

STRUCTURE OF TURBULENCE IN A NON-UNIFORM SAND BED CHANNEL WITH EMPHASIS ON THE BEDLOAD TRANSPORT

A Thesis submitted

In partial fulfillment of the requirements for the degree of

Doctor of Philosophy

Submitted by

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I, Anurag Sharma, declare that this thesis “STRUCTURE OF TURBULENCE IN A NON-UNIFORM SAND BED CHANNEL WITH EMPHASIS ON THE BEDLOAD TRANSPORT” and the work presented in it are my own and have been generated by me as the result of my own original research

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CERTIFICATE

This is to certify that the thesis entitled **Structure of Turbulence in a Non-uniform Sand Bed Channel with Emphasis on the Bedload Transport** submitted by Anurag Sharma to the Department of Civil Engineering, Indian Institute of Technology Guwahati, is a record of bonafide research work under my supervision and is worthy of consideration for the award of the degree of Doctor of Philosophy of the Institute.

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Abstract

The present research work experimentally investigates the flow over non-uniform sand bed channel. Experimental data collected from a straight flume having a bed covered with non-uniform sand was used to analyze different flow characteristics for non-uniform sand bed channel. This study, using an experimental approach, focuses on the effect of flow on non-uniform sand bed channel morphology and corresponding turbulent flow characteristics. Experiments have been carried out in a non-uniform sand bed channel with two different sediment beds i.e., median diameter = 0.425 mm and 0.50 mm sands. An important parameter of sand bed channels, downward seepage was considered in the present work. Two different seepage percentages, 10% and 15%, were considered for exploring the change in flow characteristics with respect to no seepage case. Steady flows over non-uniform sand bed channel were simulated experimentally with downward seepage applied through the boundary. Measures of turbulent parameters such as velocity, Reynolds shear stresses, thickness of roughness sub layer and shear velocities were found to be increasing with seepage. The turbulent diffusivity and mixing length decrease in presence of seepage. The quadrant analysis suggests that the relative contributions of bursting events increased throughout the flow layer and the thickness of the zone of dominance of sweep event increases with seepage which is responsible for increment in bed material transport. The mean time of occurrence of ejections and sweeps in downward seepage are more persistent than those in no seepage. The analysis of third order moments states that an upward downstream flux of turbulent kinetic energy is observed over the near bed surface giving rise to sweeps with seepage. The turbulent kinetic energy fluxes increase in presence of seepage within the near bed flow. The increase in bed load transport with seepage associated with an increase in flow turbulence production corresponds with decrease in turbulent kinetic energy dissipation, pressure energy diffusion and turbulent diffusion. The traversing length decreases and size of eddies increases with seepage.

The current study experimentally investigates the flow characteristics and temporal variations in the sheet flow profile of a non-uniform sand bed channel. Experiments were done to explore turbulent structures in the presence of a sheet flow layer with and without seepage. The turbulent

events, such as stream wise velocity, Reynolds shear stresses, and turbulence intensities were found to be increasing and vertical velocity was found decreasing with a sheet layer. The presence of a sheet layer also effects the turbulent energy production and energy dissipation. All the turbulence parameters with and without a sheet layer have also been influenced by the presence of downward seepage. The rate of sheet flow movement is increased with seepage, owing to increased turbulence with seepage. The current study used wavelet analysis on temporally lagged spatial bed elevation profiles obtained from a set of laboratory experiments and synchronized the wavelet coefficients with bed elevation fluctuation at different spatial scales. A spatial cross correlation analysis at multiple scales, based on the wavelet coefficients, has been done on these bed elevation datasets to observe the effect of downward seepage on the dynamic behavior of sheet flow at different length scales. It is found that seepage increases average bed celerity and also increases the celerity of sheet flow of similar length scales. This increase in the celerity has been hypothesized as the increase of sheet flow movement as well as the increase in turbulent parameters with seepage, which destabilizes the bed particles resulting in a disruption in the continuous propagation pattern of the sheet flow. The increase of sheet flow celerity with seepage is confirmed from the saturation level of the wavelet power spectra of the bed elevation series. The presence of seepage also affects the non-uniformity of collective sheet material.

Flume experiments were carried out to investigate bedload transport in non- uniform sand bed channel for both no seepage and seepage flow. Measures of turbulent parameters such as velocity and Reynolds shear stresses were found increasing with seepage which leads to increase in bedload transport with seepage. An empirical model of bedload transport is proposed considering seepage as an explicit variable. The experimental data are also being used to evaluate the accuracy of existing bed load transport equations and has been found that Meyer Peter and Muller equation performs well for no seepage case, whereas none of the existing equations perform well for seepage case.

In this work, we have carried out an experimental study on the temporal changes of bed load transport rate in the presence of downward seepage. The present study used wavelet analysis on spatial bed elevation profiles obtained from a set of laboratory experiments and synchronized the

wavelet coefficients with bed elevation fluctuation at different spatial scales. A spatial cross correlation analysis at multiple scales, based on the wavelet coefficients, has been performed on these bed elevation datasets to observe the effect of downward seepage on the dynamic behaviour of sediment transport at different length scales. It is found that seepage increases average bed celerity and also increases the celerity of sediment transport of similar length scales. This increment in the celerity has been hypothesized as the increment of sediment transport with seepage, which destabilizes the bed particles resulting in a disruption in the continuous propagation pattern of the sediment transport. The increment of bed celerity with seepage is confirmed from the saturation level of the wavelet power spectra of bed elevation series. In the present study, we have observed that the bedload transport increased quickly with time for no seepage and seepage runs, and reached its maximum value, beyond which it gradually decreases till it becomes approximately constant. Based on the experimental data of no seepage and seepage runs for whole permeable sand bed, we have proposed the empirical expression of bedload transport rate as a function of time. The proposed equation also predicts time-series sediment transport in the rivers, helping river engineers formulate timely corrective strategies.

Keywords: Bedload, sheet layers, downward seepage, turbulent flow, wavelet, singular spectrum

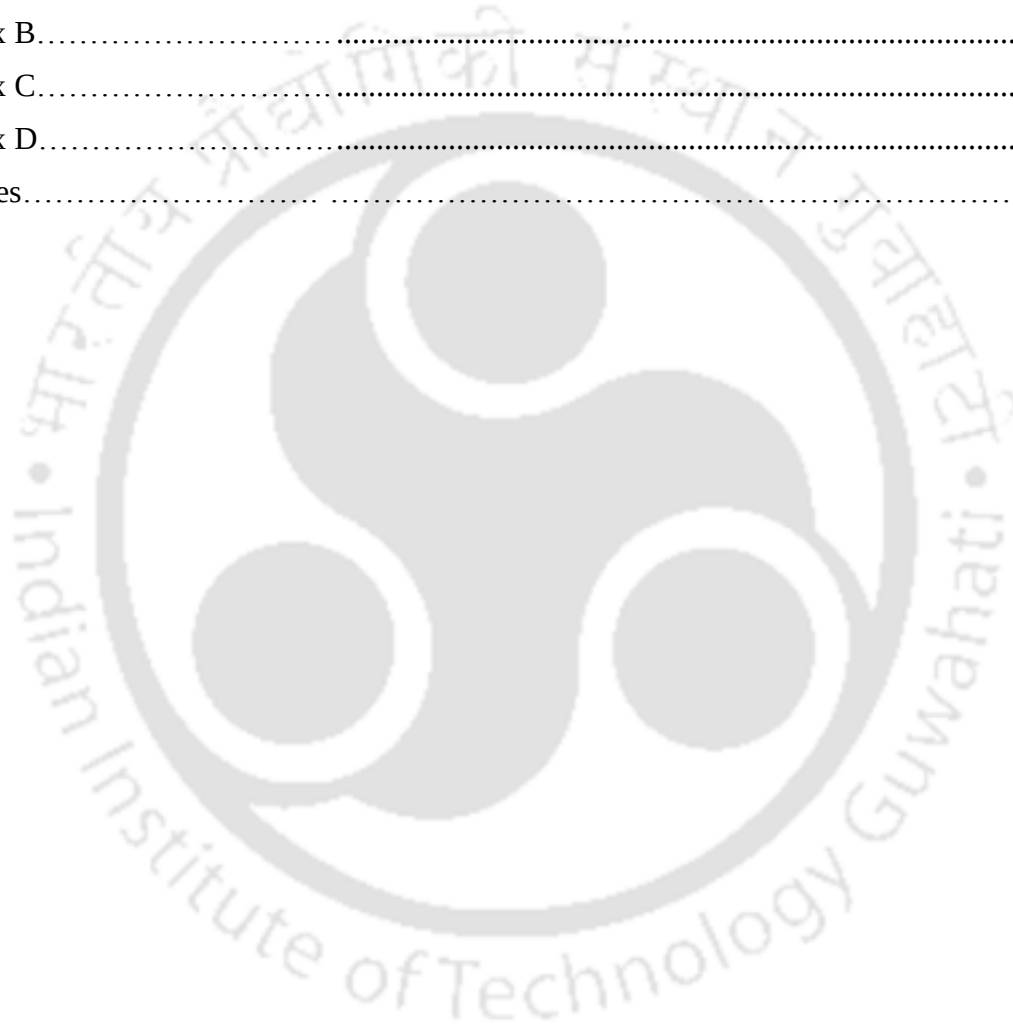
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List of Symbols

σ_g	Geometric standard deviation
d_{50}	Median particle diameter
d_{84}	Grain size for which 84% of the sample is finer by weight
d_{16}	Grain size for which 16% of the sample is finer by weight.
d_i	Geometric size of two consecutive size fractions
Δp_i	Percentage of material present in the sediment mixture.
M	Kramer uniformity coefficient
Q_0	Main Channel discharge
ϕ	Angle of repose (dry)
C_d	Coefficient of discharge
L_n	Length of the rectangular notch
H	Height of water above the rectangular notch
E	Voltage generated in a conductor
u_c	Velocity of the conductor
B_s	Strength of the magnetic field
D	Length of the conductor
A_p	Area of pipe in the flow meter
q_s	Seepage discharge
d	Distance from the probe to the reflecting surface
C_s	Celerity of the ultrasonic waves
h	Flow depth
S_u	Standard uncertainty
S_d	Standard deviation
u, v, w	Time-mean velocities in the streamwise, transverse, and vertical directions respectively.

U_i, V_i, W_i	Instantaneous velocities in the streamwise, transverse, and vertical directions respectively
u', v', w'	Fluctuating component of velocities in the streamwise, transverse, and vertical directions respectively.
F_{uu}	Streamwise velocity power spectra
f	Frequency of the signal
z	Distance of measurement point from the channel bed
u_s	Average flow velocity
u_*	Shear velocity
S_0	Bed slope of the channel
S_w	Water surface slope
S_f	Friction slope
A	Cross-sectional area of the channel
R_h	Hydraulic radius
F	Froude number
u_s	Average velocity of flow for seepage case
Q_s	Average discharge for seepage case
X	Distance from the tail gate
p	Perimeter
V_s	Seepage velocity through the sand bed of length L
τ	Bed shear stress
ρ_w and ρ	Density of the water and sediment respectively
γ	Specific weight of the fluid
k	Von karmann constant
z_0	Depth of the zero velocity level
Δz	Depth of the virtual bed level
K_s	Effective roughness height
L_f	Length of the flow developing zone
x	Longitudinal section starting from the upstream
Re	Flow Reynolds number

δ	Boundary layer thickness
u_{rms} and w_{rms}	Root mean square value of u' and w' respectively
n	Number of samples
τ_{uw}	Reynolds shear stress
τ_{turb} and τ_{visc}	Turbulent and viscous shear stress
ν	Kinematic viscosity of water
$z^+ = z/h$	Normalized depth of flow
D_u and C_u	Empirical parameters suggested by Nezu and Nakagawa (1993)
$SK(u_i)$	Coefficient of skewness of a velocity time series
ε^+	Normalized zero velocity level
σ_u	Streamwise turbulent intensity
σ_w	Vertical turbulent intensity
r_{uw}	Correlation coefficient
ε_f	Eddy viscosity
l	Prandtl mixing length
H	Hyperbolic hole region
$I_{i,H'}$	Indicator function
T	Total sampling time of velocity pulse
$S_{i,H'}$	Fractional contribution of Reynolds stress
$P_{i,H}$	The probability of occurrence of the bursting events
T_E and T_S	Normalised mean time of occurrence of ejection and sweep events respectively
M_{30} and M_{03}	Flux of Reynolds stress in streamwise and vertical directions
M_{12} and M_{21}	Advection of Reynolds stress in streamwise and vertical directions
K_u	Fourth order correlation

f_{ku} and f_{kw}	Flux of turbulent kinetic energy in the streamwise and vertical direction respectively
F_{ku} and F_{kw}	Normalized flux of turbulent kinetic energy in the streamwise and vertical direction respectively
ε	Energy dissipation
p'	Pressure fluctuations
T_P, T_D, P_D and E_D	Non dimensional turbulence production, turbulent diffusion, pressure energy diffusion and energy dissipation respectively
t_P, t_D, p_D , and v_D	Turbulence production, turbulent diffusion, pressure energy diffusion and viscous diffusion respectively
K	Turbulent kinetic energy
J	Wave number in the stream wise direction
$\theta(J)$	Non-dimensional spectral density of u'
C_K	Non-dimensional Kolmogorov constant
λt	Taylor micro scale
λ_d	Normalized Taylor micro scale
E_L	Eulerian integral length scale
E_T	Eulerian integral time scale
$R(t)$	Autocorrelation function
$\hat{u}, \hat{w}$	Normalized fluctuating component of velocities in streamwise and vertical direction
$P_{\hat{u}}(\hat{u})$ and $P_{\hat{w}}(\hat{w})$.	Probability density function (PDF) for stream wise and vertical velocity fluctuation
$f_{\hat{u}}(\hat{u})$ and $f_{\hat{w}}(\hat{w})$	Relative frequencies of the random variables $\hat{u}$ and $\hat{w}$
K_0 and K_1	Modified Bessel function of the second kind and orders 0 and 1 respectively.
$P_{\hat{\tau}}(\hat{\tau})$	Probability density function (PDF) for Reynolds shear stress

