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GZS College of Engineering &
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pkaurdap@gmail.com

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Wave Model Studies For Feasibility Of Developing A Passanger Jetty At An Open Coast- A Case Study

R.K.Chaudhari[1], S Kulkarni,[2] A.M.Vaidya, [3] M.D. Kudale [4]

[1]Mr, Scientist 'B', CWPRS,Khadakwasla Pune,India -411024

[2] Mrs, Scientist 'D', CWPRS, Khadakwasla Pune,India -411024

[3]Mrs,Scientist 'D', CWPRS, Khadakwasla Pune,India -411024

[4] Mr, Scientist 'E', CWPRS, Khadakwasla Pune,India -411024

Email: chaudhari_rk@cwprs.gov.in

ABSTRACT

Adequate area for maneuvering of the passenger vessels and berthing place with sufficient depth and tranquility are essential requirements for development of a Passenger Jetty. Also passenger berth must be easily accessible from the sea to enable quick embarkation and disembarkation of passengers and it must be sheltered to avoid excessive vessel movement and offer convenience and comfort. Understanding of environmental conditions of waves, tides, currents prevailing at the site is essential in designing the layout of the Passenger jetty. Maharashtra Maritime Board have a proposal of developing a passenger jetty at Kashid, Studies were carried out for optimization of harbour layout with the desired wave tranquility for the berthing of the passenger vessels at Kashid. Using numerical models different layouts were examined to get the optimum tranquility at the Port.

Keywords: Wave Tranquility, Numerical model, Random wave, Refraction, Passenger jetty

1. Introduction

Maharashtra has 35 creeks and rivers spread across 720 km long coast line. As per available information, approx. 15 million passengers in the coastal districts of the state use Inland Water Transport annually. Transport through waterways is the most convenient and the cheapest mode. In order to promote the water transport Maharashtra Maritime Board have a proposal of developing a passenger jetty at Kashid for ferry operations which will boost tourism in this area. Studies were carried out for optimization of harbour layout with the desired wave tranquility for berthing of the passenger vessels at Kashid.

Understanding nearshore waves is indispensable for the proper design and construction of coastal infrastructures. Waves attack coastal structure and infiltrate harbor entrance, generating disturbance in the sheltered waters. Hence, it is necessary to know beforehand the magnitude and direction of propagating waves to design the optimal layout of harbors. Outray wave model was used to transfer offshore wave data of wave heights and directions to the project site in 7m depth. Further, Boussinesq wave model was used for the prediction of the wave field due to combined refraction diffraction of directional random waves in a harbor. Using these numerical models two layouts were examined to get the desired tranquility in the Port. The siltation pattern at the site was also taken into consideration while finalizing the layout.

2. Site conditions:

Kashid situated at Latitude 180 27' 22.25"N and Longitude 720 53' 37"E on the west coast of India, in northern Konkan region of Maharashtra is famous for its beautiful beaches. The wind at kashid is mainly from SW direction having magnitude of about 5–10m/s. wind condition at kashid is moderate. While the tidal range at Kashid is 2.0m. This tidal increment is considered for the model studies.

Wave is the single most important parameter which influence design passenger jetty. Normally, measured wave data from the site of interest are not available, the only data available being from offshore locations. In such situation, it becomes necessary to study wave propagation from offshore region to the area of design interest. The offshore wave data at kashid reported by India Meteorological Department as observed from ships plying in deep waters off Kashid for 33 years were analyzed for studies (CWPRS T.R. No.5001). The frequency distributions of wave heights from different directions during entire year for the above offshore data are given in Tables 2. Fig.1

Table 2

WAVE HT(m)	0.5	1	1.5	2	2.5	3	3.5	4	4.5	TOTAL
DIRECTION								CALM %		6
22.5	0.97	1.71	1.16	0.58	0.16	0.05	0.02	0.01	0	4.65
45	0.99	1.58	1.09	0.75	0.21	0.12	0.02	0.03	0	4.79
67.5	0.32	0.57	0.56	0.33	0.08	0.07	0.01	0.01	0	1.96
90	0.49	0.77	0.47	0.23	0.1	0.06	0.01	0.02	0	2.15
112.5	0.59	0.46	0.34	0.14	0.05	0.01	0.01	0.01	0	1.61
135	0.5	0.57	0.36	0.15	0.04	0.02	0.01	0	0	1.66
157.5	0.48	0.88	0.6	0.4	0.13	0.05	0.02	0.01	0	2.57
180	0.56	1.46	1.04	0.79	0.27	0.14	0.04	0.05	0.02	4.36
202.5	0.31	0.75	0.77	0.58	0.28	0.22	0.1	0.05	0.04	3.1
225	0.72	1.47	1.22	1.34	0.8	0.77	0.37	0.44	0.2	7.35
247.5	0.45	1.17	1.44	1.8	1.39	1.52	0.8	0.85	0.29	9.72
270	1.19	1.93	2.12	2.19	1.65	1.85	0.72	0.82	0.23	12.7
292.5	1.4	2.1	1.88	1.37	0.77	0.71	0.22	0.28	0.06	8.79
315	2.27	3.51	2.08	1.22	0.32	0.15	0.08	0.07	0.01	9.72
337.5	2.08	3.65	2.81	1.38	0.32	0.18	0.03	0.01	0.01	10.47
360	1.59	3.15	2.2	1.07	0.24	0.09	0.04	0.03	0	8.42
TOTAL	14.92	25.75	20.15	14.3	6.8	6	2.5	2.7	0.86	100

Seasonal variation of shoreline is also seen on the south end of proposed site. Erosion takes place during monsoon due to high waves resulting in recession of the shoreline. During post monsoon period deposition of sediments results in advancement of the shoreline. The shoreline in the north portion of the site is formed of rocky outcrops and no movement of sediments is observed in this area.

3. Experimental conditions:

The permissible wave disturbance at the berthing place depends on the ship size, mooring and berthing system, the method of loading and unloading used at berth and on orientation of the berth with respect to wave direction etc. The proposed jetty at Kashid would be mainly used for Passenger transport. Keeping in view the size of passenger boats a permissible limit of 0.3 m for wave heights at the berth is considered for smooth embarkation/disembarkation of passengers. As waves propagate toward shore, a combination of shoaling, refraction, reflection, diffraction, and breaking effects modify the waveform, and the wave characteristics will be very different from those in deep water. To get the wave conditions from offshore to nearshore at 7m depth. The numerical model OUTRAY was used for studies. This model takes into account refraction and shoaling of waves, which are important in the transformation of waves from offshore to inshore. The model is based on wave ray technique, in which the wave ray is defined as orthogonal to wave crests, along which wave energy propagates. It makes use of the principle of reversibility of wave rays i.e. the path of a wave ray traced backwards is identical to the path traced forward This model creates a transfer function for wave conditions at the Refraction Point with respect to the offshore boundary (H.R.Wallingford 1989). Using the transfer function for different offshore wave directions and frequencies, wave heights and wave directions at the inshore point for different incident wave conditions are derived.

The OUTRAY model was run to obtain nearshore wave climate at the Refraction Point in 7 m depth contour. The wave direction and the transfer function (ratio of wave heights) in 7 m depth (Refraction Point) for waves with 10 sec wave period with different directions of wave incidence at the offshore boundary are given in the following Table:3

Table 3

Sr. No.	Offshore Direction	Inshore Direction	Transfer Function
	(Deg. N)	(Deg. N)	(Hi /Ho)
1	135.0 (SE)	219.8	0.28
2	157.5 (SSE)	222.4	0.47
3	180.0 (S)	226.5	0.66
4	202.0 (SSW)	232.1	0.82
5	225.0 (SW)	239.1	0.92
6	247.5 (WSW)	248.2	0.97
7	270.0 (W)	257.9	0.96
8	292.5 (WNW)	266.2	0.89
9	315.0 (NW)	272.6	0.77
10	337.5 (NNW)	277.3	0.6
11	360.0 (N)	280.1	0.4

Hi = Wave height at Refraction Point Ho = Offshore wave height

This transfer function was applied to the deep water wave data [Table2] to obtain the frequency distribution of waves in 7 m depth for and entire year [Tables 4] Fig.2

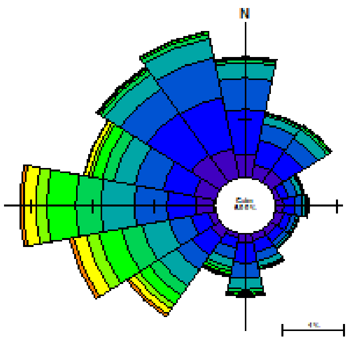
Table 4

WAVE HT(m)	<0.5	<1	<1.5	<2	<2.5	<3	<3.5	<4	<4.5	TOTAL
DIRECTION								CALM %		14.89
225	5.27	4.47	1.74	1.02	0.54	0.17	0.05	0.04	0	13.3
247.5	2.36	4.57	4.78	5.33	3.84	4.14	1.89	2.11	0.72	29.74
270	10.49	15.58	5.79	3.12	0.96	0.8	0.3	0.28	0.06	37.38
292.5	4.58	0.08	0	0	0	0	0	0	0	4.66
TOTAL	26.35	24.37	12.89	8.29	5.17	4.99	2.86	1.4	0.29	100

It is seen from these studies that the predominant directions at 7m depth are from SW, WSW, West and WNW with percentages 13%, 30%, 37% and 5%. Following are the wave conditions from different directions considered for simulation of wave transformation in the harbour using MIKE 21-BW model.

Input wave conditions for MIKE 21-BW Model

Wave direction (deg. N)	Wave height(m)
SW	2
WSW	3.5
West	2.5
WNW	1



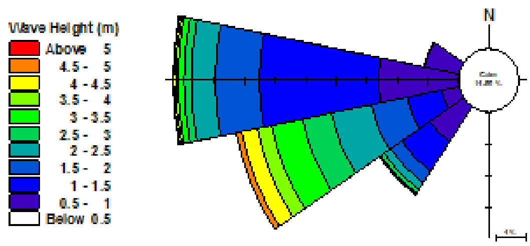
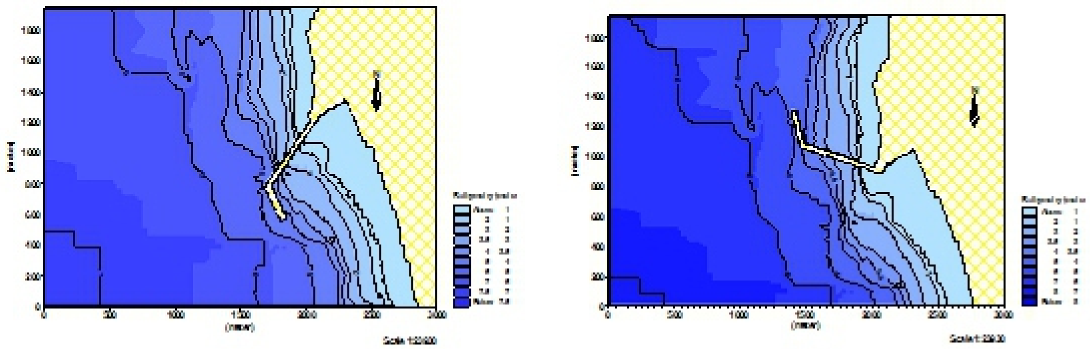


Fig.1: Rose diagram of Offshore Wave (Entire Year)   Fig.2: Rose diagram at 7m depth (Year)

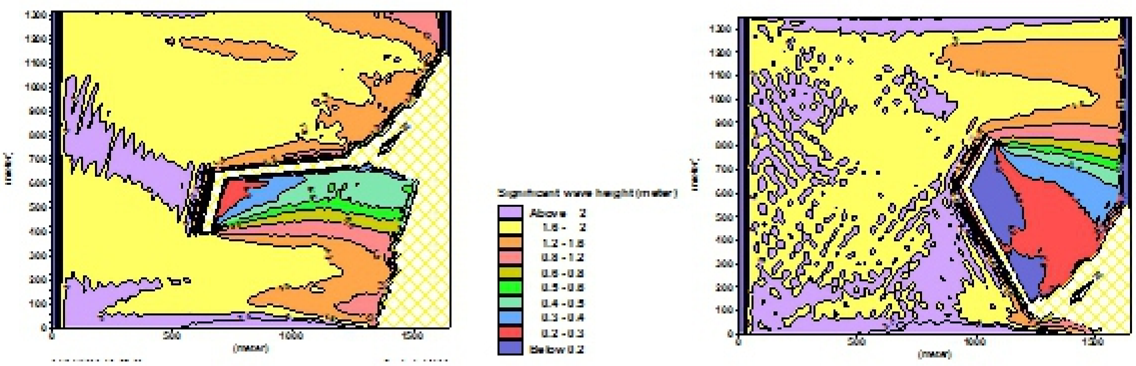
Wave propagation inside the harbour was simulated for above mentioned wave conditions for two layouts shown in Fig.3 Layout-1 consists of a breakwater of 770 m length. Shore connected part of the breakwater is 540 m long with a 230 m long arm of breakwater making an angle of 110 to it. Layout-2 consists of a breakwater of 780 m length. The breakwater consists of a shore connected arm of 550m length and a 230m long arm making an angle of 120 to the shore.



