the conclusions. A 100- to 250- Word abstract should be placed between the title and the keywords with the body text to follow. Besides an abstract are advised to have a summary in English, at the end of the article, after the Reference list. The summary should be structured and long up to 1/10 of the article length (it is more extensive than the abstract).

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Acknowledgements

The name and the number of the project or programmed within which the article was realized is given in a separate note at the bottom of the first page together with the name of the institution which financially supported the project or programmed.

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All the captions should be in the original language as well as in English, together with the texts in illustrations if possible. Tables are typed in the same style as the text and are denoted by numerals at the top. Photographs and drawings, placed appropriately in the text, should be clear, precise and suitable for reproduction. Drawings should be created in Word or Corel.

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