$f_{\hat{\tau}}(\hat{\tau})$	Relative frequencies of the random variables $\hat{u} \hat{w}$
$f_{\hat{\sigma}_u}(\hat{\sigma}_u)$ and $f_{\hat{\sigma}_w}(\hat{\sigma}_w)$	relative frequency of the random variables $\hat{u} \hat{u}$ and $\hat{w} \hat{w}$
$P_{\hat{\sigma}_u}(\hat{\sigma}_u)$ and $P_{\hat{\sigma}_w}(\hat{\sigma}_w)$	Probability functions for turbulent intensity in streamwise and vertical directions
g	Acceleration due to gravity
$\Psi_{a,b}(t)$	Wavelets
$W_f(a,b)$	Continuous wavelet transform of a signal $f(t)$
b	Location parameter
a	Scaling parameter
$g_2(t)$	Mexican hat wavelet
$S_x(f)$	Power spectral density function
$R_x(\tau)$	Auto-correlation function
q_b	Bedload transport rate
R_*	Friction Reynolds number
Rs	Seepage Reynolds number
t	Observation time
Φ	Non-dimensional bedload transport rate
θ	Shield's parameter
θ_c	Critical Shield's parameter
τ_c	Critical shear stress
q_{b-peak}	Peak value of volumetric sediment transport rate per unit width
Y	Dimensionless sediment transport rate
t_{off}	peak-value offset

1 Introduction

1.1 Background

All natural rivers have the characteristics of either erosion or deposition based on flow parameter and riverbeds. Flow of water in river channel is affected by river engineering works due to which sediment transport in channel is also affected. This change of sediment movement alters the shape and plan form of river channel. This alteration in the shape and plan form of a channel further affects the flow conditions in the channel, its conveyance and the composition of the bed sediments. Sediment characteristics of river channels have been approached from various directions in the past several decades. Riverbeds are usually composed of non-uniform sediment mixtures and the corresponding particle size distribution of sediment in transport is generally finer than the distribution of bed material because of selective transport. This makes the prediction of sediment load for natural rivers more difficult than for uniform sediment in laboratory flumes.

Fluvial processes include the sediment transport and aggradations or degradations of the riverbeds. The flow induces a shear stress on the bed formed by the loose sediment. From the experimental channels and natural river channels, it is observed that there is some threshold condition of flow at which sediment particle start to move. The threshold condition of flow is defined as bed shear stress and if bed shear stress is greater than stabilizing resistance to the sediment particles, the sediment can be mobilized. The specific velocity or bed shear stress at which the particles on the bed surface starts to move is called the threshold velocity or threshold shear stress. **Shields (1936)** in his pioneering study characterized this flow condition as the condition of incipient motion. This incipient motion condition has often been incorporated into the theories of sediment transport. The transport of sediment particles in a stream and the flow are closely related to each other. Sediment transport by the stream flows can occur in different modes. Sediment in rivers is transported as bed load, suspended load and wash load. Further,

during the sediment transport, the riverbed takes different bed features which affect the hydraulic resistance and the flow.

An alluvial channel is a water channel made up of loose sediments called alluvium. These alluvial channels are mostly used for irrigational and navigational purposes. Fluvial hydraulics is the branch of science which deals with the hydraulics of rivers owing through loose non-cohesive material called alluvium. The subject fluvial hydrodynamics is being important in the fields of civil engineering, environmental engineering, sedimentary geology, and earth sciences and also often used to know whether erosion or deposition of sediment or even transport of sediment can occur.

The sediment transport in rivers which affects the morphological condition of river is an important aspect in fluvial processes. The sediment transport often called load, refers to the quantity of sediment that is transported in a stream. Specifically, it is used to define the rate (volume or weight per unit time and width) at which the sediment is transported. When the bed shear stress induced by the flow exceeds the critical bed shear stress for the initiation of sediment motion, the sediment particles forming the bed are in mobile condition. The bed load transport is the mode of sediment transport where the sediment particles slide, roll, or saltation near the sand bed. Figure 1.1 presents a schematic of different modes of sediment transport.

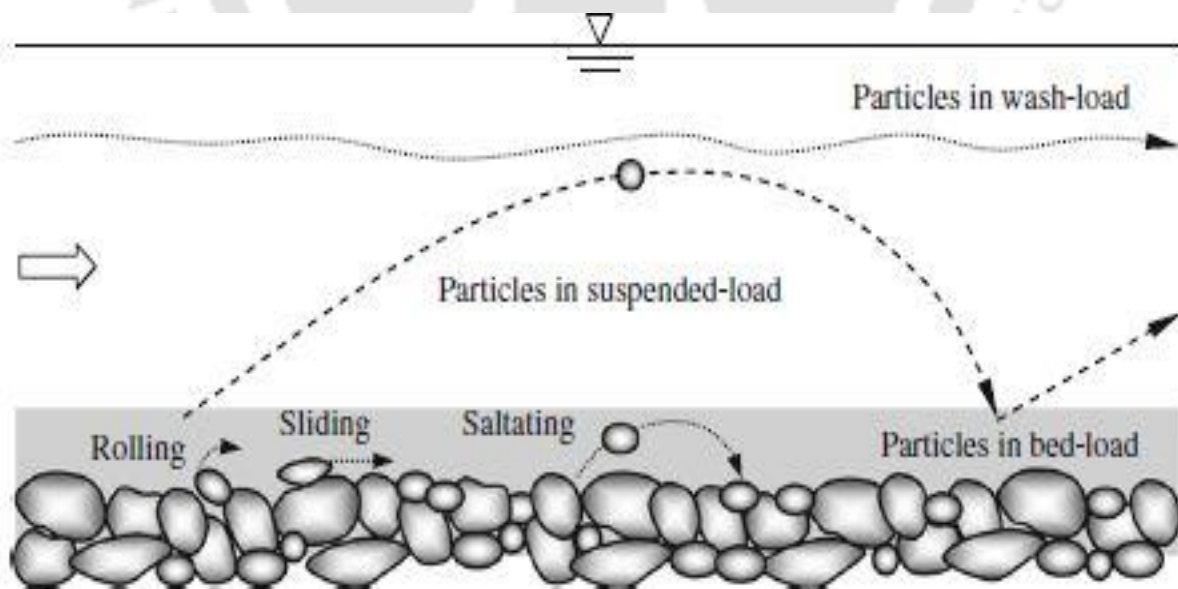


Figure 1.1: Different modes of sediment transport (Dey, 2014)

The transport rate of sediment mixtures is affected by the presence of the other sizes in the mixture hence the transport of sediment mixture in open channel is complicated. The finer fractions moving as bed load are sheltered by the coarse fractions while coarser one are exposed to the finer fraction under the action of flow. As a result, at a given shear stress, the force experienced by a certain fraction of sediment mixture is different from what it would be at the same shear stress if the material were uniform. This difference in forces is a function of the bed composition, fluid and flow characteristics. The size distribution of the transported material is dependent on the shear stress which is different from that of the bed material. Therefore, the bed load calculated using any methods based on a single representative size results in large error in such cases. It is thus for the accurate prediction of bed load transport rate and its composition, one should know transport rates of individual fractions of the bed material. There are several researchers who have given their contribution in calculation of bed load of sediment mixture.

1.2 Studies on Non-uniform Sediment

Einstein (1942, 1950) defined the bed-load transport that takes place within a thin layer having a thickness of two particle diameters above the bed by sliding, rolling, or traveling in succession of jumps with a stream wise distance of a few particle diameters. **Bagnold (1956)** defined the bed-load transport that takes place by successive contacts of the particles with the bed being limited by the gravity effect. With increase of bed shear stress, the turbulence production near the bed and its diffusion in upward direction lift up relatively finer sediment particles from the bed keeping them in suspension, as they are transported by the flow. The sediment transport in suspension mode is termed suspended load. **Bagnold (1956)** defined the suspended-load transport that takes place by balancing submerged weight of the particles with upward diffusion of turbulent eddies. Wash load in natural stream is that portion of sediment load that is carried by the flow such that it always remains close to the free surface.

Einstein (1942, 1950) presented the most valuable analysis on bed load transport, based on fluid mechanics and probability. **Einstein (1942)** obtained two dimensionless parameter defined as bed load intensity ϕ and the flow intensity Ψ respectively.

$$\phi = \frac{q_b}{\gamma_s} \left(\frac{\gamma}{\gamma_s - \gamma} \frac{1}{gd^3} \right)^{1/2} \quad (1.1)$$

$$\psi = \frac{(\gamma_s - \gamma)d}{\gamma R'S} \quad (1.2)$$

Where q_b is bed load intensity per unit width, d is the characteristics grain diameter, R' is the hydraulic radius with respect to grain roughness, S is bed slope, γ_s and γ are the specific weight of sediment and fluid respectively. The relationship $\Phi = F(\psi)$ was found based on empirical data.

Einstein (1950) studied the direct computation of transport rates by size fractions in which the bed-material load is calculated after the computation of transport capacities corresponding to each size group by the summation of Fractional sediment transport rates from $q_b = \sum_{i=1}^N q_{bi}$ where q_b = bed material load per unit width by dry weight; i = size fraction number; and N = number of size fractions present in the sediment mixture. He proposed a comprehensive hiding factor in his bed load function to account for the interaction effects between coarse and fine particles which are a function of d_i/d_x . Hiding factor is unity for uniform grains but its value increases for smaller particles in a mixture.

Egiazaroff (1965) developed threshold model for the entrainment of sediment particles which was examined using spherical grain geometry. As a result, priorities given to lift forces acting on the grain were found to be assumed in his sediment transport model. The theoretical values of Shields parameter by his equation are also shown to be considerably similar to the empirical values of Shields parameter derived from the analysis using field data for coarse bed materials, in the vicinity of uniform size ($d_i/d_m < 2.5$ where d_i is the diameter of an arbitrary grain in a non-uniform mixture and d_m is the size determining the channel roughness).

$$\frac{\theta_{ci}}{\theta_c} = \left[\frac{\log 19}{\log \left(\frac{19d_i}{d_m} \right)} \right]^2 \quad (1.3)$$

$$\theta_{ci} = \frac{\tau_{ci}}{(\gamma_s - \gamma)d_i} \quad (1.4)$$

Ashida and Michiue (1973) described the physical meaning of hiding and exposure in the transport of non-uniform sediment mixtures. These effects result in a discrepancy between the critical shear stresses for uniform sediments and non-uniform sediment. In their bed load transport equation for non-uniform mixtures, they modified **Egiazaroff's (1965)** expression for critical shear stress.

$$\left. \begin{aligned} \theta_{ci} \left(\frac{d_i}{d_{50}} \geq 0.4 \right) &= \theta_c \frac{\log^2 19}{\log^2 \left(\frac{19d_i}{d_{50}} \right)} \\ \theta_{ci} \left(\frac{d_i}{d_{50}} \leq 0.4 \right) &= \theta_c \frac{d_{50}}{d_i} \end{aligned} \right\} \quad (1.5)$$

Hayashi (1980) developed threshold model for the entrainment of sediment particles by considering the influence of sediment particle size.

$$\frac{\theta_{ci}}{\theta_c} = \left[\frac{\log 8}{\log \left(\frac{19d_i}{d_m} \right)} \right]^2 \quad \text{if } \frac{d_i}{d_m} \geq 1 \quad (1.6)$$

$$= \frac{d_m}{d_i} \quad \text{if } \frac{d_i}{d_m} < 1 \quad (1.7)$$

Qin (1980) proposed the formula for the threshold near-bed velocity u_{cri} of a size fraction d_i in a non-uniform sediment mixture.

$$u_{cri} = 0.786 \left(\frac{h}{d_{90}} \right)^{1/6} (\Delta g d_i)^{0.5} \left(1 + 2.5m \frac{d_{am}}{d_i} \right)^{0.5} \quad (1.8)$$

Where, $m(\eta_d < 2) = 0.6$, $m(\eta_d \geq 2) = 0.76059 - 0.68014(\eta_d + 2.2353)^{-1}$, $\eta_d = \frac{d_{60}}{d_{10}}$ and d_{am} is the arithmetic mean size of bed sediment.