Layout-1 Layout-2

Fig.3: Proposed layouts of Harbour

The MIKE 21 BW simulations were carried out for above two alternative layouts. The model has been operated for the different wave conditions that we get from the result of OUTRAY model. The Numerical model MIKE21-BW was used for studying the wave disturbance in the harbour. This model is based on time dependant Boussinesq equations of conservation of mass and momentum obtained by integrating the three-dimensional flow equations without neglecting vertical acceleration. They operate in the time domain, so that irregular waves can be simulated. These equations include nonlinearity as well as frequency dispersion. The frequency dispersion is included in the flow equations by taking into account the effect of vertical acceleration or the curvature of stream lines on pressure distribution. The model simulates the processes of shoaling, refraction, diffraction from breakwater tips and bed friction. It also takes into account partial reflections from the boundaries, piers and breakwaters (DHI-2005). As the operation at the passenger jetty would be only for fair weather in such condition SW direction is the most critical and important wave direction for design consideration point of view so the output of the Mike 21 BW module for both layouts, for waves coming from SW direction are shown in fig.4.



Layout-1 Layout-2

Fig.4: Wave Height distribution for waves incident from SW direction (Layout - 1 and Layout -2)

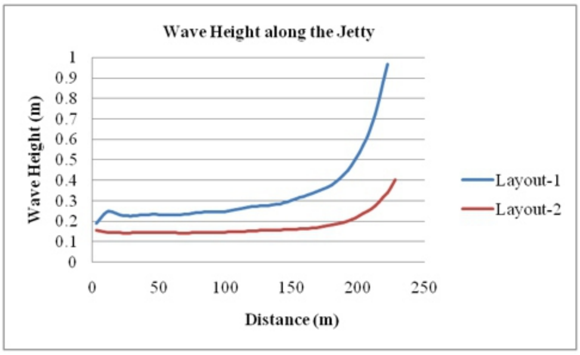


Fig.5: Wave Height distribution along the jetty for wave incident from SW Direction

For operation of the passenger vessels, wave height upton 0.3 m is desirable in the harbour As it is clearly seen from the fig:4 that the Layout -1 is mostly exposed to the higher wave heights while at Layout-2 the wave heights are within limit. It is seen from Fig.5 that the wave heights in the in Layout-1 is varying from 0.2 m to 0.9m near the jetty, for waves incident from SW direction however for the same direction at Layout-2 the variation is very small and wave heights are within limits, and also more tranquil harbor area is available with Layout-2 then the Layout-1. From considerations of probable siltation in the harbor, Layout -2 is more suitable.

4. Conclusion:

The layout of the passenger jetty was optimized through the numerical model studies to get the desired wave tranquility at the jetty. These studied were done by taking into account the development needs, planning consideration and the existing site conditions. It is seen from the model studies that layout with facing North direction is more suitable for the operation of the passenger jetty during fair weather. Thus the use of numerical model has been found to be beneficial in designing of a passenger jetty. The feasibility of developing a port is site specific and can be well predicted by the numerical models.

5. Acknowledgments

Authors are grateful to Mr. S. Govindan, Director, Central Water and Power Research Station for his continuous encouragement and motivation during the course of studies and for granting permission to publish this paper.

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Investigation of Gate Valve as a Flow Measuring Device

Ms. Vrinda. S. Suryawanshi[1], Dr. G. A. Hinge [2], Dr. N. J. Sathe [3]

1Assistant Engineer, Buldana Irrigation Project Circle, (Water Resources Dept, Govt. of Maharashtra), Buldana-443001, India.

2Prof. and Head, Dept. of Civil Engg., BSCOER, Pune-411041, India.

3Prof. Dept. of Civil Engg, Trinity Academy of Engineering, Pune-411058, India

Email: vrindasuryawanshi@gmail.com

A B S T R A C T

Gate valves are traditionally used for controlling the discharge in pipes. The valve is operated by rotating the wheel provided on the top a rod which is connected to circular disc on other side of it. Every single rotation of wheel produces specific linear movement of disc which in turn tends to change the area of flow. It is expected that for same disc position, the discharge should be same, but the analysis showed that the discharge varied considerably. Further analysis revealed that it is happening on account of the fluctuating separation zones formed on either side of the disc. To address this issue, it is proposed to use a flexible membrane pipe inside the gate valve. This membrane will not allow the separation zones to form, as its shape will automatically change depending upon the disc position and intensity of discharge. It is similar to that of venturimeter with dynamically adapting convergent and divergent cones. With this new adaption an experimental setup is fabricated with 1 inch pipe. The results have shown excellent improvement in the relationship between disc position and discharge. Further it is calibrated with respect to rotation angle of wheel. The experimental study has shown that the new gate valve can also be used as flow measuring device. It is then proposed to conduct tests with higher diameter pipes. The paper describes the journey of gate valve from flow control device to flow measuring device.

Keywords: Gate valve, Separation zone, Flexible membrane pipe, flow measuring device

1. Introduction

Gate valve (refer Fig. 1) is commonly used for many household and industrial applications. Many industrial applications demand accurate flow control for various processes. In fact Gate Valve itself is a flow control device. But on account of its inability to precisely control the flow, it needs to be supplemented with separate flow measuring device. Therefore gate valves, when used along with some flow measuring device like 'Venturimeter' or 'Orificemeter', can act as flow control device.

Fig. 1 Gate valve

Mechanism of gate valve:

The wheel of gate valve is connected to vertical disc by means of a rod. The clockwise and counterclockwise movement of wheel results into vertical movement of disc in downward and upward direction respectively.

Partially open gate valve tend to form separation zones on either side of disc as shown in Fig. 2. As the separation zones keep on fluctuating, it affects the coefficient of discharge which also keeps on changing over a period of time and results into discharge variation for same disc position.

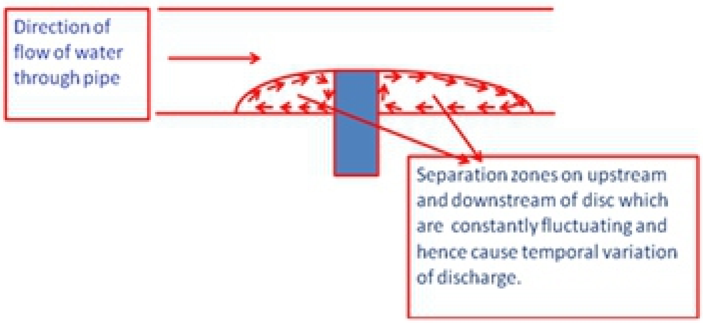

Fig. 2 Separation zones on either side of disc of gate valve

2. Literature Survey

Hinge G. A. (Hydro, 2004) has modified an experimental setup of flow through orifice by designing a novel inlet float valve. With the help of this valve the rate of inflow can be automatically controlled. The gadget works without any fluctuation over a wide range of pressure on upstream of gate valve.

For setting certain head on existing setup of tank orifice, the inlet valve as well as overflow pipe are required to be operated simultaneously. But it was observed that, once a constant head is acquired, after some time it was changing. This created lot of difficulties in carrying out the experiments. Fluctuation of inflow discharge was the main reason behind this disturbance. And hence to overcome this problem a flexible pipe was inserted in the float valve and the outlet of which was dropped in the main tank. This caused perfect steady uniform inflow. Therefore for eliminating the similar problem of gate valve, it was decided to make use of flexible pipe.

3. Methodology

To experimentally investigate the flow conditions, two setups, identical in all respect, were fabricated. The only difference was, one is having traditional gate valve and the other was having flexible membrane pipe along with gate valve. The length of flexible membrane pipe was designed based on the consideration of length of separation zone. The inflow was taken from the same supply main. For both the models various trials were taken to measure discharge for same disc position of valve. Such trials were taken for different rotational angles of wheel. By using results of model study with flexible membrane pipe, mathematical equation for discharge was formulated.

4. Experiential observations

Fig. 3 shows the experimental setup with conventional gate valve. The discharge was measured on volumetric basis. While carrying out the tests, the fluctuating separation zones were observed as shown in Fig. 4.

Fig.3 Experimental setup of conventional model

For the conventional model 25mm metallic gate valve is selected. Complete closing of this valve requires 8 complete and 1/3rd of rotation of wheel which results into angular measure of 30000. Before fixing the valve 600 interval points are marked along the periphery of bonnet of the valve. On both the sides of valve, acrylic pipe of length equal to development length of 1250mm is provided. Transparency of acrylic pipe helped to observe the flow conditions.

For the conventional model 25mm metallic gate valve is selected. Complete closing of this valve requires 8 complete and 1/3rd of rotation of wheel which results into angular measure of 30000. Before fixing the valve 600 interval points are marked along the periphery of bonnet of the valve. On both the sides of valve, acrylic pipe of length equal to development length of 1250mm is provided. Transparency of acrylic pipe helped to observe the flow conditions.

Fig.4 Zone formation in partially opened gate valve

The experiments were started with completely open valve and measuring the corresponding discharge. Then it is partially closed by rotating it in clockwise manner through an angular interval of 360º, that is one rotation of wheel and the discharge was measured. The same procedure was repeated until the valve was completely closed. On similar line, the readings were taken while opening the valve at an interval of 360º by rotating the wheel in anticlockwise direction. 3-sets of such pilot readings were taken. For the reference one of the set is given below. It was noticed that, for same angular position of wheel, the discharge was not remaining constant. This has given an indication of the problem associated with traditional gate valve.

Table 1 Readings-set 1 (clockwise rotation)

Serial No.	Volume (ml)	Angle of rotation (degree)	Time (second)	Discharge (mlps)
1	500	0	0	0
2	500	360	105.2	4.75
3	500	720	4.1	121.95
4	500	1080	3.2	156.25
5	2000	1440	10.2	196.078
6	2000	1800	8.5	235.29
7	2000	2160	7.1	281.69
8	2000	2520	6.3	317.46
9	2000	2880	5.5	363.63
10	2000	3000	5.1	392.156

Table 2 Readings-set 1 (anticlockwise rotation)

Serial No.	Volume (ml)	Angle of rotation (degree)	Time (second)	Discharge (mlps)
1	2000	3000	5.2	384.62
2	2000	2880	5.9	338.98
3	2000	2520	6.7	298.507
4	2000	2160	7.2	277.78
5	2000	1800	8.7	229.88
6	2000	1440	10.3	194.174
7	500	1080	3.1	161.29
8	500	720	4.3	116.28
9	500	360	92.3	5.417
10	500	0	0	0

Similarly as in first set of readings next two set of pilot readings are taken in both directions. After the 3 sets of pilot readings in clockwise and anticlockwise direction, the angular interval was reduced to 60º in view of enhancing precision. With this new interval one more set of readings with clockwise and anticlockwise rotation was recorded.

Another model with identical parameters is assembled along with flexible membrane pipe as shown in Fig. 5. Purpose of using flexible pipe is to avoid formation of separation zones.