Karim and Kennedy (1981) introduced the hiding factor W_i into the bed material fraction approach (BMF) to reflect the influence of other sediment particles in the mixture on the

transport of given size fraction i . They expressed W_i as a simple power function of D_{si} / D_{50} . BMF approach assume that the sediment transport rates by size fractions can be determined by multiplying the potential transport rate corresponding to a given size fraction with the percentage of material present in the bed material. After introduction of hiding factor, BMF approach can be expressed as

$$C_{ti} = W_i P_{bi} C_{pi}, C_t = \sum_{i=1}^N C_{ti} \quad (1.9)$$

Where C_{pi} = potential bed-material load concentration. In BMF concept, the potential concentration, C_{pi} , for a given size fraction i , is computed by bed-material load formulas by replacing the representative size with the average (or geometric mean) diameter, D_{si} , of the corresponding size fraction of the bed material.

Parker et al. (1982) developed an empirical gravel transport relationship based on the shear stress correction approach and the analysis of field data from Oak Creek. They suggested the following threshold for the incipient motion of non-uniform sediment.

$$W_{ei}^* = \frac{q_{bi} \left(\frac{\rho_s}{\rho} - 1 \right) g}{p_{bi} u_*^3} = 0.002 \quad (1.10)$$

where W_{ei}^* is a reference transport parameter; q_{bi} is the volumetric transport rate per unit width for the i th fraction of bed-load; ρ_s and ρ are the specific densities of sediment and fluid; g is the gravitational acceleration; P_{bi} is the gradation of the i th fraction of bed material; and u_* is the bed shear velocity.

White and Day (1982) investigated incipient motion conditions for individual grain sizes in a mixture using recirculating flume data. They predicted fractional transport rates by correcting the initial motion parameter used in **Ackers and White's (1973)** approach. The results contribute to a hiding function that is related to D_{si}/D_A where D_A is the scaling size in a mixture defined by them.

Proffitt and Sutherland (1983) investigated the effect of sediment non-uniformity by comparing the transport rates of individual fractions in a mixture with that of uniform material of

the same size. They considered **Paintal's (1971)** and **Ackers and White's (1973)** transport functions as the basis for studying the effect of non-uniformity and suggested corrections to be applied to these two functions. They expressed ξ_{bi} as a function of D_{si}/D_u by analysing experimental data. Proffitt and Sutherland's correction factor does not consider all the relevant parameters, and the development of relation for ξ_{bi} covers a relatively narrow range of flow conditions.

Van Rijn (1984) was investigating the motion of the bed-load particles and to establish simple expressions for the particle characteristics and transport rate both for small and large particles. Using the expressions for the saltation height (δ_b), the particle velocity (u_b) and bed-load concentration (c_b) resulting in bed load transport rate $q_b = u_b \delta_b c_b$. The proposed equations predict a reliable estimate of the bed-load transport in the particle range 200-2000 μm based on a verification study using 580 flume and field data, resulting in a score of 77% of the predicted bed-load transport rates in the range of 0.5-2.0 times the measured values.

Van Rijn (1984) proposed an empirical equation to estimate the representative diameter d_s for suspended sediment load. The equation was determined by trial and error to give the same value for the suspended load as that computed with Einstein's method. This equation is expressed as

$$\frac{d_s}{d_{50}} = 1 + 0.011(G - 1)(T - 25) \quad (1.11)$$

Where, T = Transport stage parameter defined by $T = (\tau' - \tau_c)/\tau_c$; G = Relative specific density of sediment; τ' and τ_c = grain shear stress and critical shear stress respectively.

In the methods for determining the incipient motion of non-uniform sediment developed by **Egiazaroff (1965)**, **Ashida and Michiue (1973)**, **Hayashi et al. (1980)**, **Parker et al. (1982)** and **Andrews (1983)**, as well as in the methods for the fractional sediment transport rate developed by **Proffitt and Sutherland (1983)**, **Bridge and Bennett (1992)**, **Fang and Yu (1998)** and **Karim (1998)**, the correction factors were related to bed-material size by

$$\eta_i = f_1 \left(\frac{d_i}{d_m} \text{ or } \frac{d_i}{d_{50}} \right) \quad (1.12)$$

Where, d_i is the diameter of the i th fraction of sediment; d_m and d_{50} are the arithmetic mean and 50% sieve diameters of bed materials.

Misri et al. (1984) study is mainly based on the modification of **Einstein (1950)** because Einstein's bed load function does not accurately predict the transport rates of individual fractions in a mixture and this is primarily because his correction for sheltering effect is at variance with experimental data. They assumed that the motion of fine particles is dominated by the lift force while the motion of coarse particles is by the drag force, and proposed a semi-theoretical hiding-exposure correction factor.

$$\xi_e = f\left(\frac{\tau'_0}{\Delta\gamma_s d_i}, \frac{\tau'_0}{\tau_{oc}}, M\right) \quad (1.13)$$

However, the correction factors given in Equation (1.12) only involve the non-dimensional grain size (d_i/d_m or d_i/d_{50}) and cannot effectively account for the influence of bed-material gradation. The correction factor proposed in Equation (1.13) introduces the Kramer's uniformity coefficient, but it is more complex and thus less attractive in applications. They developed the relation for the transport of individual fraction in the sediment mixture.

$$q_B = f\left(\frac{\tau'_0}{\Delta\gamma_s d_i}, \frac{\tau'_0}{\tau_{oc}}, \xi_e\right) \quad (1.14)$$

Samaga et al. (1986) pointed out the inadequacies of **Misri et al. (1984)** over a wide range of

$\frac{\tau'_0}{\tau_{oc}}$, therefore they conducted experiments on the bed load transport of non-uniform sediments

over a larger range of $\frac{\tau'_0}{\tau_{oc}}$. The measurements of the present study have confirmed the findings of

Misri et al. (1984) about the limitations of Einstein's method in calculating the transport rates of sediment mixtures. Modification of the approach of **Misri et al. (1984)** has been presented which has the merit of being based on both field and laboratory data. The relationship between the dimensionless grain shear stress and the dimensionless bed load transport rate developed by **Misri et al. (1984)**, for uniform sediments was made applicable to sediment mixtures by introducing a multiplying correction factor. The parameters affecting this correction factor were

identified and a relationship obtained for the same by analysis of an extreme amount of laboratory and river data. Use of this factor along with the transport law for uniform sediments enables determination of the rate of bed load transport of the individual fractions of a mixture.

Holly and Karim (1986) stated that the transport capacity fraction (TCF) approach for computation of fractional transport of sediment mixtures was developed through heuristic reasoning supported by data analysis of measured suspended sediment size distributions of the Missouri, the Niobrara, and the Middle Loup Rivers. The TCF approach first compute the total sediment concentration by the use of a bed-material load equation and then converted into fractional concentrations for size groups by a transport capacity distribution function.

$$C_{ti} = P_{ci} C_t, \sum_{i=1}^N P_{ci} = 1 \quad (1.15)$$

The method was developed basically for relatively uniform 0.18-0.4 mm sediments, and was reported to yield reasonably accurate predictions for suspended sediment transport. The disadvantage of this method is that it was basically developed for suspended loads and it did not take into account the influence of the bed material size and hydraulic conditions.

Dou et al. (1987) suggested a simple method for the calculation of suspended load using transport capacity fraction (TCF) approach by relating P_{ci} to P_{si}/ω_i where P_{si} is the fraction of the suspended load by dry weight corresponding to size fraction i , and ω_i is the fall velocity of sediment corresponding to particle size D_{si} .

Diplas (1987) reanalysed Parker et al.'s data stating that Parker et al.'s approach constitutes only a first-order approximation of reality. He observed the hiding function is not only dependent on grain size ratio, D_{si}/D_{50} , but also dependent on average shear stress.

Hsu and Holly (1992) have pointed out the shortcoming of using bed material fraction approach in non-uniform sediment transport models. They emphasized that the BMF approach does not account for the interactions between different sized particles present in sediment mixtures. They also point out one important disadvantage of this approach is the difficulty in obtaining the correct gradation and total transport amount, and another minor disadvantage is its sensitivity to the number and distribution of class intervals. Therefore they carried out an interesting study on

the bed load transport for sediment mixtures. In their study, they proposed a model for the computation of the mean size of sediments in transport. Based on observation of sediment-mixture experiments, they determined that the fraction of each size class in transported material is proportional to the joint probability of the relative mobility of each particle size and the availability of each size class on the bed surface.

Niekerk *et al.* (1992) developed a one-dimensional numerical model of sediment routing to simulate erosion, transport, and deposition of individual size-density fractions in the bed material within a relatively straight, non bifurcating alluvial channel. The bed load transport rate of each size fraction is calculated from a modified Bagnold equation by considering the effects of turbulent fluctuations in the bed shear stress. Critical shear stresses for entrainment of particles from the mixed bed are determined using relationships in between grain protrusion and hiding. The transport of suspended particles is modelled using a convection-diffusion sediment continuity equation, either explicitly solved by the Rouse equation or implicitly solved by finite difference scheme. This model is more suitable over previous models because of treatment of turbulent fluctuations of bed shear stress, minimization of calibration factors and explicit consideration of multiple grain densities.

Bridge and Bennett (1992) developed a model for the bed load transport of sediment grains of different sizes in which the shape and density were also considered. The behaviour of model is compared with natural data from flumes and rivers. The predicted threshold of entrainment of individual size fractions of sediment mixture bed agrees well with the observation as long as pivoting angle is specified as a function of grain size. The rate and size distribution of bed load transport agrees well with the natural data as long as effective shear stress is defined in presence of bed form, bed load transport measurement and size distribution of available sediment is accurately specified.

Patel and Ranga Raju (1996) checked the performance of the relationships of **Ashida and Michiue (1973)**, **Proffitt and Sutherland (1983)**, and **Bridge and Bennett (1992)** based on a large number of bed load data obtained from flumes as well as fields. They found that none of the present methods give satisfactory results. Therefore they proposed an empirical relationship for the fractional bed load transport which is calibrated using both flume and field data. They

represented fractional bed load transport intensity Φ_{bi} as a function of $\theta_{ci}\xi_b$ where ξ_b is hiding exposure correction factor used in the correction of threshold Shields parameter θ_{ci} corresponding to fractional size d_i .

$$\xi_b = \frac{0.0713}{C_m (C_s \theta_{ci})^{0.75144}}, \quad \theta_{ci} = \frac{\tau'_{0i}}{\Delta \rho g d_i} \quad (1.16)$$

$$C_m(M > 0.38) = 1, C_m(0.05 < M \leq 0.38) = 0.7092 \log M + 1.293 \quad (1.17)$$

$$\log C_s = 0.0644\tau^{*3} - 0.1949\tau^{*2} - 0.9571\tau^* - 0.1957, \quad \tau^* = \log \left(\frac{\tau'_{0i}}{\tau_{0cg}} \right) \quad (1.18)$$

Where M is the Kramer's uniformity parameter, $\tau'_{0i} = \rho g R_b' S_0$ and τ_{0cg} is the threshold bed shear stress for geometric mean size of non-uniform sediment mixture as per Shields.

Wilcock and McArdeil (1997) and Wilcock (1997) investigated partial transport and fractional transport of sediment mixtures. They found the fractional transport rate is the product of spatial entrainment, displacement frequency, and displacement length. Their fractional transport expression emphasized the partial transport of individual size fractions as features of gravel-bed rivers which may not be a major concern in case of sand bed rivers.

Molinas and Wu (1998) established a size gradation compensation factor in order to incorporate the effect of size distribution on the transport of sediment mixtures. The resulting equivalent representative diameter (d_e) can be expressed as

$$d_e = \frac{1.8d_{50}}{1 + 0.8 \left(\frac{V_*}{w_{50}} \right)^{0.1} (\sigma_g - 1)^{2.2}} \quad (1.19)$$

Where V_* is shear velocity; σ_g is dimensionless standard deviation of bed material = $\sqrt{\frac{d_{84}}{d_{16}}}$ and

w_{50} is fall velocity of sediment corresponding to particle size d_{50} . This equivalent representative diameter can be used for existing sediment transport formulas to produce more accurate prediction of transport rates for non-uniform mixtures. The representative diameter based on

Equation (1.19) was mainly developed to compensate for sediment non uniformity effects for existing transport formulas in bed-material load computations, so it lacks generality.

Chatelus *et al.* (1999) investigated experimentally to find out the effect of size gradation in sediment transport. The experiment was conducted in the large oscillating water tunnel of WL/Delft hydraulics under various hydraulic conditions with plane bed. The objective of their study is to increase knowledge of the basic mechanisms of bed-load and suspended load transport in oscillatory flow conditions when graded sediments are present in the bed. The sediment bed consisted of a mixture of two well sorted sands that have been used before in the tunnel in order to make comparisons possible. The experiment determined measurements of net sediment transport rates, bed composition change; time averaged suspended sediment profiles, time dependent concentrations in the suspension and sheet flow layers and time dependent velocity profiles.

Wu *et al.* (2003) developed a new method to compute the fractional bed-material concentrations for non-uniform sediment mixtures. This approach has the profit of avoiding discrepancies due to the distribution and number of class intervals in the estimation of bed-material load, and it limits the errors in calculation of concentrations for individual size fractions. The new expression developed for transport capacity distribution accounts for the sheltering and exposure effects that exist in mixtures. The proposed method has advantage of improving the predictions of fractional transport rates in all size fractions for non-uniform sediment mixtures.

Wu and Yang (2004) incorporated the effect of near-bed coherent flow structures into the entrainment of mixed-size sediments. The fourth-order Gram Charlier type probability density function (GC pdf) of near-bed stream wise velocity is incorporated to account for the higher-order correlations associated with turbulent bursting. The published data compilation over a wide range of bed roughness is used to investigate the near-bed coherent flow structures, including the second, third, and fourth order moments of velocity fluctuation required in the fourth-order GC pdf. The present model using the fourth-order GC pdf significantly improves the accuracy of lifting probabilities for unisize sediments especially at higher dimensionless shear stress. The present model is able to evaluate the entrainment probabilities of mixed-size sediments under

partial and fully transport conditions, while previous models fail to accurately evaluate the entrainment probabilities for partially transported mixed-size sediments.

Wu et al. (2004) investigated the effect of non-uniformity of bed material on the transport of sediment mixtures in sand bed channels. Sediment transport rate based on d_{50} for uniform sediments usually underestimate the transport rates for non-uniform sediment mixtures. It is therefore to introduce the size gradation correction factor K_d as a function of the geometric standard deviation (σ_g) of bed material. It is theoretically computed from the fractional transport approach based on a lognormal particle size distribution of the bed material.

$$K_d = e^{0.5(b \ln \sigma_g)^2} \quad (1.20)$$

Where, b is an exponent. The sediments in transport also follow a lognormal size distribution similar to the bed material size distribution. Due to the selective transport of grains by flow, the median diameter of sediment in transport is generally finer than the median diameter of bed material. The relative median size of sediment in transport, d_{50t}/d_{50} decreases as size gradation increases and relationship between them can be expressed as

$$\frac{d_{50t}}{d_{50}} = \sigma_g^{-b \ln \sigma_g} \quad (1.21)$$

Mazumder et al. (2005) conducted the series of flume experiments with five beds of heterogeneous sediment mixtures having different values of bed roughness and observed that for fixed height and velocity, the amount of suspension concentration increases up to a certain value with increase of bed roughness and then decreases with further increase of bed roughness. This indicates depletion of concentration in suspension due to increase of coarse sediment in the bed. They also developed the mathematical models for computation of actual amount and size distribution of suspended materials over different sediment beds, based on Hunt's diffusion equations for sediments and water. The proposed models give results which compare well with the observed values of actual amount and grain-size distributions comprising the wide range of grain sizes.

Roos et al. (2007) investigated the effect of non-uniform sediment on dynamics of offshore tidal sandbanks, with a particular aim to explain observed surficial grain size variations over tidal

sandbanks from a process based modeling perspective. In this model the sediment transport and bed evolution modules are extended by introducing an active layer and a bimodal sediment mixture. The model results show that for symmetrical tidal conditions, coarse grains tend to accumulate at the bank crests. For asymmetrical tidal conditions they find a spatial phase shift between topography and the mean grain size fraction, indicating an accumulation of coarse grains on the lee side of the bank. The model results qualitatively agree with observations from banks on the Belgian continental shelf.

Yang *et al.* (2009) used conception of exposure degree to account for the hiding and exposure mechanism of non-uniform sediment transport. The force arms of particles on the bed slope are assumed to be a function of the exposure degree, which is stochastically related to the size and gradation of bed materials. Based on this conception, the formula for the angle of repose of non-uniform sediment is developed. The formula has been tested against a wide range of laboratory data; the prediction by the newly proposed formulas is very good.

Ghosal *et al.* (2010) conducted the series of flume experiments to compute the grain-size fractions in the bedload transported over the five heterogeneous sediment beds of different values of bed roughness. The existence of an entrapment factor associated with the sorting observed from the bed to active layer was modeled based on the modified critical shear stress to estimate the grain-size fractions in the transport layer under given hydraulic conditions. The efficiency of these models was tested with the observed data. Subsequently, the patterns of observed grain size distributions in the transport layer were tested to identify the distributions developed in the active layer due to sorting using three probability density functions (pdf), such as, log-normal, log-hyperbolic and logskew- Laplace. Tests indicated that a log-skew-Laplace distribution fitted best for 49%, a log-hyperbolic for 31%, and a log-normal for 20% out of forty-five bedload samples collected under unidirectional flow with changes in flow discharge and bed roughness.

Wu *et al.* (2010) introduced hiding and exposure correction factor which is related to the bed-material gradation and the hidden and exposed probabilities. Hiding and exposure factor is defined by

$$\eta_i = \left(\frac{P_{ei}}{P_{hi}} \right)^m \quad (1.22)$$

Where m is an empirical parameter, P_{ei} and P_{hi} are the total exposed and hidden probabilities of particles d_i given by equation as

$$P_{hi} = \sum_{j=1}^N p_{bj} \frac{d_j}{d_i + d_j} \quad (1.23)$$

$$P_{ei} = \sum_{j=1}^N p_{bj} \frac{d_i}{d_i + d_j} \quad (1.24)$$

Where, N is the total number of particle size fractions of sediment mixtures. Based on correction factor defined by Equation (1.22), the formula of determining the critical bed shear stress for the incipient motion of non-uniform sediment,

$$\frac{\tau_{ci}}{(\gamma_s - \gamma) d_i} = \theta_c \left(\frac{P_{ei}}{P_{hi}} \right)^m \quad (1.25)$$

Where, τ_{ci} is the critical shear stress for particle d_i in non-uniform sediment mixtures; and θ_c can be interpreted as the non-dimensional critical shear stress for the corresponding uniform sediment or the mean size of bed materials. The correction factor used in Equation (1.22) is not only related to the grain size but also to the bed material gradation. Better prediction can be achieved with Equation (1.22) even when the bed-material gradation is very wide and highly irregular.

Wu *et al.* (2010) has also established the formulas for calculating the fractional transport rates of non-uniform bed-load and suspended load and tested by using a large number of laboratory and field data of both uniform and non-uniform sediment transport. Formula for the non-dimensional fractional transport rate of non-uniform bedload is obtained as

$$\phi_{bi} = 0.0053 \left[\left(\frac{n'}{n} \right)^{1.5} \frac{\tau_b}{\tau_{ci}} - 1 \right]^{2.2} \quad (1.26)$$

$$q_{bi} = \phi_{bi} p_{bi} \sqrt{\left(\frac{\gamma_s}{\gamma} - 1\right) g d_i^3} \quad (1.27)$$

Where, q_{bi} is the volumetric bed-load transport rate per unit width; τ_{ci} is determined with Equation (1.25) which has taken into account the hiding and exposure mechanism of non-uniform sediment transport. Bed shear stress $\tau_b = \gamma R_b J$ where R_b is hydraulic radius of channel bed and J is energy slope. n is the Manning's roughness coefficient for channel bed, and n' is the Manning's roughness corresponding to grain roughness, calculated with $n' = \frac{d_{50}^{1/6}}{20}$. Formula for the fractional transport rate of non-uniform suspended load is given as

$$\phi_{si} = 0.0000262 \left[\left(\frac{\tau}{\tau_{ci}} - 1 \right) \frac{U}{w_i} \right]^{1.74} \quad (1.28)$$

$$q_{si} = \phi_{si} p_{bi} \sqrt{\left(\frac{\gamma_s}{\gamma} - 1\right) g d_i^3} \quad (1.29)$$

Where, q_{si} is the volumetric suspended-load transport rate per unit width; τ is the shear stress on the entire cross-section = $\gamma R J$, where R is the hydraulic radius of channel. Here U is the average flow velocity and w_i is settling velocity of sediment particle calculated with **Zhang and Xie (1993)**.

$$w_i = \sqrt{\left(\frac{13.95\nu}{d_i}\right)^2 + 1.09 \left(\frac{\gamma_s}{\gamma} - 1\right) g d_i} - \frac{13.95\nu}{d_i} \quad (1.30)$$

Where, ν is kinematic viscosity. Fractional transport rate for non-uniform bed-material load is the summation of q_{bi} obtained from Equation (1.27) and q_{si} obtained from Equation (1.29). These newly proposed formulas given by Equation (1.27) and Equation (1.29) provide better predictions than several existing methods.

Patel et al. (2010) proposed the relationships expressing the ratio of dimensionless critical tractive stress (CTS) of a particular size fraction to the dimensionless CTS of scaling/representative size in sediment mixtures, as a function of their respective size ratios. The predictor of dimensionless CTS of scaling size is a mandatory requirement to predict the CTS of

a particular size fraction of a sediment mixture. Data of the CTS for non-uniform sediments from the previous investigations have been analyzed to check the adequacy of these predictors. An equation is proposed to predict the CTS of scaling sediment size in terms of Kramer's uniformity coefficient. The performance of the proposed relationship was also tested by comparing observed and predicted results with independent laboratory and field data. The proposed relationship performs better for both unimodal and bimodal grain size distributions.

Patel et al. (2013) have used the experimental data of **Patel et al. (2010)** to verify the adequacy of existing relationships for CTS of non-uniform bimodal sediments. Keeping in view the performance of such relationships, a new relationship for computation of dimensionless CTS of individual size fraction of non-uniform unimodal and bimodal sediments has been proposed in the form of power law. The coefficient and exponent of the proposed power law have been empirically correlated with sediment characteristics for available sediment data. The proposed relationship has been validated with independent flume and field data to assess their applicability for unimodal and bimodal sediments.

Patel et al. (2015) pointed out that the existing relationships, for computation of fractional bed load transport of non-uniform sediments, have been assessed for their applicability to bimodal sediments. The relationships could not work well for bimodal sediments as less than 50 % data points are laying within 0.25-4.0 range of discrepancy ratio (R_d). A new relationship for fractional bed load transport has been proposed based on recent method available for fractional critical shear stress of non-uniform unimodal and bimodal sediments. The proposed relationship for fractional bed load transport rates has been verified with independent flume and field data, available from past investigations along with data collected in present study. The relationship performed satisfactorily for whole range of data as around 75% data points are laying within 0.25-4.0 range of discrepancy ratio (R_d).

Mohtar et al. (2016) evaluates the applicability of representative sediment size in terms of median grain size and the effective sediment sizes for the transport of non-uniform natural sediment mixtures. To account for the non-uniformity of the bed materials, a comparison between the calculated bedload discharges using d_{50} was made with the bedload prediction when representative sediment diameters were used. Analysis shows that an improvement of the

statistical analysis was only observed for predictions calculated using **Brownlie (1981)** and **Camenen and Larson (2005)** expressions when effective sediment sizes were employed for non-uniform sediment mixtures. Employing the effective sediment sizes d_e or d_{em} to calculate the bedload discharge does not necessarily improve the bedload prediction and the use of d_{50} is seen as sufficient.

Elhakeem and Imran (2016) developed a bedload model for non-uniform sediment considering the density function of sediment entrainment with no active layer. Four parameters are incorporated in the model formulation, namely the Shields stress, the critical Shields stress, the Kramer coefficient of uniformity, and the relative grain size. The model collectively accounts for the hiding effects and the variation of the transported bedload material fractions from the surface and subsurface material. Further, the model provides more insight in the expected errors in the predictions of both the substrate-based and surface-based models. Model results show that errors in the predictions of some fractions in the sediment mixture can be as large as 100% and 350% for the substrate-based and surface-based models, respectively. By comparing the predictions of selected models with measured bedload data, it has been found that the proposed bedload model provides better predictions over the other models examined herein, highlighting the importance of considering the interaction between the surface and subsurface material when estimating the bedload rate material.

1.3 Studies on the Importance of Seepage

Water percolates in the form of seepage through boundaries of alluvial channels, rivers and streams due to porosity of the sandy material and difference in water level in the stream and ground water table. Flow in natural channels is a complex interaction between surface and subsurface flow. The presence of seepage of the flow from a channel is termed as suction (Figure 1.2) and into a channel is termed as injection (Figure 1.3). Seepage takes place through the porous boundary of the alluvial channels because of the difference between the water level in an alluvial channel and surrounding ground water table. As a result, mass and momentum of flow is transferred across the interface between the fluid and porous media takes place through the permeable boundaries. The sediment particles exert an additional hydrodynamic force under the presence of seepage. These additional forces can have significant effect on the processes of

sediment transport including the threshold of motion of the sediment particles and the flow resistance in alluvial channels which may lead to the change in their geometry.