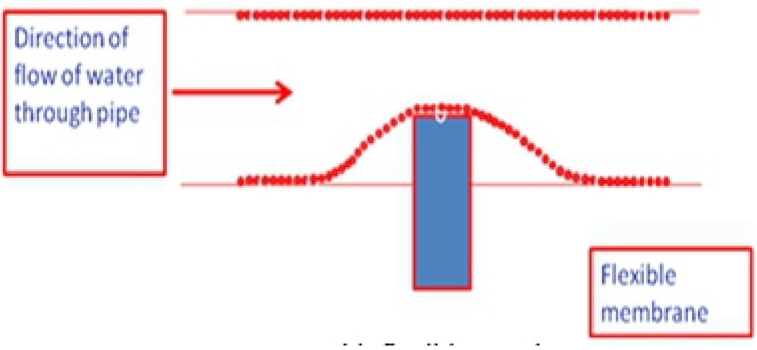

Fig 5 System with flexible membrane

This system consists of 25mm metallic valve fixed along the acrylic pipe on either sides having flow development length of 1250mm with the same pressure at inlet. This flexible pipe is made up of nylon material which used in canopy of umbrella. Length of the flexible pipe depends on the distance of formation of separation zones on either side of the disc. For the present case it is 128mm on either sides of valve.

Length of flexible pipe

= length of separation zone on either sides + width of valve

= (128+128) +150

=406mm

To be on safer side 450 mm length of flexible pipe was taken.

Fig.6 Experimental setup with flexible membrane

Same procedure is followed for the recording of reading. First of all 3 sets of pilot readings were taken which are having interval of one complete rotation (3600) in clockwise and anticlockwise direction.

Table 3 Readings-set 1' (Clockwise rotation)

Sr. No.	Volume (ml)	Angle of rotation (degree)	Time (second)	Discharge (mlps)
1	1000	0	0	0
2	1000	360	51.7	19.34
3	1000	720	18.8	53.19
4	1000	1080	12.9	77.51
5	1000	1440	12.2	81.96
6	1000	1800	10.1	99.00
7	1000	2160	9.9	101.01
8	1000	2520	9.5	105.26
9	1000	2880	9.1	109.89
10	1000	3000	8.9	112.36

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Table 4 Readings-set 1' (anticlockwise rotation)

Sr. No.	Volume (ml)	Angle of rotation (degree)	Time (second)	Discharge (mlps)
1	1000	3000	8.6	116.27
2	1000	2880	9.2	108.69
3	1000	2520	9.4	106.38
4	1000	2160	9.8	102.04
5	1000	1800	10.0	100
6	1000	1440	11.9	84.03
7	1000	1080	12.8	78.125
8	1000	720	18.6	53.76
9	1000	360	49.9	20.04
10	1000	0	0	0

After the 3 sets of pilot readings in clockwise and anticlockwise direction it was observed that the discharge was almost same for the same position of disc of gate valve. By keeping 60^0 as interval one more set of clockwise and anticlockwise readings was recorded for precision.

5. Result and Discussion

For each set of readings graphs are drawn to analyze the results. On X-axis angle of rotation of handwheel is taken by keeping the interval of 360^0 for convenience while discharge is taken on Y-axis in milliliter per second, Graphical presentation of pilot readings which are taken at an interval of 360^0 of conventional system shows scattered pattern as discharge varies each time due to formation of fluctuating separation zones. While in case of plot of pilot readings with flexible pipe, the pattern observed is in close proximity. This shows the negligible change of discharge. Graphical representation shows the difference between the discharge variation of conventional model and the model with flexible pipe arrangement.

For fig. no. 7 and 8 shows the graphical lines for 360^0 rotational interval of handwheel (3 sets of pilot readings) for conventional model and the modified setup respectively. C and A stands for clockwise and anticlockwise rotation of handwheel respectively.

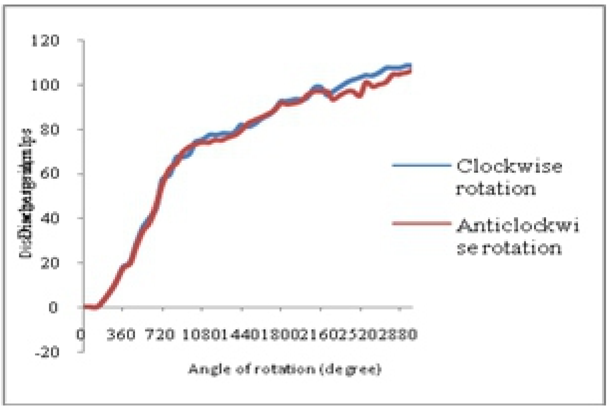

Fig 10 Graphical representation of readings of system with flexible pipe

After complete analysis of results use of flexible pipe removes the lacuna of use of gate valve and keeps the discharge constant. So gate valve can be used as a flow measuring device.

6. Conclusions

It is evident from the experimental study that in the traditional gate valve, for same disc position and at different times, one does not obtain same discharge on account of formation of fluctuating separation zones. The separation zones can be avoided by providing a flexible membrane pipe inside the gate valve. This drastically improves the performance of gate valve as flow control device. Further by proper calibration for a particular pressure on upstream, it can also be used as flow measuring device.

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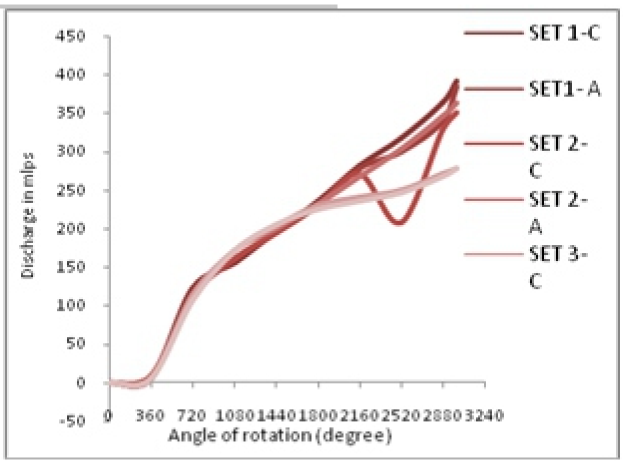

Fig 7 Graphical representation of pilot readings of conventional system

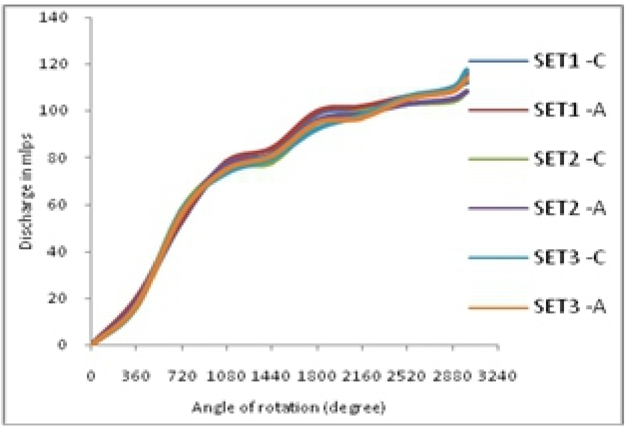

Fig 8 Graphical representation of pilot readings of system with flexible pipe

Similarly, for the readings taken at an interval of 60^0 the graphical line with flexible pipe shows closer pattern while conventional system is having scattered one. Figure no9 shows scattered and separated graphical lines, which are showing the discharge of conventional model at a rotational interval of ^{0}60 in clockwise and anticlockwise direction. Fig. no, 10 shows close proximity of graphical lines drawn for model with flexible pipe arrangement.

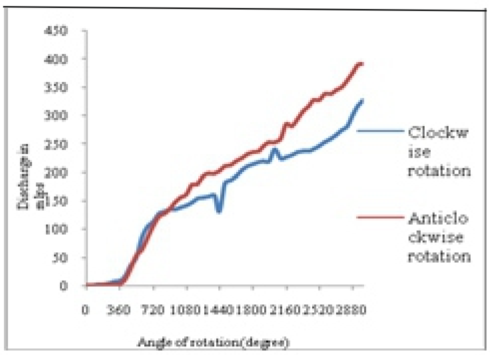

Fig 9 Graphical representation of readings of conventional system

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Runoff Estimation of Chambal River Basin for Forthcoming Years, Using VIC and ARIMA Model

Ankur Dixit,[1] Bhaskar R. Nikam,[1] Shiv Prasad Aggarwal,[1] Vaibahv Garg,[1] Prasun K. Gupta,[2] Praveen K. Thakur [1]

[1]Water Resources Department, Indian Institute of Remote Sensing-ISRO, Dehradun;

[2]Geo-Informatics Department, Indian Institute of Remote Sensing-ISRO, Dehradun

Email: ankur.dixit@hotmail.com

A B S T R A C T

Climate change and global warming have emerged as one of the most important environmental issues in the last two decades.Changing climate is affecting water resources significantly. To assess this impact on water resources requires modeling and simulation of hydrological processes. These processes can be modeled through hydrological modeling.Since the hydrological studies are done at a basin or catchment level so Chambal River Basin has been selected for this study.In this paper an attempt has been done to study the effect of changing climate on the hydrology of Chambal basin over the years and predicted for the forthcoming years subsequently. Variable Infiltration Capacity (VIC) model, a grid based macro-scale semi-distributed hydrological model, has been used to simulate the physics based hydrological processes between climate and hydrological regime. VIC requires minimum four set of parameter files i.e. vegetation parameter file, soil parameter file, global parameter file and meteorological files. Meteorological contains the components, governing atmospheric conditions to simulate the hydrological regime.Climatic data of 62 years (1951 to 2012) has been used in present study as input meteorological forcing parameters. Five meteorological parameters have been used viz. minimum temperature, maximum temperature, precipitation, wind speed and cloud cover to generate daily meteorological forcing for VIC Model. VIC model requires vegetation parameter file to account the effect of different type of vegetation on hydrological conditions, these parameters have been derived using Land Use Land Cover (LULC) map of the study area. The LULC of year 2005 mapped under ISRO Geosphere Biosphere Program (IGBP) has been used as base map and the same LULC scenario has been used to model hydrological regime of Chambal basin for the period 1951 to 2012, so that observed change in hydrology is not led by the change in LULC. After simulating the model for 62 years, the prediction for the upcoming years (till 2025) has been done using Autoregressive Integrated Moving Average (ARIMA) model. To validate the significance of projected runoff potential of Chambal basin, the cross comparison has been done using actual simulated runoff using VIC model and projected runoff using ARIMA model for the same period of time 2003 to 2012. Regression line has been plotted between both the observations and coefficient of regression comes as 0.813.

Keywords:Chambal basin,climate change, VIC model,ARIMA model.

1. Introduction

Climate change and global warming arising from anthropogenic- driven emissions of greenhouse gases have emerged as one of the most important environmental issues in the last two decades. According to IPCC(2001), climate change is likely to impact stream flow volume as well as the temporal distribution throughout the year over the Asian region, imposing significant stress on water resources. Recently, there has been growing scientific evidences that global climate has changed. Global climate changes induced by increases in greenhouse gas concentrations is likely to increase temperatures, change precipitation patterns and probably raise the frequency of extreme events (IPCC, 2001). Although the subject of climate change is vast, the changing pattern of precipitation is one topic within climate change that deserves urgent and systematic attention as it will affect the availability of food supply (Dore, 2005).According to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change (IPCC 2007; Bate et al. 2008), the global average surface temperature has increased by about (0.74±0.18)°C and precipitation has decreased over much of the northern hemisphere sub-tropical regions by about 3% during the past 100 years.The changes, induced by climatic variability, in regional hydrology include alterations in patterns of stream discharge and ground water recharge; changes in snowpack (which affect stream discharge); increased evaporation from lakes, reservoirs, and other freshwater bodies; increased incidence of severe flood and drought events; and changes in water quality.To quantify these changes in hydrological regime of any region the only feasible option available is hydrological modeling. However, these types of modeling attempts must only be done on hydrological units (i.e. watershed, catchment, basin) rather than geo-political administrative units.