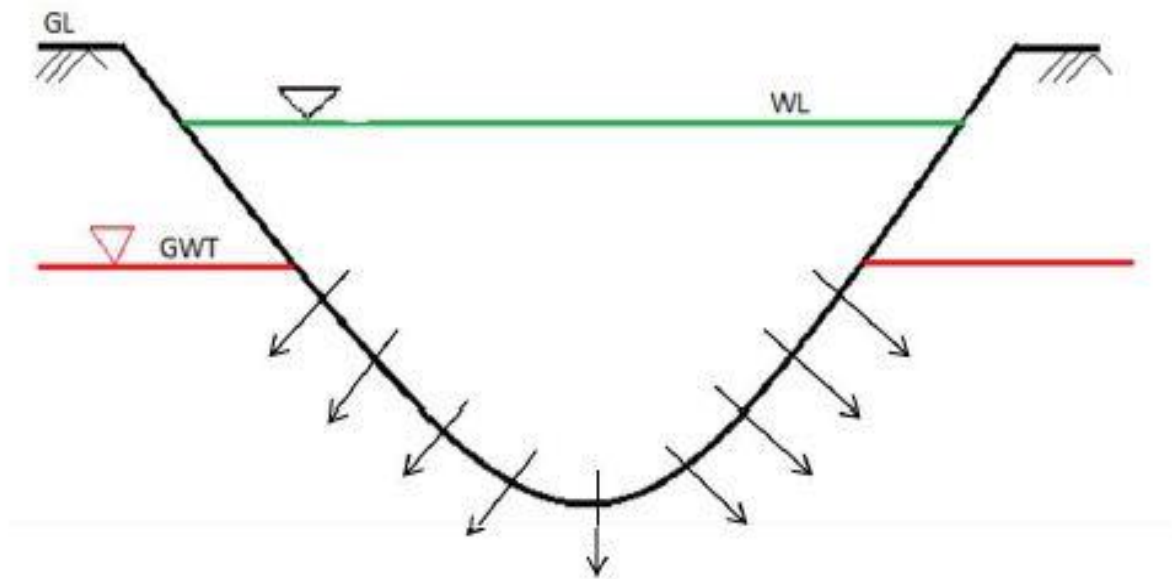


Figure 1.2: Downward seepage (Suction)

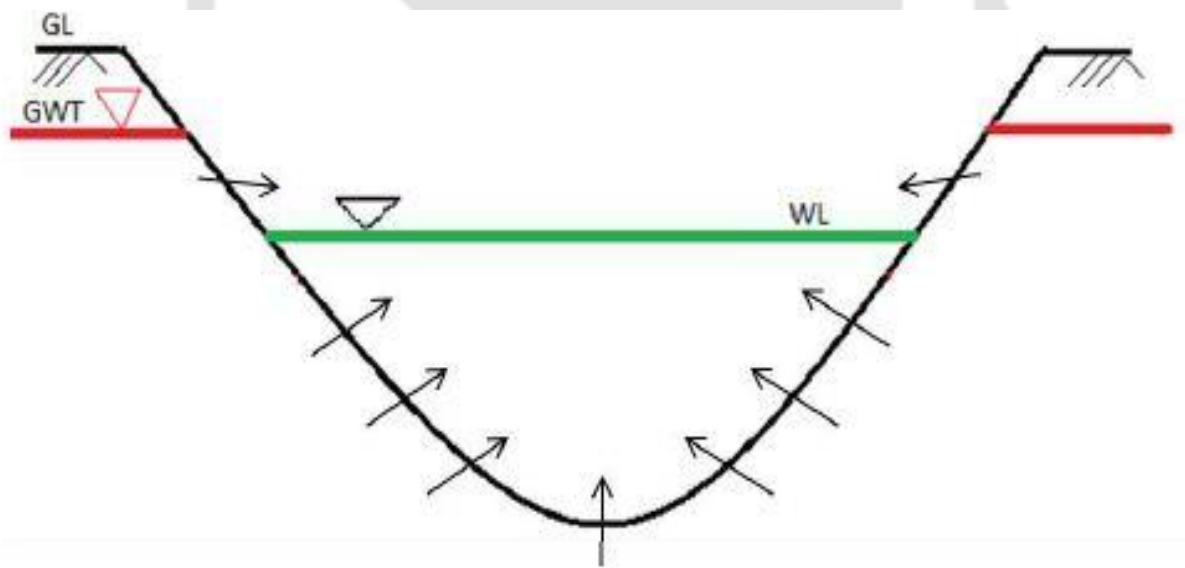


Figure 1.3: Upward seepage (Injection)

In the current scenario, the ground water table is lowering because of maximum use of tube-wells in the surrounding area of natural channels. Therefore, the difference in the ground water table and flow level is increasing day by day in the flood plains of alluvial channels, causing

downward seepage. Downward seepage plays an exceedingly important role in the design of hydraulics structures such as dams and canals, particularly, it influences the stability of bed particles and canal slopes. Previous literatures suggest that unlined irrigation canals in permeable soils allow considerable loss of water through the bed and sides of the canals, resulting to reduce the conveyance efficiency of the canal (**Krishnamurthy and Rao, 1969; Sharma and Chawla, 1975; Raja et al., 1983; Berenbrock, 1999; Carlson and Petrich, 1999; Fipps, 2005; Martin and Gates, 2014**).

Presence of seepage leads to a change in the bed deformation conditions and consequently the hydrodynamic characteristics of the channel. The study of the effect of seepage flows on the detachment of particles from the bed and further movement of the bed load is of great interest, since this study is related to the solution of important practical engineering problems. For instance, groundwater movement plays an exceptionally important role when constructing hydraulic structures, particularly dams; these challenges are urgent when solving problems concerned with the stability of dams and canal slopes. Earthen irrigation channels in permeable soils lose considerable amount of water through the bed and sides of the canals resulting in low conveyance efficiency. By the time the water reaches the field it has been estimated that depending upon the site specific condition, the seepage losses may be in the order of 45% of the water supplied at the head of the canal (**Sharma and Chawla, 1975**).

Krishnamurthy and Rao (1969) studied the water losses using radioisotopes in the Ganga canal and reported the seepage losses as $2.2 \text{ m}^3/\text{day/m}$. **Raja et al. (1983)** estimated the seepage losses using nuclear technique in an unlined channel and reported that the seepage losses to vary from 1.3 to $4.3 \text{ m}^3/106 \text{ m}^2$ of the wetted surface area. **Dukker (1994)** used inflow-outflow technique to measure the seepage losses from Lower Gugera Branch Canal and found a wide range of variation in seepage rate due to errors and uncertainties in measurements. Their results vary from 3.54 to 62.04 cm/day . **Weller and McAteer (1993)** reviewed all seepage measurement techniques and discussed the statistical treatment of random errors in the inflow-outflow method. With fairly precise current metering the errors indicated in the inflow and outflow measurements are reported to be as high as 110%.

An analysis of seepage from the New York Canal (NYC) estimates that cumulative seepage rates range between 12% and 20% at canal flows of 439 to 980 cubic feet per second (cfs) (**Berenbrock, 1999; Carlson and Petrich, 1999**). According to the estimation of **Yussuf *et al.* (1994)** and **Tanji and Kielen (2002)** in unlined canals of semi-arid regions, seepage losses can account for 20-50% by volume of the total flow. Based on field condition, the loss of water due to downward seepage in alluvial channel is about 45% of the water provided at the canal head and fluctuates from 0.3 to 7 m³/s per million square metres (Mm²) of wetted area (**Shukla and Misra, 1994**). Around 17% of the water that was conveyed for irrigation in 1985 was lost according to the report of **Carr *et al.* (1990)** in the United States due to evaporation or seepage to the groundwater. Conveyance efficiency from the point of diversion to the field in canals in the Lower Rio Grande Valley in Texas was found to be 69.7% (**Fipps, 2005**). Australian National Committee on Irrigation and Drainage (ANCID, 2006) has found that 10-30% of water is lost in the form of downward seepage from alluvial channel. **Kinzli *et al.* (2010)** used acoustic Doppler instrument to measure canal flow rates and found that more than 40% volume of the diverted water could be lost due to seepage in the Middle Rio Grande Valley of New Mexico. **Martin and Gates (2014)** quantified loss of around 15% of the main flow rate due to downward seepage.

Interactions between flow in the main stream and the ambient groundwater, or hyporheic exchanges, are important because of their role in controlling the transport of contaminants and maintaining a healthy stream ecosystem (**Brunke and Gonser, 1997; Jones and Mulholland, 2000**). Even though the seepage flow is small, this small seepage flow can affect an additional hydrodynamic force on the sediment which will affect the process of sediment transport such as sediment entrainment and formation of bed features. **Willems and Drossos (1975)** noted that a water intake, when drawing faster moving water towards the sediment bed, can increase the local bed shear stresses resulting in bed erosion around the intake structure. More complex situations occur in the coastal environments, for example in swash zone, where seepage varies both temporally and spatially (**Turner, 1995; Karambas, 2003**). The hydrodynamic process in such a physical system may result in changes to the structural features of the flow, implying modification to flow-resistance, sediment entrainment characteristics and morphology of the streambed as compared to that with an impermeable boundary, as is often encountered in a laboratory study. Knowledge of flows over permeable boundaries clearly is not only useful for

fundamental research, but also has practical importance in engineering applications. Despite the important influence of seepage on many aspects of practical engineering, relevant information in this area remains scarce. **Watters and Rao (1971)**, **Willetts and Drossos (1975)**, **Maclean (1991a)** and **Rao and Sitaram (1999)** reported that suction increases the bed material transport. **Watters and Rao (1971)** and **Rao and Sitaram (1999)** reported that injection reduces the sediment transport rate and increases the stability of the particles or it does not aid in initiating their movement. Hence, in order to have a deep understanding about the seepage effects on the channel stability, and to make use of proper equations in the alluvial channel design, it is essential to perform the experiments in the laboratory to understand the problem.

Waters and Rao (1971) studied the hydrodynamic effect of seepage on bed particles. They measured the drag and lift forces on the plastic spheres and observed that lift forces should be considered in the stability criteria for bed particles as the lift and drag forces on a particle are of comparable magnitudes. When the drag forces on particles are considered; downward seepage increases particle motion, while upward seepage hinders it. When lift forces are considered; upward seepage prevents the motion of a plane bed particle but the motion of a particle above the general bed level is increased. Flow configuration near the bed is affected by the presence of seepage through the bed of an alluvial channel and seepage modifies the sub layer thickness of the channel bed.

Oldenziel and Brink (1974) found that a small increase in shear stress appreciably increases the rate of transport of sand particles. They concluded that the sediment transport rate is always decreased in case of downward seepage, while upward seepage increases the transport rate.

Willetts and Drossos (1975) through their experimental study in a narrow flume with small downward seepage zone observed that downward seepage produces a localized scour hole in the downward seepage zone and grains move at a faster rate in the downward seepage zone than elsewhere in the flume.

Richardson *et al.* (1985) observed that upward seepage caused a very small increase in the stream power for flows near or in the upper regime. In low regime flows sediment transport was increased with upward seepage as compared to that when no seepage was present.

Maclean and Willetts (1986) investigated the changes of bed shear stress with and without seepage conditions by visualizing the initiation of motion of the indicator grains. It was found that bed shear stress increases with the application of downward seepage.

Nakagawa *et al.* (1988) investigated the effect of seepage on sediment pick-up rates. They found that the changes induced by seepage occur mainly with the pick-up rate rather than the step length related to the motion of sediment particles. The bed load transport model previously established was thus modified to include the effect of seepage, showing that the bed load transport rate is enhanced by upward seepage and reduced by downward seepage.

Maclean (1991a) carried out velocity measurements in two sets of experiments in an open channel and in a wind tunnel with high rates of downward seepage. He observed gradual decrease of shear velocity in the downstream direction over the seepage zone.

Maclean (1991b) observed that the increase in bed shear stress in the presence of downward seepage, leads to local scour around the seepage zone. He found that an increase of downward seepage velocity by 10% causes around two times increase in bed shear stress as compared to uniform shear stress.

Prinos (1995) solved the Reynolds-averaged Navier-Stokes equations and studied the effect of downward seepage on the boundary shear stress. The bed shear stress increases with increasing downward seepage rate in the seepage zone. For downward seepage velocity of 9% of the mean channel velocity, the increase in bed shear stress with downward seepage is about eight times to that of without downward seepage.

Cheng (1997) carried out experimental study to find out the effect of upward seepage on open channel flow. He observed increase in the root mean square values of velocity fluctuations and Reynolds shear stresses in the near-bed region.

Cheng and Chiew (1998b) derived an equation to evaluate the shear velocity by integrating the continuity and momentum equations. The derived equation for bed shear stress based on momentum integral equation compared well with the results obtained from the measured Reynolds shear stress distribution (**Cheng, 1997**). **Cheng and Chiew (1998a)** proposed another method for evaluating the bed shear stress by directly fitting the modified logarithmic law for

upward seepage. Results indicated that the shear stress reduced noticeably at the beginning of the upward seepage zone and for the higher seepage intensity the reduction became more apparent. Nevertheless, gradual increase in the bed shear stress has been observed towards the downstream end of the upward seepage zone.

Cheng and Chiew (1999) carried out analytical and experimental studies considering additional force because of seepage on a sediment particle in threshold condition. They showed that the critical shear velocity decreases with the application of upward seepage.

Rao and Sitaram (1999) investigated the seepage effects on incipient motion of sand-bed particles. They found that seepage through a sand-bed in a downward direction reduces the stability of particles and it can even initiate their movement. The bed erosion is increased with the increased rates of suction. However the seepage in an upward direction increases the stability of bed particles and does not aid in initiating their movement.

Krogstad and Kourakine (2000) investigated localized upward seepage effects on the turbulence structure in a boundary layer. It has been observed in their experimental results that as the incoming flow entered the upward seepage region, the bed shear stress was significantly reduced.

Chen and Chiew (2004) investigated the influence of downward seepage on the flow hydrodynamics on the plane bed channel using experimental approach. They observed that near the channel bed (towards water surface) velocities increases (decreases) with application of downward seepage. It was observed that the vertical distribution of stream wise velocities does not follow universal law of the wall in the presence of seepage. In addition to this, origin displacement, slip velocity, and shear velocity increases with increase in relative downward seepage, indicating distortion of boundary layer.

Lu et al. (2008) carried out an extensive review on the effects of seepage on the movement of sediment particles and they showed that the effective weight of a sediment particle increases with the application of downward seepage. The graphical representation of effective weight of particles and bed shear stress variations in accordance with the downward and upward seepages is shown in Figure 1.4. In the Figure 1.4, the positive and negative signs signify the increase and

decrease of the movement of bed particles, respectively. As discussed earlier that downward and upward seepage cause increase and decrease of bed shear stress, respectively. It can be observed from quadrant 2 that stability of bed particle reduced because of increase in bed shear stress and decrease in the effective weight of bed particles. However, stability of bed particle enhanced because of decrease in bed shear stress and rise of effective weight as shown in quadrant 4. Hence both of these phenomena cannot happen together. Apart from this, it can be observed from quadrant 1 that downward seepage increases the effective weight of particles, resulting in the increase in stability. In contrast, downward seepage has the propensity to increase bed shear stress, causing reduction in the stability of bed particles. From these scenarios, it signifies the movement of bed.

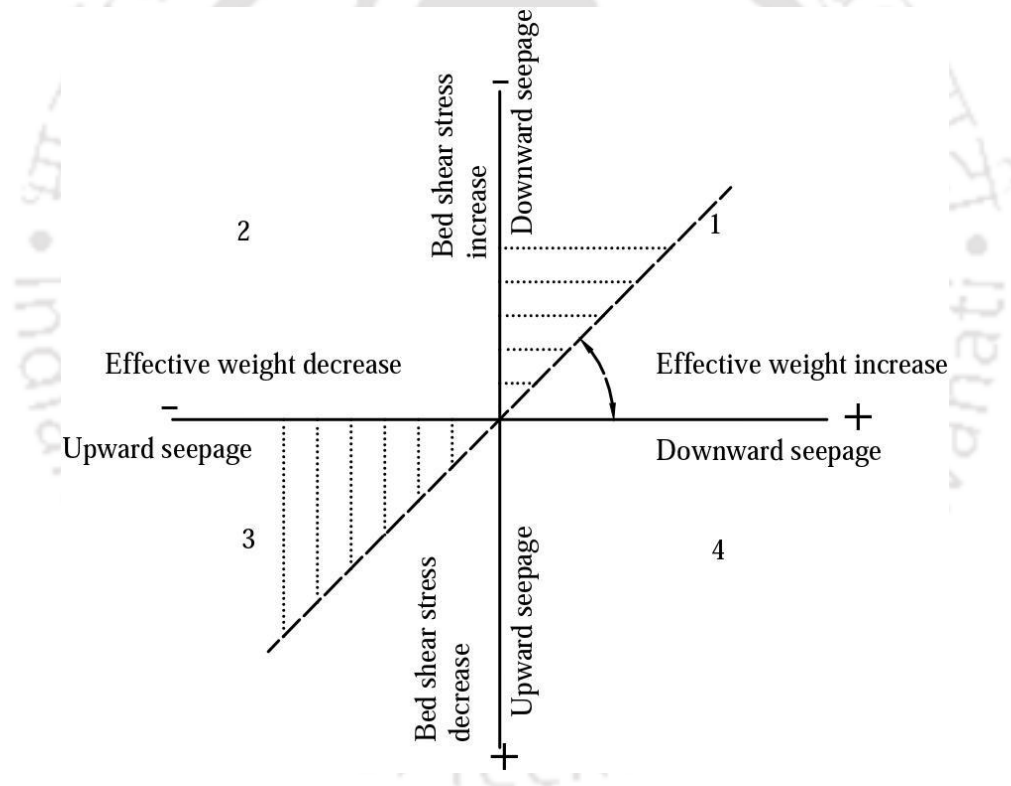


Figure 1.4: Description of bed particle stability under the influence of seepage using quadrant analysis of various forces acting on a particle (Lu *et al.*, 2008)

Rao and Sreenivasulu (2009) studied the importance of seepage in the design of channels and revealed that seepage either in “upward” or “downward” can change the resistance as well as mobility of sediment particles. Therefore, it is concluded that the effect of seepage must be

considered in canal design. Experimental studies show that suction reduces the stability of the bed particles and initiates their mobility whereas injection increases the stability of the bed particles and reduces their mobility.

Kavcar and Wright (2009) modified the Shields parameters by considering the influence of vertical pore water flux (downward seepage) on the movement of sediment particles. Required bed shear stress for incipient motion of bed sediment was increased as the amount of downward seepage was increased. They also found that the bed hydraulic slope plays a significant role in order to characterize the initiation of sediment motion.

Sreenivasulu et al. (2010) observed higher velocities with seepage as compared to no-seepage in their experiments. Variation in the friction slope was small for no seepage as compared to that with downward seepage. Bed shear stress increased along the length of the flume for no seepage and a reverse trend was observed when downward seepage was applied. Overall bed shear stress was higher for downward seepage than that for no seepage.

Rao et al. (2011) carried out experiments relevant to the regime behavior of an alluvial channel affected by seepage. Consideration of seepage in flow analysis of regime channel alters its stability. Regime theory was used to define the regression equations for a channel's dimension under stable conditions. Bivariate and trivariate regression relationships have been developed for an alluvial channel design under seepage.

Sreenivasulu et al. (2011) presented the variation of stream power in the presence of downward seepage affected channels and observed higher stream power at upstream end of channel and it decreases towards downstream direction because of changes in seepage quantity. It was observed that the higher stream power caused erosion at upstream end, and consequently, eroded material get deposited at downstream end due to reduction in stream power.

Liu and Chiew (2012, 2014) performed both theoretical and experimental investigations in order to understand the seepage effects on the initiation of cohesionless sediment particles. In their theoretical analysis, external force exerted because of downward seepage was examined. They conducted five experiments on uniform sand bed to quantitatively show downward seepage effects on sediment entrainment. These analytical and experimental works provide an overall

understanding regarding seepage effects on the sediment movement. In their study, results indicate that downward seepage increase shear velocity and upward seepage decrease shear velocity.

Cao and Chiew (2014) carried out experiments to observe the effects of downward seepage on sediment transport rate in closed-conduit flows. Their results indicate that the near-bed velocities are closely related to the sediment transport rate with downward seepage and bed load transport rate increased with rate of downward seepage. They also carried out numerical simulation of the closed-conduit flows and observed results similar to their experiments.

Deshpande and Kumar (2016b) conducted an experimental study on alluvial channel and found that at high bed shear stress in alluvial channels made of non-cohesive material, sediment transport occurs as sheet flow layer of high sediment concentration. The sediment transport in the form of sheet flow is observed when suction is applied to the non-transporting channels designed on incipient motion condition. The erosion of the channel banks contributes to the sheet flow because of the increased channel bed shear stress. An empirical relation for the thickness of sheet flow layer is developed which includes suction as independent parameter along with others.

Devi and Kumar (2016) conducted the laboratory experiments in a tilting flume to investigate flow characteristics in a threshold channel with submerged vegetation. The presence of aquatic vegetation changes the flow characteristics in a channel. The impact of the presence of submerged, flexible vegetation in a channel with an additional parameter such as downward seepage is analyzed in their work. They observed the presence of an inflection point near the vegetation top because of the difference in velocity in the vegetation zone and the upper flow region. This inflection point leads to the formation of vortices that mix the whole flow region. Velocity is increased to an average value of 10% in the lower flow region with the presence of seepage, with higher velocities for higher seepage percentages.

Patel and Kumar (2017) reported the findings from an experimental study on a parabolic cross-sectional sand bed channel with uniform fine sand under no seepage and downward seepage conditions. Through experiments, they observed that an alluvial channel, which remained at threshold condition of sediment movement during the no seepage experiment, started transporting sediments with the application of seepage in the downward direction. Shields stress

of the threshold channel increased significantly from its critical value after the application of downward seepage, which led to the deformation of the cross-sectional shape and consequent development of the bedforms. It has been further observed that the variation in Shields stress and corresponding bedform geometry reached an equilibrium state in the presence of downward seepage when the experiments were run over a longer period of time.

1.4 Studies on the Turbulent Parameters of Flow

Various researchers (Lu and Chiew, 2007, Dey and Nath, 2010, Dey *et al.*, 2012, and Deshpande and Kumar, 2016a) carried out experiments to observe the effect of the presence of seepage on the turbulent characteristics of flow in open channels. Their findings about the effects of downward seepage and upward seepage on bed shear stress and sediment transport rate are summarized in Table 1.1. Sediment transport dynamics can be viewed as one part of the field of the two-phase flow i.e. flows of a fluid that contains discrete solid particles within it. Turbulent sediment- transporting flows represent one of the most challenging problems in all of hydraulics and fluid mechanics for two reasons. First, the presence of the included phase can alter the turbulence structure of the including phase. Second, in many cases the flow boundary consists of the sediment particles, and then the flow can shape its own boundary but in turn may get affected by that shaped boundary. Changes in the essential nature of the flow, starting with turbulent flow in a closed- conduit and sediment-transporting flow over a loose sediment bed in the last have been depicted in Figure 1.5. The step from a circular pipe to a rectangular duct brings the presence of a weak but non-negligible secondary circulation, but there is no big difference in the structure of the shear-flow turbulence. In the next step from rectangular duct to an open rectangular channel, the turbulence structure is again only slightly different, as are the secondary circulations. But the presence of deformable free surface here in open rectangular channel makes different effects in unsteady flows. In the next step to an open rectangular channel with a loose sediment bed, is the big one: because the flow here can mold the bed now, and the bed too has a strong effect on the flow, the turbulence structure and the free-surface geometry are significantly different in certain ranges of flow. The last step, to a channel with erodible boundaries, makes for greatly different bed geometry, at least in certain ranges of the flow.

Table 1.1: Effect of seepage on channel hydrodynamics

Authors	Flume dimensions			Seepage effects			
	Length (m)	Width (m)	Seepage zone (m)	Bed stress	shear	Sediment transport rate	
				US	DS	US	DS
Watters and Rao (1971)	4.6	0.7	1.5	NA	NA	-	+
Oldenziel and Brink (1974)	15	0.5	4	NA	NA	+	-
Willetts and Drossos (1975)	3.6	0.076	0.125	NA	NA	NA	+
Richardson <i>et al.</i> (1985)	9.45	0.6	3	NA	NA	+	-
Maclean & Willetts (1986)	5	0.076	5	NA	+	NA	NA
Maclean (1991a)	12	0.3	0.28	NA	+	NA	NA
Maclean (1991b)	5	0.075	0.13	NA	+	NA	+
Prinos (1995)	Analytical model			NA	+	NA	NA
Cheng (1997)	7.6/30	0.21/0.7	0.5/2	-	NA	+	NA
Cheng and Chiew (1998a)	30	0.7	2	-	NA	NA	NA
Cheng and Chiew (1998b)	30	0.7	2	-	NA	NA	NA
Cheng and Chiew (1999)	7.6	0.21	0.5	-	NA	+	NA
Rao and Sitaram (1999)	3.6	0.158	2.4	+/-	+/-	-	+
Krogstad and Kourakine (2000)	3.5	0.46	0.12	-	NA	NA	NA
Chen and Chiew (2004)	30	0.7	2	NA	+	NA	NA
Lu and Chiew (2007)	30	0.7	2	NA	NA	-	+
Dey and Nath (2010)	12	0.6	2	-	+	NA	NA
Sreenivasulu <i>et al.</i> (2011)	25	1.8	20	NA	+	NA	NA
Dey <i>et al.</i> (2010)	15	0.6	2.1	-	+	NA	NA
Rao <i>et al.</i> (2011)	25	1.8	16	NA	NA	NA	+
Cao and Chiew (2014)	9	0.3	0.5	NA	NA	NA	+
Deshpande and Kumar (2016a)	17.2	1	15	+	-	+	-

(US) upward seepage, (DS) downward seepage, (+) Increasing, (-) Decreasing, (NA) Not applicable