The Chambal River, a tributary of the Yamuna River in central India has an irreplaceable role in supporting sustainable development of the society and economy in northernIndia.It is one of the last remnant rivers in the greater Ganges Riversystem,which has retained significant conservation values. Since there is a large population people living under the stake of Chambal River so the uncertainty of water resources needs to be address. The status and availability of the water in the river catchment area is essential to study for the water resources management and better resource utilization point of view. Therefore the objective of this study is to assess the present status of potential runoff in the basin and availability of water for forthcoming years also. In this study more than half of century (1951-2012) data has been modeled through Variable Infiltration Capacity (VIC)model to estimation runoff of the basin area. To setup VIC model, the parameters files were generated in the prerequisite format of VIC model. There are four essential parameter files i.e. vegetation parameter file, soil parameter file, global parameter file and meteorological files. These parameters files were generated using VICTOOL (open source software, developed at Indian Institute of Remote Sensing, ISRO to prepare the parameters file in their proper format) (Gupta 2012).Along with these essential files Elevation band file was also used for better approximation of energy transfer scheme between land surface and atmosphere in high elevation area. Once the parameters files are generated, model has been run to compute the runoff estimation. Since we have estimated potential runoff of the basin for more than half of century so the these results will certainly indicate the climate induced changes in hydrological regime. If the long term runoff is analyzed using time series analysis model then the response of hydrological regime towards changing climate can be predicted. So this long term runoff is analyzed using ARIMA model. The estimated runoff is undergone for time series analysis using ARIMA model and the water availability is projected for forthcoming years.

1.1 Distributed Hydrological Model (VIC)

Variable Infiltration Capacity (VIC) model is a macro-scale hydrological model based on soil-vegetation-atmosphere transfer scheme (SVATS), which was designed to describe the land surface in numerical weather prediction and climate, describe the variation and transfer of water and energy (Liang et al 1994; Aggarwal et al. 2013). This model is developed by Xu Liang at the University of Washington (Liang et al., 1994) and later improved by Lohmann et al. (1998), and Liang and Xie (2001). The model solves the physical processes of water and energy exchanges between soil, vegetation and atmosphere. It has widely been used for climate change impact study by many researchers. To simulate the hydrological scheme, VIC solves the physical model for water balance and energy balance while the relationship between infiltration, runoff and base flow is derived empirically (Liang et al 1994).

The VIC model can be run in various modes such as energy balance mode, water balance mode and full energy balance mode for sub-daily simulation. In present study two layered VIC model was set up at 24 hour time step.

2 . Study Area

The Basin of Chambal River, a tributary of the Yamuna River has been selected as study area in present analyses. The entire Chambal basinlies between a longitude of 74° 45' to 75° 50' E and latitude of 23° 30' to 25° 10' N and spread over the states of Madhya Pradesh and Rajasthan as shown in Fig. 1. The Chambal River which originates from Vindhya hills flows into Madhya Pradesh and finally drains through Rajasthan. It has immense potential of water harvesting. Four large water resources projects, namely Gandhi Sagar reservoir in Madhya Pradesh, RanapratapSagar reservoir, JawaharSagar reservoir and Kota Barrage in Rajasthan, constructed on river Chambal, have a total catchment area of about 27000 sq. km. A large portion of catchment area, about 22850 sq. km is however in Madhya Pradesh and the rest about 4150 sq. km falls in the state of Rajasthan. The river has a course of 965 km up to its confluence with the Yamuna River in the Etawah district of Uttar Pradesh.

The climate of the region in general is of sub-humid with an annual average rainfall varying from about 650 mm to 850 mm. In general the topography of the region is undulating. The predominant slope in the hilly region ranges between 15 to 40 percent and in the valley portion it varies from almost level to 10 percent. Erosionproblem in the catchments is quite intensive because of undulating topography, misuse of land and degradation of forest. Sheet and gully erosion are common in the area especially near the river.

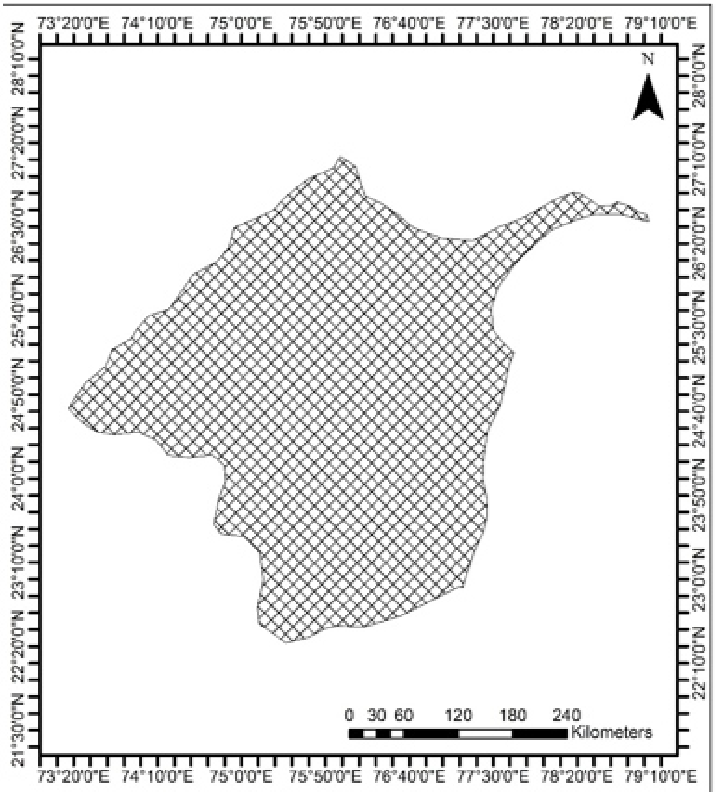

Figure -1: Chambal Basin boundary map

3. Data Used

Brief description of data used to simulate the hydrological regime of Chambal basin using VIC model has been given in this section.

3.1 Digital Elevation Model

ASTER DEM with 30 m spatial resolution has been used in present study to derive topographical parameters (basin boundary, flow direction, average elevation per grid, elevation band file, etc.) needed for modeling hydrological regime of the basin using VIC model. Figure-2 represents the DEM of Chambal basin.

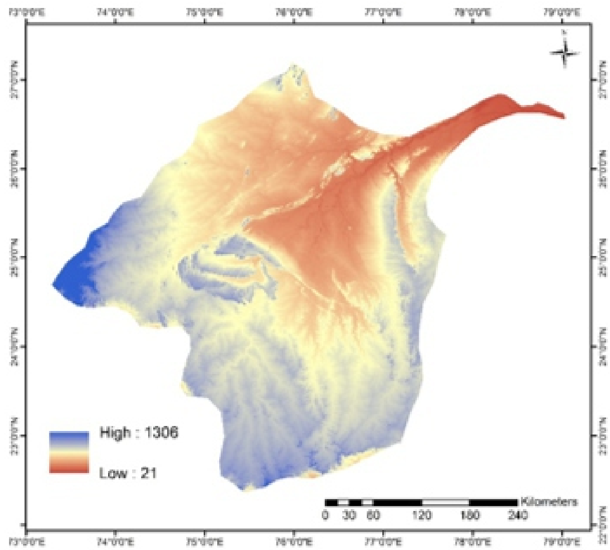

Figure -2: DEM of Chambal Basin

3.2 MeteorologicalData

Meteorological datasets are used as an external forcing inputs to the hydrological modeling. In VIC model minimum four parameters are required for water balancing of the basin. These three forcing inputs are Minimum Temperature, Maximum Temperature, Wind Speed and Precipitation. Cloud Cover has also been used in addition to the minimum set of parameters required. These datasets have been provided by the Indian Meteorological Department (IMD) in text file format. These text files then converted into the raster image format. Figure 3 depicts the annual total rainfall over Chambal basin in year 2005. The daily meteorological raster files were stacked together for each year and then the grid wise meteorological forcing files were generated using the VICTOOL(Gupta, 2012). The total precipitation map of Chambal Basin for year 2005 has been shown in Figure - 3.

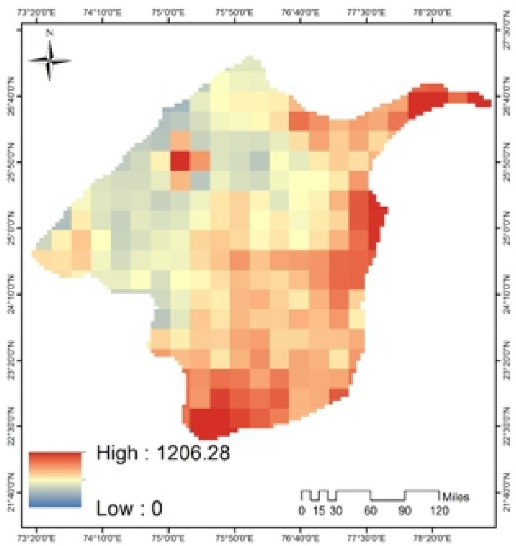

Figure – 3: Map of Total Precipitation (mm) in Chambal Basin in year 2005.

3.2 Land Cover/Land Use and Soil

Land Use Land Cover of Chambal basin has been used to prepare the vegetation parameter file which consists the fraction of the all LULC classes present in the hydrological unit. LULC that has been used consist nine classes named water body, mixed forest, broad leaf forest, shrub land, plantation, grass land, cropland, built up area and barren land is used. The LULC map of Chambal Basin is shown in Figure - 4.

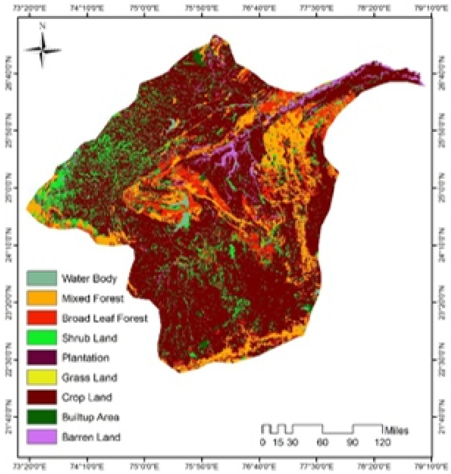

Figure – 4: LULC Map of Chambal Basin of Year 2005

3.3 Soil layer

Soil map is used to define the soil type and their fractional area in each grid. The soil map that has been used is provided by NBSSLUP, Nagpur.

4. Methodology

The flowchart of overall methodology adopted in present study is shown in Figure-5.

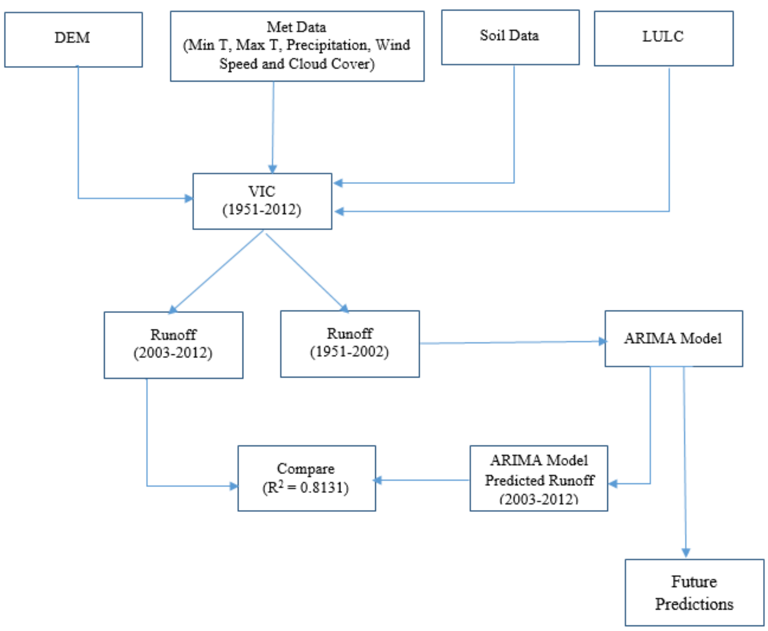

Figure – 5: Flowchart of the adopted methodology

Meteorological forcing parameters, soil type information, LULC type and elevation derived from DEM have been assimilated into the VIC model to simulate hydrology of the basin. There are many outcome of this simulation however in present study major emphasis is given to surface runoff only. To estimate the runoff for future years we need to have base information about the changing scenario. To acquire this information we have run VIC model for the past 62 years i.e. from year 1951 to 1962. We have not changed LULC and kept the same land cover inputs for all the 62 years i.e. as per LULC of year 2005. The reason behind not changing the LULC over the years is to negate the compensation effect of changing LULC classes over changing climate. Since LULC changingwith climate can change the hydrology with significant effect, however, we are interested in knowing only the changes caused by climatic change. As the 62 years runoff of any basin can be treated as a base period to understand the changing conditions of the basin and same can be predicted for the upcoming years subsequently. To analyze this time series of surface runoff, ARIMA model has been used.

The ARIMA model has been fed with runoff of past 62 years (1951 to 2012) and projected till year 2025. This model needs to be verified with some evidence. Since the observed discharge is not available of this basin so we have cross verified runoff with simulated and projected values. For cross verification ARIMA model has been run to project runoff of years 2003 to 2012 while inputs have been given from 1951 to 2002. Then projected and simulated runoff values have been compared for the same period.

5. Result And Discussion

Total runoff at each grid has been simulated and shown in Figure – 7. Total precipitation map is shown in Figure – 6.

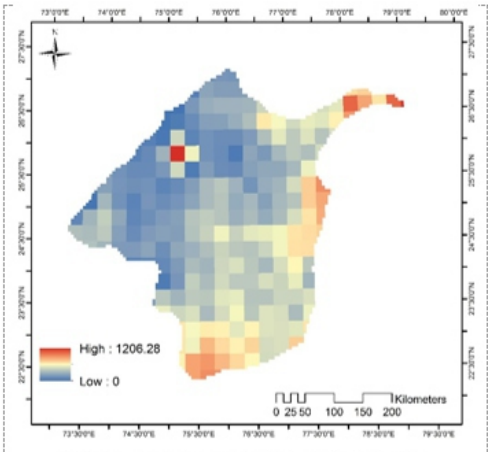

Figure – 6: Total Rainfall Map of Chambal Basin

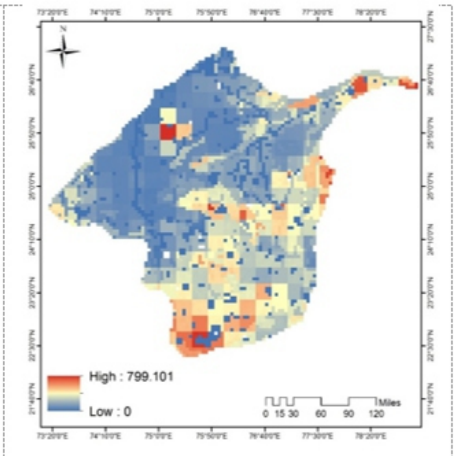

Figure – 7: Total Runoff Map of Chambal Basin.

From the maps of rainfall and runoff it is very clear that the area where rainfall is higher, runoff is high and similarly where rainfall is lower, runoff is low. The highest rainfall is 1206 mm while the highest runoff is 799 mm.The runoff values are simulated using VIC model for the years from 1951 to 2012. This runoff is fed up into ARIMA model to analyze the time series runoff data. This runoff is used as base runoff data for the prediction of upcoming years (year 2016 to 2025). The predicted runoff values are shown in Figure – 8.

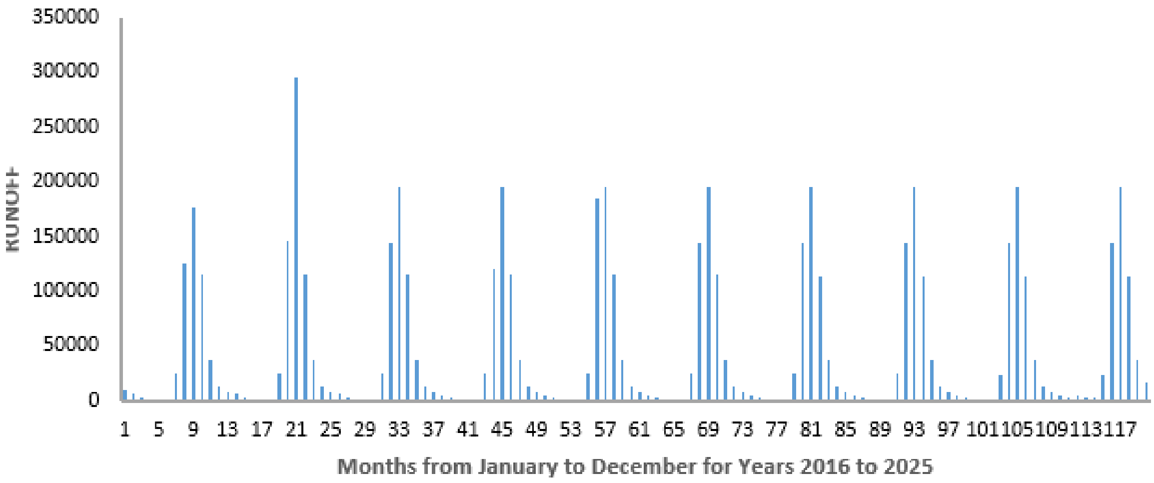

Figure – 8: Estimated Runoff of years 2016 to 2025

These predicted values are subjected to validation. Since the Chambal Basin is under the classified basin so observed data is not available. To validate the results we have cross compared the results by VIC model and ARIMA model. The VIC model has been run for the time period of 1951 to 2002. This runoff data is then fed up into the ARIMA model to predict the runoff values for years 2003 to 2012. Now ARIMA predicted runoff and VIC simulated runoff for years 2003 to 2012 have been compared and plotted, the plot is shown in Figure- 9. As the Figure-9 shows both the observations are showing good relation to each other.

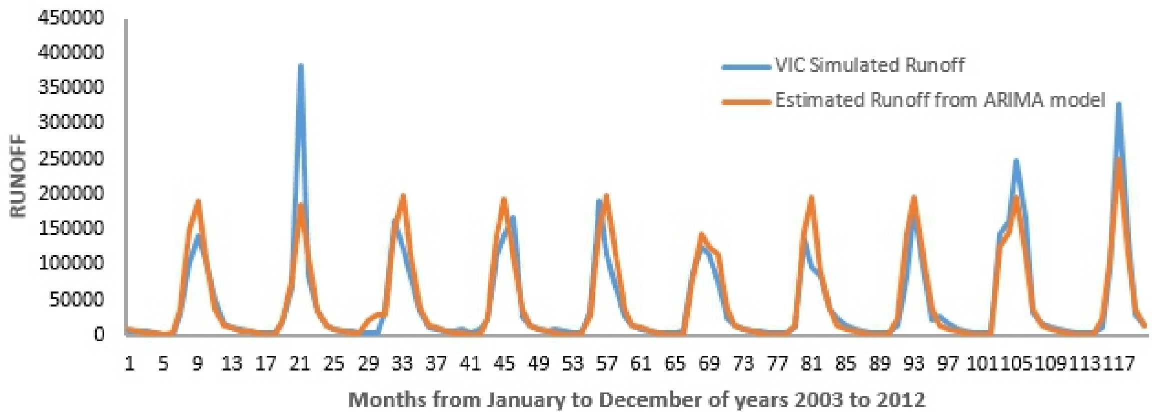

Figure – 9: Graph between simulated runoff through VIC model and estimated by ARIMA model

The regression plot between ARIMA predicted and VIC simulated runoff has been plotted. Both of the observations are competing very well to each other with the coefficient value of determination is 0.8131. The regression plot has been shown in Figure-10.

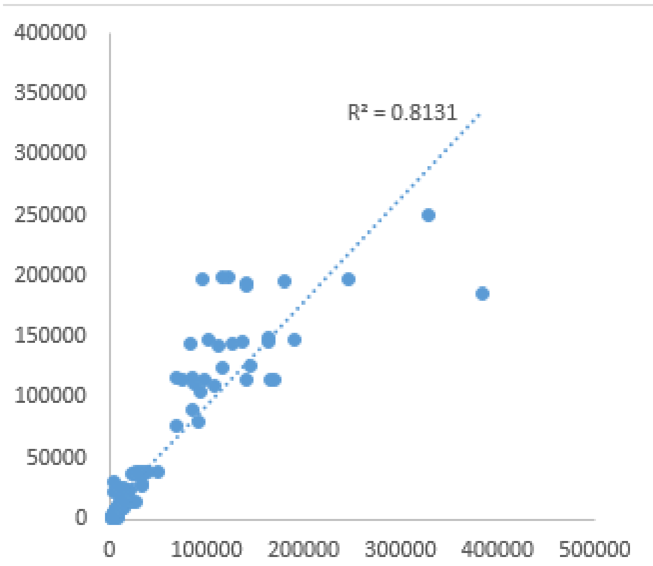

Figure – 10: Regression plot between simulated runoff through VIC model and estimated by ARIMA model

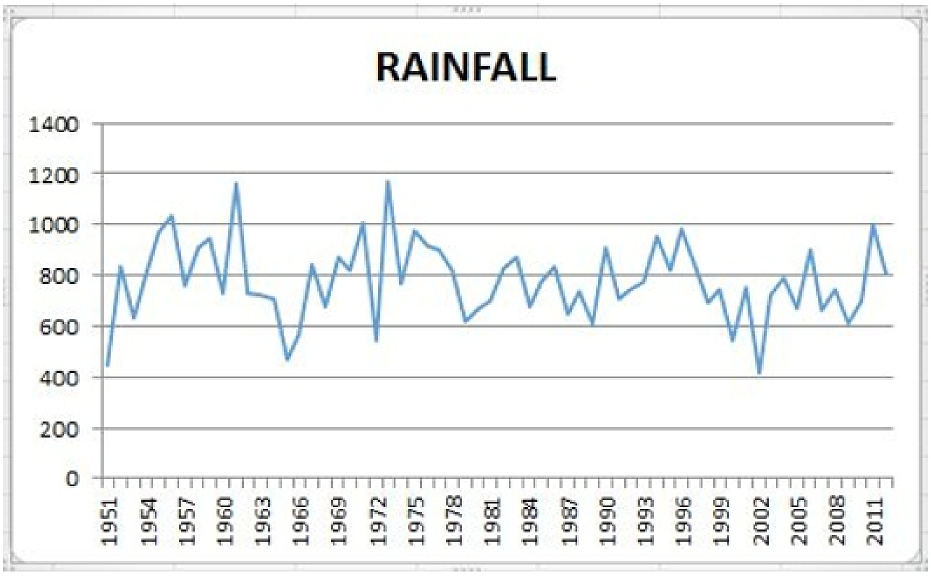

Figure – 11: Rainfall data from year 1951 to 2011.

From Figure-11, Rainfall is decreasing over the years. Rainfall values for years 1951 to 1980 is significantly high but after that it is having decreasing trend. Subsequently runoff potential has not altered much of the basin for upcoming years. If the other variables are same then for ideal situations runoff should also decrease if rainfall is decreased.Although runoff is decreasing but not as much as rainfall is so relatively runoff is high.

This is an evidence of alteration in hydrological regime due to climate variability and this scenario arise the situation of uncertainty for the future water availability.

6. Conclusions

Since water availability is very important for water resources management, planning and distribution so if future water availability can be predicted it will be very efficient for the policy planning and better utilization. Since there is no way to simulate the resource availability for future except modeling. So in this study an attempt has been made to model the available water potential of Chambal Basin. In this study it has also been observed that over the time rainfall has decreasing trend i.e. less average rainfall for upcoming years but runoff is not decreasing much. So water availability is decreasing over the time which necessitates to imply the better water resources planning for management and distribution.