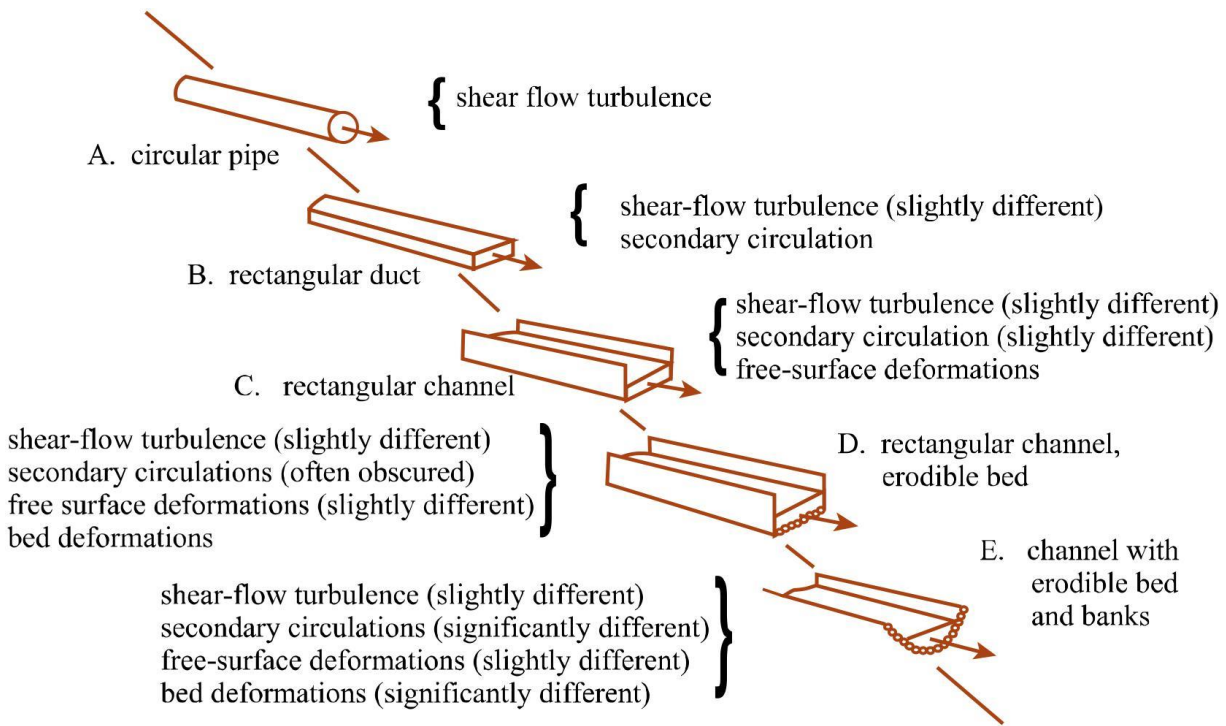


Figure 1.5: Complexity in sediment-transporting flows (Image source: MIT Open Course-Ware)

Sediment transporting flows in the natural streams and canals are turbulent in nature. Elegant work has been carried out in the past to understand the characteristics of turbulent flows and its interaction with sediment transport. With recognition of bursting phenomenon (**Kline *et al.*, 1967**), more insights into the coherent structure of turbulent flows were made possible. Various researchers (**Sutherland, 1967; Thorne *et al.*, 1989; Clifford *et al.*, 1991; Best, 1992; Cao, 1997; Dey and Raikar, 2007; Dwivedi *et al.*, 2010; Dey *et al.*, 2011**) studied the turbulent structure of flow over bed particles that were on the threshold of motion. Turbulent characteristics of flows over mobile beds have also been studied by **Heathershaw and Thorne (1985)**, **Drake *et al.* (1988)**, **Song *et al.* (1994)**, **Bennett and Bridge (1995)**, **Nelson *et al.* (1995)**, **Gallagher *et al.* (1999)**, **Nikora and Goring (1999, 2000)**, **Sumer *et al.* (2003)**, **Venditti *et al.* (2005)** and **Dey *et al.* (2012)**.

However, sediment transport in alluvial streams is highly influenced by the presence of seepage. Water in alluvial streams continuously seeps into or out of the channel bed and channel banks (**Richardson *et al.*, 1985**). In previous studies, the effect of seepage through porous boundaries is clearly visible on the turbulent characteristics of flow. **Nezu (1977)** observed that the higher

velocity region is attracted towards the wall in the case of downward seepage and it is lifted away from the wall in the case of upward seepage. **Oldenziel and Brink (1974), Maclean (1991a) and Chen and Chiew (2004)** have also observed increased velocity in the near-bed region in the case of downward seepage.

Similarly, **Dey and Nath (2010)** observed increased streamwise velocity towards the wall in the presence of downward seepage and decreased streamwise velocity in the presence of upward seepage. **Antonia *et al.* (1988), Krogstad and Kourakine (2000) and Oyewola *et al.* (2004)** observed increased anisotropy in the near-bed region in flows affected by downward seepage, while **Dey and Nath (2010)** observation suggests strongly anisotropic turbulence. **Watters and Rao (1971)** observed increased hydrodynamic roughness and decreased levels of turbulence with lesser momentum exchange between fluid particles in the case of downward seepage. **Nezu (1977), Antonia *et al.* (1988), Antonia *et al.* (1995), Lu and Chiew (2007), Dey and Nath (2010)** have observed that turbulent intensities decrease in case of downward seepage, while they increase in case of upward seepage (**Nezu, 1977; Cheng and Chiew, 1998b; Krogstad and Kourakine, 2000; Dey and Nath, 2010**).

1.5 Research Gap

Most of the studies in the experimental flumes by previous researchers to observe turbulent characteristics of flow with downward seepage and upward seepage have been carried out on either plane or fixed bed. Almost all the investigations which focus on flow field in sand bed comprising of heterogeneous sediment mixture, however, have neglected the downward seepage related with mobile beds. The open question is whether to neglect the contribution due to downward seepage is a correct assumption, since in plane beds the same assumption would be considered a rough approximation. Although the flow characteristics in non-uniform sand channel raise the interest of the researcher community, the hydrodynamics of non-uniform sand bed channel with downward seepage is not approached. After extensive review of various literatures, it is felt necessary to have a detailed and comprehensive research in the laboratory to evaluate the different flow characteristics of non-uniform sand bed open channel. It may be concluded that, sand-bed channels are losing a substantial part of the usable water through seepage. Seepage loss not only depletes water resources but also alters the hydrodynamic

behavior of the channel. Therefore, seepage loss should be considered while designing a channel section. The hydraulic designers are still lacking of a rational method to determine the hydrodynamics in non-uniform sand riverbeds with downward seepage. Because of practical difficulties in obtaining sufficiently accurate and comprehensive data in actual field experiments, well designed laboratory studies are preferred as a truthful method to provide information concerning details of the hydraulic parameters. Such information is important in application and development of numerical models that can help to predict flow characteristics in non-uniform sand bed open channel. In the present work, an experimental approach to study the flow characteristics in a non-uniform sand bed channel affected with downward seepage is necessary for proper understanding of the problem

Although previous studies explored bedload transport in the case of non- uniform sand, the bed load transport of alluvial channel composed of non-uniform sediment with downward seepage remains unexplored. Considering the importance of downward seepage as discussed, the current work takes the attentions of the influence of seepage flow on bed load transport over a mobile rough boundary. Flume experimental data are generally chosen for the experimental observations because of capability of controlling over flow parameters and bed materials (**Chen and Stone, 2008**). Field investigations normally contain several confusing issues such as dimension of bed material distributions, parameter channel geometry and flow environments that influence the data quality. A wisely planned flume experiment can avoid most of the said problems and offer a more quality data set. Thus, experiments have been done for free-surface flows exposed to downward seepage from the boundary.

The drawback of previous study (**Liu et al., 2014**) is that their work is restricted to the channel with local seepage (seepage zone length of only 2m out of flume length 30m), which does not satisfy the actual field condition as well as the natural rivers. The seepage zone length is important in the study of the time dependent sediment transport because sediment transport rate increases with seepage zone length (**Cao and Chiew, 2014**). Also, Riverbeds are usually comprised of non-uniform sediment mixture (**Wu et al., 2004**). Thus, in the present work, we have explored the time dependent sediment transport over the non- uniform sand bed channel with the whole permeable bed and to modify the empirical expression given by **Liu et al. (2014)** in order to relate the bedload transport rate with time.

From all the previous research and discussion, it can be stated that alluvial channels experience a significant loss of water through their bed and banks in the form of downward seepage. Apart from depleting the conveyance capacity of the alluvial channel, hydrodynamic behavior and hydraulics alter in the presence of downward seepage. Therefore, it is needed to consider the influence of downward seepage with an appropriate precision in order to design the earthen canals for the hydraulic engineers. Because of practical complications in measuring data in actual field condition, it is intended to gain insight into the physics behind the development of bed-features in the presence of downward seepage by using instruments and experimental method.

1.6 Objectives of the Study

Riverbeds are generally comprised of non-uniform sand and the distribution of transporting bed material is usually finer. Majority of the previous experimental studies are performed to analyze the turbulent flow characteristics and bed morphology on the plane and fixed beds composed of uniform sand. Although, there has been earlier research on hydrodynamics of channel composed of uniform sand, the flow hydrodynamics with downward seepage in a non-uniform sand bed channel are still unexplored. Therefore, the aim of this study is to observe turbulent flow characteristics and bed morphology on the non-uniform sand beds and also to investigate the influence of downward seepage on turbulent flow characteristics and bed morphology in a non-uniform sand bed alluvial channel. The present study examines the role of turbulent flow in the development of bedload transport with the application of downward seepage to a non-uniform sand bed channel. Based on this, following are the objectives of the present study.

1. Experimental investigation of boundary layer development over non-uniform sand bed channel with downward seepage.
2. Investigation of the turbulent flow structure for no seepage and with seepage conditions in non-uniform sand bed channels.
3. Statistical analysis and temporal variation of the bedload transport at different percentages of seepage.
4. Observation of the hydraulics of sheet flow and its propagation over time and space on a non-uniform sand bed channel with downward seepage.

A schematic view of the objectives outlined in the present research work is shown below:

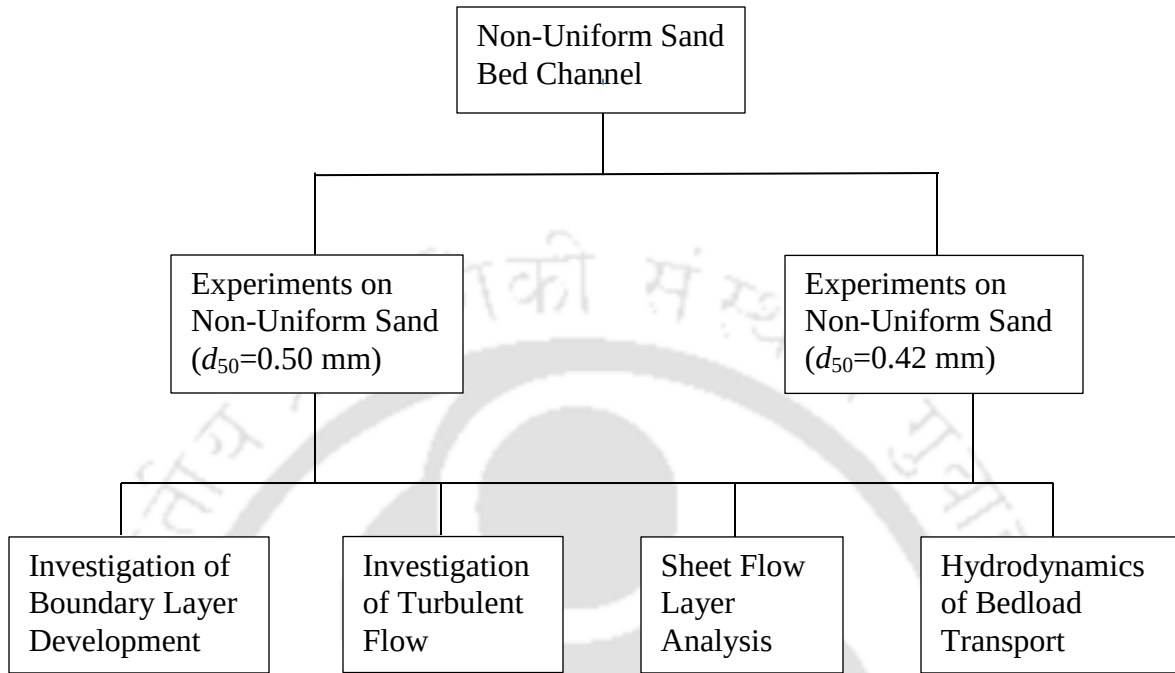


Figure 1.6: Experimental flow chart of the present research work

1.7 Thesis Organization

The thesis contains six chapters. Brief review of each chapter is as follows:

Chapter 1 presents an introduction of the research with the review of literature of various pioneering investigators in the field of non-uniform sand. It highlights various experimental investigations performed using various types of sediment mixture. A detailed review regarding effect of downward seepage on sediment transport in alluvial channel is documented. Turbulent flow in channels and the studies carried out by previous researchers in the context are discussed. After an extensive review, research gap, objectives of the study are outlined in this chapter.