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Determining Friction Factor for DI Pipes By Direct Surface Roughness Measurement

D.A.Shah[1], Dr. D. T. Shete [2]

[1]Assistant Professor, Civil Engineering Department, Faculty of Engineering & Technology, Parul University, At & Po. Limda, Ta. Waghodia, Vadodara-391760, India

[2]Professor, Civil Engineering Department, Faculty of Engineering &Technology,Parul University, At & Po. Limda, Ta. Waghodia, Vadodara-391760, India

Email: devang.shah@paruluniversity.ac.in

A B S T R A C T

Since Moody published his world famous friction factor charts based on Colebrook-White formula, engineering fraternity seems to be following it so religiously that the research gap highlighted by Colebrook and Moody themselves way back in the last century, in the form of absence of direct method for determination of roughness values for commercially available pipes, is mostly forgotten with advent of time. The present study addresses this issue and presents the improved methodology for determining friction factor by direct measurement of surface roughness for different diameter cement mortar lined DI pipes. Roughness of DI pipes was directly measured by TR100 Surface Roughness Tester (Profilometer) at laboratory of Mechanical Engineering Department, Faculty of Engineering & Technology, Parul University, Limda, Vadodara.The values obtained for surface roughness height,considering mean peak to valley height Rz; measured with stylus instrument, is observed in the range of 0.00378 to 0.038 mm. This is well in agreement with value of Rzfor cement lining material given by Farshad et al. (2005) and equivalent surface roughness e given by CPHEEO Manual (1999) and establishes validity of the mechanical measurement of roughness. The friction factor is calculated using Churchill's equation by replacing equivalent roughness e with mean peak to valley height Rz. Friction factor for different diameters of DI pipes varying from 200 to 600 mm were calculated and their relation with Reynolds number was analyzed. It was found that up to Reynolds no. of 2,200 tenth order polynomialequation; from 2,200 to 3,900 ninth order polynomial equation, from 3,900 to 18,000 tenth order polynomial equation and from 18,000 to 62,000, from 62,000 to 1,33,000and from 1,33,000 to 5,00,000 fifth log or fifth inverse log equation with different values of constants; and from 5,00,000 to 19,01,000 fifth log equation can be used to determine friction factor directly for given Reynolds number. The regression equations given above establishing direct relationship between friction factor and Reynolds number eliminated the iterative procedure mandatory in case of Churchill's equation.When friction factors calculated by Churchill's equation were plotted against those obtained by the aforesaid equations for the given Reynolds numbers, the resulting equation was y = x – 6.19E-8 and coefficient of correlation R2 = 1.The present paper is a step forward towards more accurate measurement and representation of surface roughness parameter in order to derive more realistic value of friction factor leading to efficient design of pipes.

Keywords:DI Pipes, Friction factor, Mean-peak-to-valley-height Rz, Colebrook-White, Moody's Chart, Measured Roughness

1. Introduction

The hydraulic design and analysis of flow in pipe networks is depended on correct estimation of the friction factor f. Since the mid 1940s, practicing engineers, academicians and students of engineering have made use of a diagram of friction factor for pipe flow published by Lewis Moody in the Transactions of A.S.M.E. in 1944 (Moody 1944). It is very important to understand how internal-surface wall-roughness, texture and its spatial distribution affects the flow of fluids in pipeline for real time exact estimation of head loss, combating the effects of pipe corrosion, establishing new piping materials and coatings, increasing the ease of fluid flow in pipes, and validating such improvements. The present study is carried out to establish method for physical determination of surface roughness for cement mortar lined DI pipe and accurate determination of friction factor.

Central Public Health and Environmental Engineering Organization Manual on Water Supply &Treatment (1999) page 106 states, “In today's economic climate it is essential that all water utilities ensure that their resources are invested judiciously and hence there is an urgent need to avoid overdesigning of the pipelines. Despite technological advancements, improved methods of manufacturing all types of pipes and advent of new pipe materials, the current practice of adopting conservative Coefficient of Roughness (C values) is resulting in under utilization of pipe materials.” On page 113 it is stated “...Continued usage of roughness coefficients estimated without recognition of these advances is bound to result in conservative design of water supply system......The extent of underutilization varies 13 to 40 percent for CI pipes, 23 percent for RCC and AC pipes; and 8.4 percent for HDPE and PVC pipes.”

Further, as mentioned in the same manual the best method for friction factor determination is Darcy-Weisbach equation followed by modified Hazen-Williams formula. Bombardelli&Garci (2003) and Bennett & Glaser (2011) had shown that use of Hazen-Williams formula have very limited range of applicability and may have very detrimental effect on pipe design. So, obviously best alternative for all flow regime and all types of pipes is dimensionally balanced Darcy-Weisbach equation.

1.1The research gap

The determination of friction factor is normally done by universally adopted practice using Moody's chart. The fact that the friction factor diagram in the form set out by Moody still endures is due to the use of dimensionless quantities and an effective consensus to continue to use the constants and underlying equations that were used by Moody . Moody and his contemporaries were under no illusions about accuracy of the diagram provided by them. Moody (1944) stated “It must be recognized that any high degree of accuracy in determining f is not to be expected....But, in the transition and rough pipe regions, we lack the primary and obviously essential, a technique for measuring the roughness of a pipe mechanically. Until such a technique is developed we have to get along with descriptive terms to specify the roughness; and naturally this leaves much latitude." And this obviously essential fact has either been neglected or lost with passing years. The Moody's charts are based on Colebrook equation which bridged the gap between uniform sand grain roughness curves (Nikuradse 1933) and non uniform commercial pipe roughness.

Hence, while using either Nikuradse (1933) or Colebrook (1939) equations it is very important to understand that the roughness parameter is either defined by size of uniform sand grain particle used in pipes todevelop artificial uniform surface roughness by Nikuradse or for commercial pipe back calculated hydraulically using,

$$\frac{\tau_0}{\rho U^2} = \frac{1}{8}\left(2\log\frac{3.7d}{k}\right)^{-2} \qquad (1)$$

Where,

$\tau 0$ = Shear stress at wall,

ρ = Density of fluid

U = Mean Velocity

d = Pipe diameter

k= Diameter of roughness grain

or simply by comparing pressure drop data from fluid flow experiments to the Moody Chart and solving Colebrook-White equation for obtained friction factor.

1.2Status quo

Many researchers had tried to evaluate validity of Nikuradse work and find shortcomings; nevertheless Nikuradse sand-grain roughness experiments have been the main reference on flow in rough pipes for over 60 years (Sletfjerding&Gudmundsson 2003). At the time of inception itself the Colebrook-White equation was in error by 3-5%, when compared with actual experimental data (Bhave2006). Moreover, several researchers have found that the Colebrook–White equation is inadequate for pipes smaller than 2.5 mm. In recent years the use of the Colebrook–White equation for the computation of commercial pipe friction factors, particularly for small-diameter pipes was discouraged by several researchers (Yoo& Singh 2005). Instead they suggested using a power law with minor modifications. But still usage of the formula is well prevalent as standard practice and recently Mcgovern (2011) had updated the

Moody's chart in vector graphics version. However, significant breakthrough in finding shortcomings of these standard works had been achieved by Bernuth& Wilson (1989) and Kandlikar(2005). Bernuth& Wilson showed that Blasius equation and not Colebrook-White equation is an accurate predictor of Darcy-Weisbach friction factor for small diameter plastic pipes for Reynolds numbers less than 1,00,000. Kandlikar (2005) highlighted further need for research to establish perfect understanding of the micro scale flow geometries. Taylor et al., (2006) has summarized these developments in chronological order and predicted future path. A whole gamut of new formulas had been proposed by Achanta et al. (2000), Yoo& Singh (2004), Yoo& Singh (2005), Sonnad&Goudar, (2006), Winning &Coole (2009), Yang & Joseph (2009), Ghanbari et al. (2011), Li et al. (2011), Dobrnjac (2012), Babajimopoulos&Terzidis (2013) and so on. These formulas have resulted from regression analysis of results available till date from published studies and minor modifications in patterns of equation without major consideration to fundamental change in parameter definition or relationship.

There are several other approaches to solve pipe flow problems e.g. Ahsan (2014) had used Computational Fluid Dynamics(CFD) modeling for numerical analysis of turbulent flow. But significantly new approach has been proposed by Farshadet. al (2005) and (Afzal, 2007a & 2007b). Taylor et al.(2006) had also introduced six new terms to describe surface roughness to be used for higher relative roughness in micro tubes and micro channels.

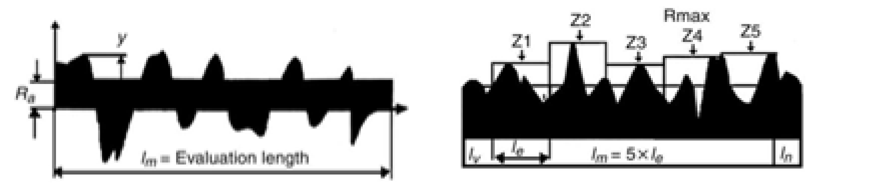
The work of Sletfjerding&Gudmundsson (2003) was pioneer to develop a method of estimating the friction factor in pipes directly from measured values of the wall roughness. They studied correlation between various measured surface roughness parameters and measured friction factor. This was the research gap existing since times of Moody and Colebrook. Farshad et al. (2005) used mean-peak-to-valley- height Rz in place of equivalent surface roughness in Colebrook-White equation to calculate friction factor. He preferred Rz over Ra(average arithmetic roughness) because of better representation of absolute roughness achieved by Rz in their other research work (Farshad&Pesacreta 2003). The measured value of surface roughness is 0.033 mm for cement liningusing direct surface roughness measurement technique as demonstrated by Farshad et al.(2005). They had also measured and tabulated surface roughness for other latest piping materials and presented new roughness chart. However, the exact mechanism behind the turbulence and the role of distribution of protuberances was not discussed.

The work of Afzal (2007a & 2007b) gave in depth consideration to this problem and a new alternate scale for better understanding and representation of surface roughness had been proposed.

The relevancy of direct roughness measurement parameters with all major available datasets obtained from experiments had been evaluated and the correlations of the alternate roughness parameters like Ra, Rz etc. and equivalent sand grain roughness were documented. The relationship between equivalent sand grain roughnesse with Rz was obtained for Sletfjerding&Gudmundsson (2003) data of artificially roughened pipes with glass beads as e= 1.244 Rz (Afzal 2007b). Adams et. al (2012) had developed a simple algorithm with which various measured surface roughness parameters can be converted to equivalent sand-grain roughness. For equivalent sand grain roughness and measured Rz the relationship e= 0.978 Rz was obtained by conducting experiments on Copper, Aluminum, GI and Steel tubes. Thus, strong evidence had been observed supporting direct measurement of roughness in commercial pipe to evaluate friction factor. Looking to these correlations, for drinking water pipes the direct measurement of roughness without conversion seems convenient. In future with more sophistication and establishment of (Afzal 2007b) theory further micro flow geometry can be studied. In present study the cement mortar lined DI pipes are focused for roughness measurement due to their universal adoptability in water supply projects.

2. Experimental program

Despite its practical relevance, until now, little work has been published on the measured surface roughness of new material pipes and internally coated pipes. Further, work of Farshad & Pesacreta (2003) had proved superiority of surface profiler to study 'roughness' or 'smoothness' of pipe material compared to AFM. Stochastic evaluations are performed on the profile data to develop relative-roughness charts and equations. Arithmetic roughness, Ra, is defined as the arithmetic average value of the roughness profile determined from the deviations about the centerline within the evaluation length, lm. The mean peak-to-valley height, Rz, is the arithmetic average of the maximum peak-to-valley height of roughness values within five consecutive sampling lengths, le, over the assessment length of the roughness profile.



(a) Arithmetic roughness, R_a (b) Mean peak-to-valley height, R_z

Figure 1Graphical representation of surface roughness parameters (Farshad et al. 2005)

The data from direct profilometer measurements are used to solve the Churchill's equation for the Darcy friction factor, *f,* which is a dimensionless correlating function.

Absolute roughness and relative roughness are two terms used in fluid mechanics to describe solid surfaces. Absolute roughness, e, is a measure of the distance from the base of an individual roughness projection to its peak. Surfaces with a standardized absolute roughness, e, have been made from uniformly-sized sand grains. Figure 2 illustrates such a surface, where Ks, is the roughness height of a monodispersed sand. This sand grain height, Ks, is assumed to act as the absolute roughness e, for that surface. For pipes having non-uniform roughness, equivalent surface roughness e can be regarded as being the roughness height of a sanded surface giving the same resistance-coefficient as the non-uniformly roughened surface.

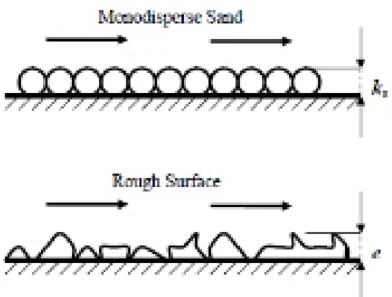


Figure 2 Ideal Roughness Height, Ks, of Monodisperse Sand Contrasted with Rough Surface of Absolute Roughness, e (Schlichting 2000)

To calculate the friction factor using profilometer measurements, the statistically determined Rz and Ra values are substituted for the equivalent roughness in Churchill's equation. The sample for different diameter DI pipes concrete linings were collected and surface roughness was measured with help of TR100 Surface Roughness Tester at Mechanical Engineering Department laboratory at Parul Institute of Engineering and Technology. The tracing length of 6 mm was followed and several results on same specimen were taken. The instrument used for measurement is shown in Figure 3.

Figure 3 A sample roughness being measured by TR100 Surface Roughness Tester at Mechanical Engineering Department Laboratory of Parul Institute of Engineering and Technology.

The outliers from the measurement data were omitted for getting final average value of surfaceroughness for concrete lining samples. The samples were arranged and hold in position on thermocol sheetby pushing the sample in thermocol and then inserting pins around it as shown in Fig. 3

This was required to hold the samples in position without any shift in position during the movement of stylus on sample.

Farshad et al.(2005) acknowledged the work of Southwest Research Institute (SRI) regarding friction factor measurement and validation. Results of their measured friction-factor/surface-roughness values are in good agreement with Rz values from profilometer measurements. Again, their research indicated that Rz values better represent absolute roughness than Ra values. The fact was also endorsed by (Adams et al. 2012). Surprisingly, the reason behind superior presentation of roughness by Rz over Ra had been predicted by Colebrook & White (1937) way back in the form of the fact that higher protuberances will shield smaller ones as clearly demonstrated in experiments with selectively roughened pipes.

That is why Rz values have been considered in calculation of friction factor by Churchill's equation. As Churchill's equation covers all the flow regimes with an average accuracy of ±0.45% it was decided to be used in current study. The superiority of this equation is discussed by (Shete 2000) to determine Darcy friction factor.

3. Analysis, Results & Conclusions

The results of the experimental and analytical work are presented in Table 1. Internal lining samples from cement mortar lined DI pipes of different diameters were obtained and surface roughness measurement was carried out in laboratory as explained above. Surface roughness parameter Rz found to be varying between 0.0037 mm to 0.0378 mm as shown in Table 1. The Re is calculated using limiting flow obtained from limiting velocity considered for that particular diameter. The range of Re is found to be Maximum 1900986 to minimum value of 401998. The equivalent surface roughness value 'e' used in Churchill's equation to find out relative roughness e / D is replaced with Rz (mean peak-to-valley height). Friction factor f is calculated using these values of Re and Rz / D.

The maximum and minimum values of Rz / D for plotting graphs are adopted considering extreme combinations of Rz and pipe diameter D, i.e. to get limiting value, maximum Rz (0.0378 mm) is considered with minimum diameter (200 mm) and to get minimum value of Rz / D, minimum value of Rz (0.0037 mm) is considered with maximum diameter (600 mm). The maximum and minimum values of Rz / D obtained are 0.0001890 and 0.0000062 respectively, which covers all possible values possible for present study.The rounded off minimum andmaximumRz / D values are0.000005 and 0.00025.Fig. 4 depicts the variations in the friction factors for cement mortar lined Ductile Iron pipe calculated by Churchill's equation and by Regression analysis for different Rz / D values with Reynolds number.The values of friction factors obtained for each Rz / D values starting from 0.000005 up to 0.00025 are plotted against Reynolds number from4000 to 1.901 x 106.The initial two increments were kept of 0.000015 and all remaining increments were 0.000025 till value of 0.00025 was reached.

Table 1 Values of Average Arithmetic Surface Roughness Ra and Average Mean Peak to Valley Height Rz for different DI pipes (K7; IS 8329: 2000)

Sr. No.	Nominal Dia. of Pipe in mm	Thickness of lining observed at site in mm	Thickness of pipe material observed at site in mm	Average Value of Arithmetic Roughness R_a in mm	Average Mean Peak to Valley Height R_z in mm	Limiting Velocity in m/sec	Limiting Flow in m^3/sec	Limiting R_eR_e = 4Q/πDv Where, v=0.801x10^{-6} m^2/sat 30C	R_z/D
1	200	4	5	0.0008	0.0049	1.75	0.047	401998	0.0000245
2	250	4	4	0.0055	0.0347	2	0.086	582772	0.0001387
3	300	4	5	0.0050	0.0378	2.65	0.166	935605	0.0001260
4	350	4	5	0.0048	0.0188	2.65	0.224	1085144	0.0000537
5	400	4	5	0.0012	0.0064	2.65	0.296	1248577	0.0000159
6	450	5	6.5	0.0008	0.0037	2.65	0.379	1412010	0.0000082
7	500	5	7	0.0029	0.0187	2.65	0.471	1574782	0.0000374
8	550	5	7	0.0022	0.0085	2.65	0.573	1735568	0.0000155
9	600	5	7	0.0008	0.0040	2.65	0.687	1900986	0.0000067

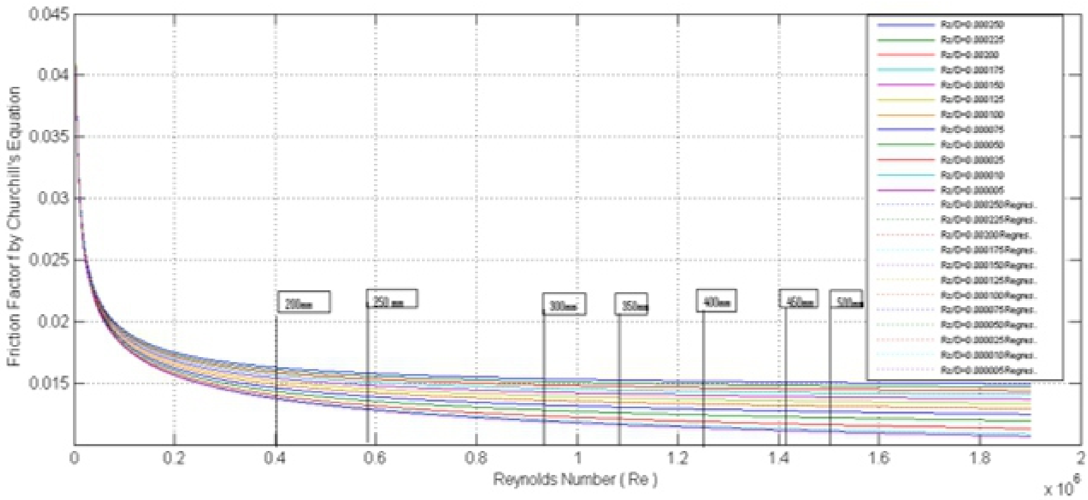


Figure 4 Variations in friction factor f obtained by Churchill's equation for different values of Rz/ D for cement mortar lined DI pipes and Reynolds No.

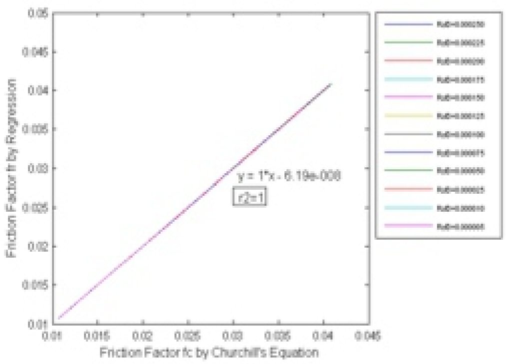


Figure5 Relation among friction factors for cement mortar lined DI pipes obtained by Churchill's equation fc and regression Analysis f_r

Table 2 Regression equations for Rz/D= 0.000225 (Such equations have been obtained for all values of Rz/D considered)

R_z/D	R_e Range		Model for determination of head loss	R^2
	From	To		
0.000 25	400	2200	*f* = -1.02415E-32*(Re)^10+1.28378E-28*(Re)^9+-6.85584E-25*(Re)^8+2.00387E-21*(Re)^7+-3.34433E-18*(Re)^6 +2.69189E-15*(Re)^5+5.96200E-13 *(Re)^4+-3.77068E-09*(Re)^3 +3.99801E-6*(Re)^2+-2.12207E-03*(Re)^1 +0.578533715	1
	2201	3900	*f* = -8.08942E-29*(Re)^9+2.16713E-24*(Re)^8+-2.56499E-20*(Re)^7+1.76030E-16*(Re)^6 +-7.71914E-13*(Re)^5+2.24302E-09*(Re)^4 +-4.31932E-06*(Re)^3+5.31600E-03*(Re)^2 +-3.795248093*(Re)^1+ 1197.882157	1
	3901	18000	*f* = 3.55368E-44*(Re)^10+-6.13006E-39*(Re)^9+-4.41688E-34*(Re)^8+-1.79878E-29*(Re)^7+4.66039E-25*(Re)^6 +-8.12937E-21*(Re)^5+9.80034E-17*(Re)^4 +-8.21931E-13*(Re)^3+4.75768E-09*(Re)^2 +-1.88363E-5*(Re)^1+7.42195E-02	0.99
	18001	62000	*f* = 0.23804+-9.94842/LN(Re) + 170.33820/LN(Re)^2+ -1437.28083/LN(Re)^3+6228.55437/LN(Re)^4 +-10635.95317/LN(Re)^5	0.99
	62001	133000	*f* = -0.11517+9.66290/LN(Re) + -265.35480/LN(Re)^2+ -3404.10308/LN(Re)^3+-20675.86582/LN(Re)^4 +49201.95323/LN(Re)^5	1
	133001	500000	*f*= -0.59959+38.31512/LN((Re))+-943.43556/LN((Re))^2 +11430.15941/LN((Re))^3+-68192.78475/LN((Re))^4 +161747.29668/LN((Re))^5	1
	500000	1901000	*f*= -2.02754E-02+5.93050E-02*LN((Re))+-1.33303E-02 *LN((Re))^2 +1.20177E-03*LN((Re))^3+-4.92676E-05*LN((Re))^4+7.67252E-07*LN((Re))^5	1

As shown in Fig. 5 the relationship between the friction factors obtained by regression analysisand Churchill's equation is given by fr = fc- 0.0000000618 and the coefficient of correlation R2 between them is1. The technique used to develop relative-roughness chart (Fig. 4) and regression equations(Table 2)can be used to establish the effect of roughness in the hydrodynamics of fluid flow in new alloys, synthetic materials and internally coated pipes very accurately.The effect of roughness will become more significant with reduced pipe diameters. Accurate determination of actual surface roughness can lead to better estimation of friction factor for small diameter pipes. The head loss for 80 mm dia. DI pipe with roughness 0.035mm is 2.6m for 1000m while it is only 0.6m for 250mm dia. pipe. More micro scale studies with innovative approach are welcome as more and more accuracy in determination of friction factor is desired for small diameter pipes or high value of relative roughness.The results obtained and considered for plotting fVs Re curves for different diameter cement mortar lined Ductile Iron pipes are giving better representation of actual conditions and exact determination of friction factor is possible accurately with use of the graphs or regression equations produced.

Acknowledgements

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Rural Landscape Planning And Design; The Potential Of Community Squares In Emerging Owerri Capital Territory, IMO State, Nigeria

Arc. Agoha, Basil Onyekozuru, MNIA,PhD
Department Of Architecture, Faculty of Environmental Sciences, Chukwuemeka Odumegwu Ojukwu University, Uli campus, Anambra State, Nigeria.

ABSTRACT

While attention on research and development are focused on the towns, not much is done on the rural areas and their landscape. In the rural landscape and planning, community squares are the main organizing elements which direct the spatial organization of physical infrastructure. Many a time when the rural areas transform into semi-urban/suburban areas into urban areas, they are neglected in the emerging urban area, at times decimated to the detriment of the socio-cultural, socio-economic and environmental quality of the built environment which they promote. As community squares are not only informal public open spaces which evolved with the people but have survived over the years and are transmitted from generation to generation they are environmental resources and heritage. This paper examinessome randomly sampledcommunity squares of the rural, suburban/semi-urban landscape of the emerging Owerri Capital Territory and develops a framework for the integration ofcommunity squares into the emerging Owerri Capital Territory. Environmental indicators that sustain the community squares were identified using relevant literature, visits, Geographic Information System,(GIS)Geographic Positioning System,(GPS) to collect data. The analysis was done with Statistical Programme for Social Sciences,(SPSS) using Analysis of Variance, (ANOVA), and Pearson correlation tools. The result showed that community squares as organizing elements in planning and design of rural landscapes have varying conditions and significant environmental values that if properly integrated, could significantly improve the quality of life in the emerging Owerri Capital Territory in particular and urban areas in general.

Keywords: Community squares, environmental, rural, socio-cultural, socio-economic

1. Introduction

Squares as organising units of traditional settlements exist at kindred, village and community levels in a typical Igbo built environment where they function as central activity areas. At the community level, they are called community squares and pervade the rural, sub-urban and semi-urban areas of the built environment. They are informal public open spaces which are environmental resources and goods that evolved, survived and developed by the people themselves over the years. They are therefore tied to the very life, of the people inalienable heritage to be protected, hence their continued survival and resilience. They form important physical, sociocultural nodes and landmarks in the community

(Lynch,1986).The adaptive, survival nature and lifecycle of community squares have made them very dynamic and sustainable to different demands and needs of succeeding generations of the society where they are located (Gunderson and Holling, 2002, Mustafa,1979, Walker, and Salt,2006). Attempts therefore to decimate them through formal planning and design are normally resisted, at times leading to physical and psychological conflict.

2. Literature Review

The Igbo Planning Philosophy

The Igbo counting philosophy based on four,(4), or quadruplet finds itself in rural planning, community life and community squares. Community squares are the basic organizing unit in traditional planning which makes planning flexible and shows in four-sided forms upon which other geometric forms, such as T, Y-junction, circle are built. The geometrical forms are created by network of foot paths, local rods, hierarchy of open spaces and tracts pervadingthe community squares,(Plates,11.11.2). These provide opportunity for axial, radial, concentric and triangular forms along which buildings and other structures are built. The openness provides not only flexible platforms for planning and design but gives limitless opportunity for physical planning.

The Igbo philosophy, dialectics, ontology, thought or principle does not shut down like the 'therefore' of Greek syllogism, they open limitless time on the transcendent, like the Arab dialectics, ontology and thought,(Oguejiofor and Onah, 2005). The geometric forms created are not rigid but organic and humane. There lies the origin of the openness of the Igbos reflected in unending family ties, gregarious lifestyle, quest for expansion and increase that builds neighbourhood, friendship, understanding and relationship ingredients needed for peace, harmony with man, nature and green environment. This philosophy implies the equality of villages in Igboland and signals qualification for creation of new villages, settlements, spaces or expansion along blood family lines and eventually leads to the growth and expansion of communities and traditional towns. Consequently, most road networks, markets squares, meeting places and social functions are located and organized based on this principle. It is interesting to note that a kindred or neighbourhood is normally made of households which are built from various family units. In this research this geometric philosophy of four, 4, as the basis of space development, community growth, organizing pattern and development of human settlements in traditional communities

Theoretical Framework

This cosmological development of settlements based on quadrants has also been identified in ancient Roman cities (Anthony and James,1979). Other elements and derivatives of this principle of geometric squares in community settlements are seen in form of cross roads, T-junctions, Y-junctions, triangles and other derived geometrical forms which richly adorn nodes, landmarks, and images in traditional Igbo communities. Within the community squares exist radial, concentric and axial concepts of planning. They form pedestrianised communication networks and pathways which are typical of Igbo traditional environment. These not only provide organizing systems but create open-ended and mobile city systems as identified in the American traditional public open spaces which transformed into modern open-ended American cities, (Rapoport,2005).Theoretically, the concepts and principles embedded in the lifecycle of these community squares no doubt reinforce the universality, multicultural, multidisciplinary nature of common planning and urban design principles, theories and concepts with apparent differences only highlighting peculiar geographic and cultural needs of the people to solve their peculiar environmental problems as they arose over the years.

Empirical Framework

Building on the geometry of planning by the Summerians and Egyptians, Hippodamus in the 5th century, developed the first philosophical environmental planning of cities by theorizing the grid iron system, (Anthony and James, 1979; Sporre,2000).Environmental planning was used as a means of creating environmental order in form of gardens, parks and squares. Thus community squares were properly identified, designated and upgraded into gardens, recreational parks, activity or civic centres in emerging urban areas. This was not the case in Owerri capital territory and it deprived the people of the positive impacts of community squares on the quality of life in urban areas, value of real estates, and settlements, (Crompton, 2007). Before the colonial era, in Northern, Urguhart,(1977),Western and Eastern community squares were used as the organising elements for landscape planning and design.The non- integration and abuse of community squares in cities in formal urban planning not only affected settlement patterns but lead to loss of environmental heritage and defaced the cityscape in the urban areas. Hence real or perceived resistance to urban planning, location of formal public open spaces outside the community squares manifested in environmental problems such as urban sprawl, defacing of the cityscape, slum development, squatter settlement, street trading, blocking of streets during family or community functions evident in most cities.

3. Aim and Objectives

Aim

The aim of this research is to identify landscape design and planning forms in the community squares in Owerri capital territory so as to provide a framework for their integration into urban milieu, for improved urban quality and cityscape.

Objectives of the Study

The objectives of the study are to;

i). Examine the conditions of community squares in Owerri capital territory.

ii). Establish the relationship of the conditions of community squares and environmental planning indicators in the community squares in Owerri capital territory.

4. Research Questions

This research attempted to answer the following questions;

i). What are the conditions of community squares in Owerri capital territory

ii).How is the condition of community squares related to environmental planning indicators in community squares in Owerri capital territory?

5. Hypotheses

i). Ho1: There are no significant difference in the conditions of the community squares in Owerri capital territory

ii). **Ho2:** There is no significant relationship between the condition of community squares and environmental planning of indicators in the community squares in Owerri capital territory

6. Study Area

Owerri capital territory covered seven (7) local government areas of Owerri Municipal, Owerri West, Mbaitoli, Ikeduru, parts of Ohaji/Egbema and AbohMbaise local government areas. The study was done for communities squares within twenty (20) kilometer radius from AmaJK/ Owerri general post office covering 1257 square kilometer, (Fingerhut and Partners,1978, Imo State Government,1978). The core urban, suburban and rural areas were defined by 5,10 and 20 kilometer radii of the concentric circles respectively (Fingerhut and Partners,1978). Its external tangent was defined as follows; along

Owerri – Onitsha road axis at Ukwu-Oji in Umunoha, in Mbaitoli local government area, along Owerri-Okigwe road axis at Atta inIkeduru local government area, along Owerri - Umuahia road axis around Enyiogugu in AbohMbaise local government area, along Owerri- Aba road around Agbala in NgorOkpala local government area, and along Owerri- PortHarcourt road axis around Obinze in Owerri West local government area. This encompassed the core Owerri urban, suburban and rural components that would form the future Owerri megacity of the Owerri capital territory and provided the platform to study the continuum transformation of community squares as the status of the places changed from rural through suburban to urban areas.

7. Scope Of The Study

The community squares were at various levels of condition and transformation depending on whether in urban, suburban and rural areas of the capital territory. It was therefore relevant to study the level of transformation and preservation of the community squares for comparative purposes. Although each community had many villages or kindred with smaller squares, there was always the central community square where the whole community gathered for their communal activities. The planning of Owerri capital territory was based on the concentric circle theory and conical growth pattern spreading from the centre outwards (Rodrigue,2015, Imo State Government,1978). This implied that Owerri Capital Territory has the urban core, the sub-urban/semi-urban and rural areas, providing opportunity for dynamic development to respond to observed inadequacies in the development of the capital territory. The research ensured the spread of the studied community squares into urban, sub-urban/semi-urban and rural areas.

8. Methodology

Through multilevel probability sampling the number of local government areas was reduced from 7 to 4 and the community squares from 49 to 13.The selected community squares were properly identified on ground for study with Geographic Positioning System, (GPS), Geographic Information System satellite images, (GIS), to see the transformation and use over the years. Three hundred and ninety, (390), copies of questionnaires were administered to measure the general perception of the people on and use of community squares out of which three hundred and fifty,(350) copies were retrieved, collated and analysed. The indicators studied were the location of community squares, hierarchy of open spaces in the community squares and network of footpaths in the community squares. The analysis was done using analysis of variance,(ANOVA),and Pearson correlation analysis statistical tools.

9. Analysis and Discussion

In objective 1; to identify differences in the conditions of community squares in Owerri capital territory, the analysis of variance, ANOVA, analysis was used to test the hypothesis on this and the test showed except for centrality of location, there were significant differences in the conditions of the community squares at 95% confidence level and so the hypothesis was rejected and the alternative hypothesis that there was significant differences was accepted.

The post hoc test also showed that differences in the conditions of the community squares existed in and betweenthe rural, sub-urban and semi-urban areas of the capital territory existed, Table: 11.1.

Table:11.1: ANOVA results on the differences in the conditions of community squares in Owerri Capital Territory

		Sum of Squares	df	Mean Square	F	Sig.
Centrality of community square	Between Groups	.842	2	.421	1.407	.247
	Within Groups	82.869	277	.299		
	Total	83.711	279			
No. of footpaths	Between Groups	26.272	2	13.136	25.851	.000
	Within Groups	169.716	334	.508		
	Total	195.988	336			
Network of footpaths	Between Groups	.991	2	.496	2.904	.056
	Within Groups	55.107	323	.171		
	Total	56.098	325			
Hierarchy of open spaces	Between Groups	12.918	2	6.459	9.525	.000
	Within Groups	221.058	326	.678		
	Total	233.976	328			

Source: Updated, Agoha,2015

i).Number of Footpaths crossing community squares

Foot paths as strong indicators created nodes and landmarks for rural landscape planning and design.

With 53.8% having between 3 to 4 number of footpaths and 35.8% having more than 5 number of foot paths in the community squares, the community squares had pedestrian footpaths with planning nodes, Table 11.2,11.3 for individual community squares.

Table 11.2: Aggregated Number of footpaths crossing community squares

Value	%	**Cumulative % frequency**
(1-2)	10.4	10.4
(3-4)	53.8	64.2
(>5)	35.8	100.0
Total	100.0	

Source: (Fieldwork, 2015)

ii).Location of community squares

Most of the community squares in Owerri capital territory were located in suburban, rural and semi-urban areas of Owerri capital territory in the proportion of 39.8%, 29.5% and 26.1% respectively while core urban area was 4.6%, Table 11.4, and Plate11.1

Table 11.4. Aggregated Location of community squares

Value label	%	Cumulative %frequency
Suburban	39.8	39.8
Semi-Urban	26.1	65.9
urban area	4.6	70.6
Rural areas	29.5	100
Total	100.0	

Source: Agoha, 2015

Plate 11.1: AmaochaAfara community square;a typical rural area

Source: Agoha, 2015

Plate 11.2: AmakohaUbi community square;a typical semi-urban area

Source: Agoha, 2015

iii). Hierarchy of open spaces in the community squares:

Most of the community squares in Owerri capital territory had hierarchy of open spaces (79.8%) created organically over the years as a result of usage by the people, Table 11.6. The network of footpaths, roads within the community squares and around the various components of the community squares created hierarchy of open spaces for various uses such as school, church, market, meeting places, play areas, workshops and variety of outdoor events or activities. Tree canopies and grasses added a lot of interests to the hierarchy of the open spaces.

Table 11.6: Aggregated Hierarchy of open spaces in the community squares

Value Label	%	Cumulative % frequency
Disagree	13.1	13.1
Fairly agree	7.1	20.2
Agree	79.8	100
Total	100.0	

Source: Agoha, 2015

ii). Objective 2; to establish the effect of environmental planning and design indicators of community squares in Owerri capital territory. The result of the test showed there was negative significant relationship between the condition of the community squares and the hierarchy of open spaces.

Table 4.1: Result of Pearson correlation bivariate analysis of the relationship between condition of community squares and Hierarchy of open spaces

		Hierarchy of open spaces	Remarks
Condition of community squares	Pearson correlation coefficient	-171	Not significant at 0.05 confidence level
	sig.(2-tailed)	0.002	
	N	345	

Source; Updated from Agoha, 2015)

Conclusions and Recommendations

I). The significant differences in the conditions of the community squares provide opportunity for organic rural landscape planning and design and preservation of the elements such as the green and forest areas, nodes and land mark features the would help sustainability and improve the quality of life in the emerging Owerri capital territory. The government with the communities and all stakeholders should acquire the community squares and designate them government reservation areas awaiting comprehensive design and development.

ii). Identification of environmental planning indicators

The negative significant relationship of the condition and hierarchy of open spaces showed that the more the condition of the community squares deteriorate the less the existence of good hierarchy of open spaces. The government acquisition of the community squares should be followed with legislation that prohibit actions such as refuse dump, encroachment that lead to the destruction of the community squares. This should be followed by actions on the development of the community squares into public parks and recreation.

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