Chapter 2 presents the detailed description of the experimentations carried out in this study. Development of the flume is described. Bed material and state of art instruments used in the proposed experimental study are presented. By following this, experimental framework and program were discussed in this chapter. Detailed procedure of the data collection and its post-

processing with the results is given. Experimental investigations of boundary layer development with and without downward seepage are discussed.

Chapter 3 presents the turbulent characteristics associated with the flow over non-uniform sand bed channel. Also, the changes in turbulent of flow structure with the application of downward seepage are discussed to get an insight into physics behind the increment of sediment transport with seepage. The probability distribution function of turbulent parameters is analyzed with and without seepage. Also, theoretical expression for probability distribution function of turbulent intensity is proposed.

Chapter 4 deals with the effect of high flow discharge on the bed features of non-uniform sand bed channel. It shows the changes in bed morphology and flow process when high flow discharge was applied on this channel. Also, the changes in bed morphology and flow process with the application of downward seepage are discussed. Hydraulic characteristics of sheet layers at different time intervals are discussed. Linkage between flow and sheet layer development is described by analyzing the turbulent flow. The statistical analysis of migrating sheet flow layer with and without downward seepage is presented.

Chapter 5 deals with the bed load transport of alluvial channel composed of non-uniform sediment with downward seepage. An empirical equation is generated for the prediction of the bedload transport of the sediment particles in non-dimensional forms considering downward seepage as an explicit variable. The influence of seepage on the non-uniformity of bed material is presented. Also, the present chapter explore the time dependent sediment transport with the whole permeable bed and develop the the empirical expression in order to relate the bedload transport rate with time.

Chapter 6 presents important conclusions of the research work. Scope for the future research work is also presented after the section conclusion. Various references that have been reviewed for the present thesis work are listed at the end of the dissertation.

2 Experimentation

2.1 Overview

Flume experimental data are generally chosen for the experimental observations because of capability of controlling over flow parameters and bed materials (**Chen and Stone, 2008**). Field investigations normally contain several confusing issues such as dimension of bed material distributions, parameter channel geometry and flow environments that influence the data quality. A wisely planned flume experiment can avoid most of the said problems and offer a more quality data set. Therefore, hydraulic engineers depend on the empirical relations, developed based on the experimental measurements, and to design the hydraulic structures. Sound physical basis for an empirical formulation is important in that it may have a far greater significance in solving complex problems. Various flow phenomena can be interpreted by experiments in laboratory with suitable dimensional analysis together with the process of reasoning.

As already discussed in Chapter-1, the objective of the present work is to analyze the effect of no seepage and seepage flow on various aspects of non-uniform sand bed channels such as boundary layer development, bedload transport, flow resistance, sheet flow layer, and changes in the turbulent characteristics of flow. An experimental approach can be used to simulate the behavior of the channel morphology in different given scenarios. To observe the overall fundamental effect, various hydraulic conditions can be provided by changing bed material, bed slope, flow rate, and seepage velocity. The objective of the present chapter is to describe the equipment and laboratory methods used for conducting experiments in the present study. Experiments have been designed to make maximum use of the equipment available while minimizing problems such as channel wall effects. All relevant material relating to the methods of operation of this equipment will also be introduced in this chapter. This chapter first describes the apparatus that was available before describing the manner in which it was used for gaining

various results. The end of the chapter describes in detail the different series of experiments that were planned and carried out during the course of the research.

2.2 Bed Material

River sands were used as the bed material in all the experimental runs. Sieve analysis was carried out to determine the particle size of the sand (Figure 2.1). Particle size distribution for given sand can be considered as non-uniform if the value of geometric standard deviation (σ_g) is greater than 1.4 (**Marsh et al., 2004**). Geometric standard deviation is calculated as:

$$\sigma_g = \sqrt{\frac{d_{84}}{d_{16}}} \quad (2.1)$$

Where, d_{84} (d_{16}) is defined as the size of grain for which 84% (16%) of the sample is finer by weight. **Kramer (1935)** proposed a uniformity parameter M that is defined as the ratio of the median sizes of the two portions in the particle size distribution curve separated by the median particle size d_{50} :

$$M = \frac{\sum_0^{50} d_i \Delta p_i}{\sum_{50}^{100} d_i \Delta p_i} \quad (2.2)$$

Where, d_i is the geometric size of two consecutive size fractions and Δp_i is the percentage of material present in the sediment mixture. Kramer's uniformity parameter $M = 1$ for uniform sediment and $M < 1$ for non-uniform sediment. Median diameter (d_{50}), geometric standard deviation, and dry angle of repose were obtained as given in Table 2.1 for the sand used in this study. Thus, both the sands are considered non-uniform.

Table 2.1: Physical characteristics of bed material

S. No.	Bed material	d_{50} (mm)	Standard deviation (σ_g)	Kramer coefficient (M)	Dry angle of repose (φ degree)
1	River Sand	0.425	1.79	0.14	33.04
2	River Sand	0.50	1.65	0.16	33.13

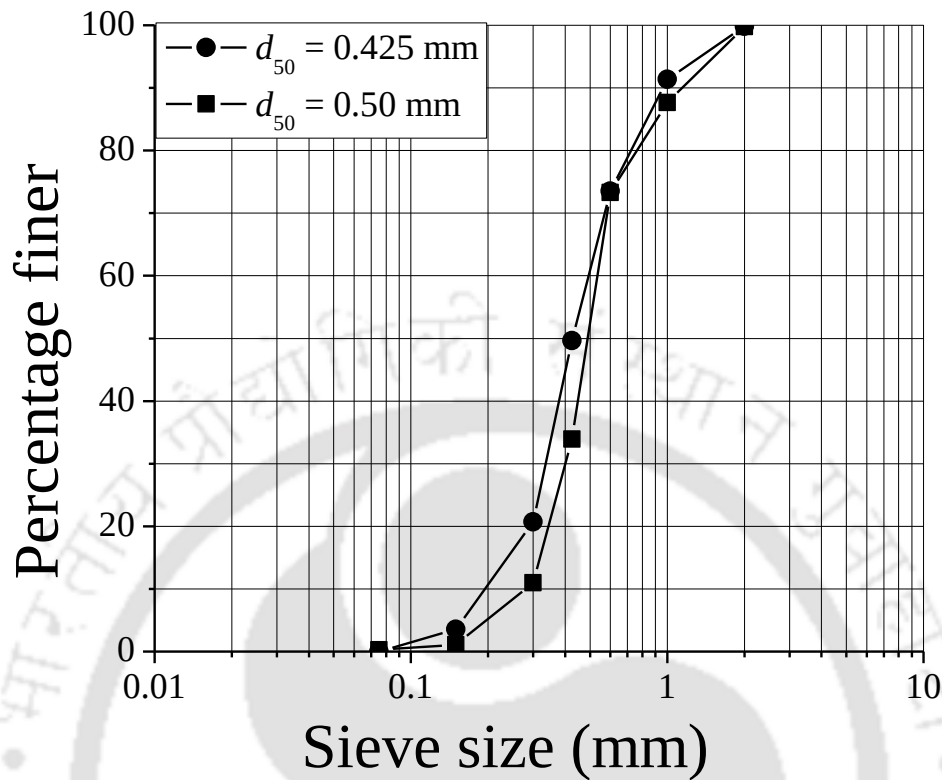


Figure 2.1 Particle size distributions of bed materials

2.3 Tilting Flume

The experiments were performed in a 20 m long, 1 m wide, and 0.72 m deep recirculating transparent plexi-glassed tilting flume (Figures 2.2 and 2.3). Two metres of upstream length of the main channel bed was made non-porous and the remaining length of the channel was made porous by covering a fine mesh (0.1 mm). This mesh arrangement was supported by steel tube structure of 0.22 m height which was placed on the bottom of the bed. Bottom pressure chamber (15.20 m in length, 1 m wide and 0.22 m deep) was formed by the area between the bottom of the channel and the fine mesh. Non-uniform river sand was used as bed material in the experiments, and was placed on the fine mesh in order to prevent its entrance into the bottom pressure chamber. The seepage chamber was used to extract water through the sand bed (uniformly) in the form of downward seepage. The amount of downward seepage was controlled by valves connected to the bottom pressure chamber at the downstream end. A tank of dimensions 2.8 m long, 1.5 m wide and 1.5 m deep was provided at the upstream of the flume to

straighten the flow prior to its introduction in to the flume. The upstream tank collected the water through the overhead tank at which water was driven by three centrifugal pumps of 10 HP capacities each. The flow in the main channel was regulated by a control valve positioned at the overhead tank. A couple of wooden baffles were fixed in the upstream tank in order to prevent turbulence in the flow that comes from the overhead tank to enter into the channel. Main channel discharge was measured using a rectangular notch provided at the downstream collection tank. An adjustable tail gate was provided at the downstream end of the flume to maintain the required depth of flow in the channel during experiments. A photograph of the side view of the experimental tilting flume is shown in Figure 2.4.



Figure 2.2 Photograph of the tilting flume

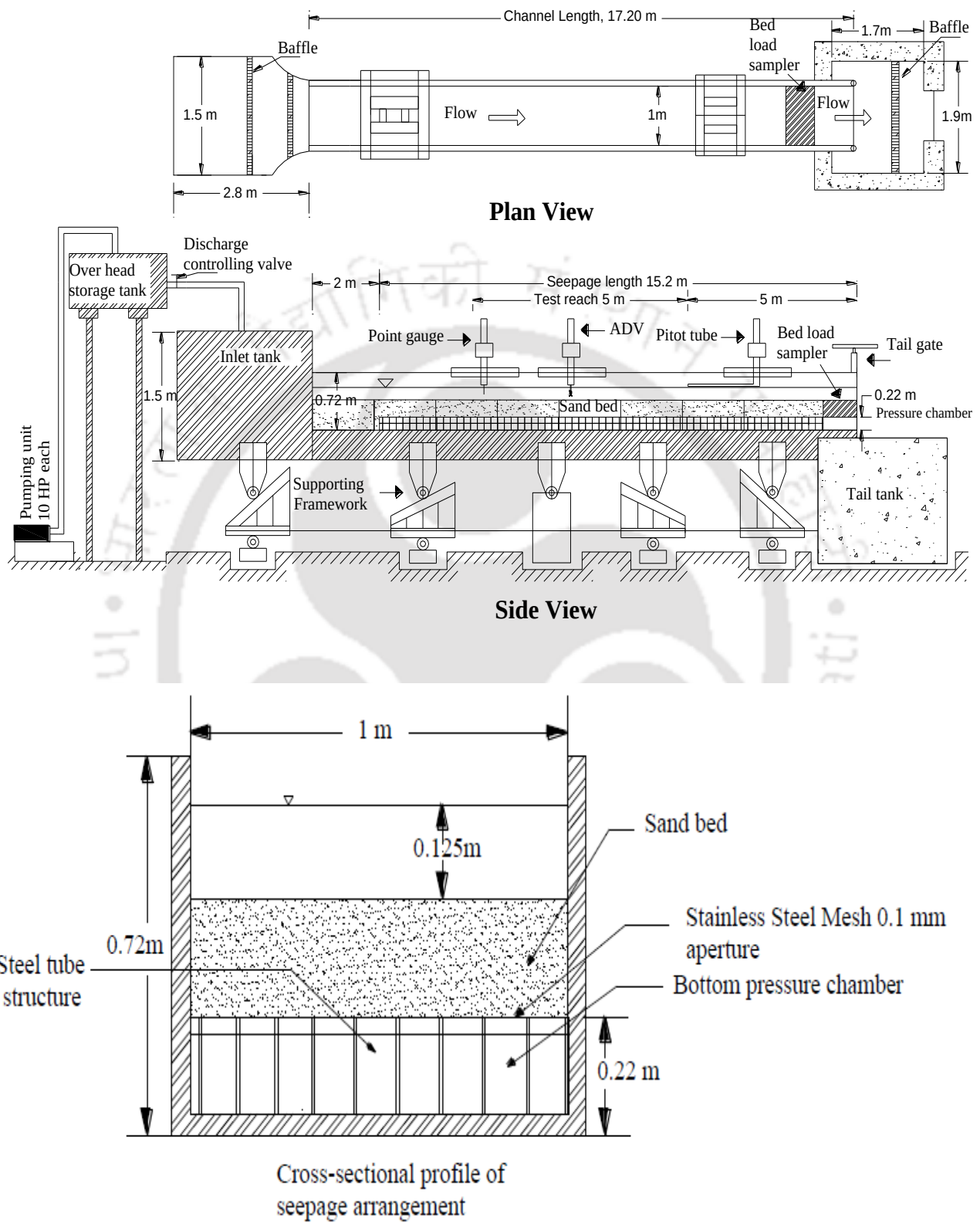


Figure 2.3 Schematic diagram of tilting flume

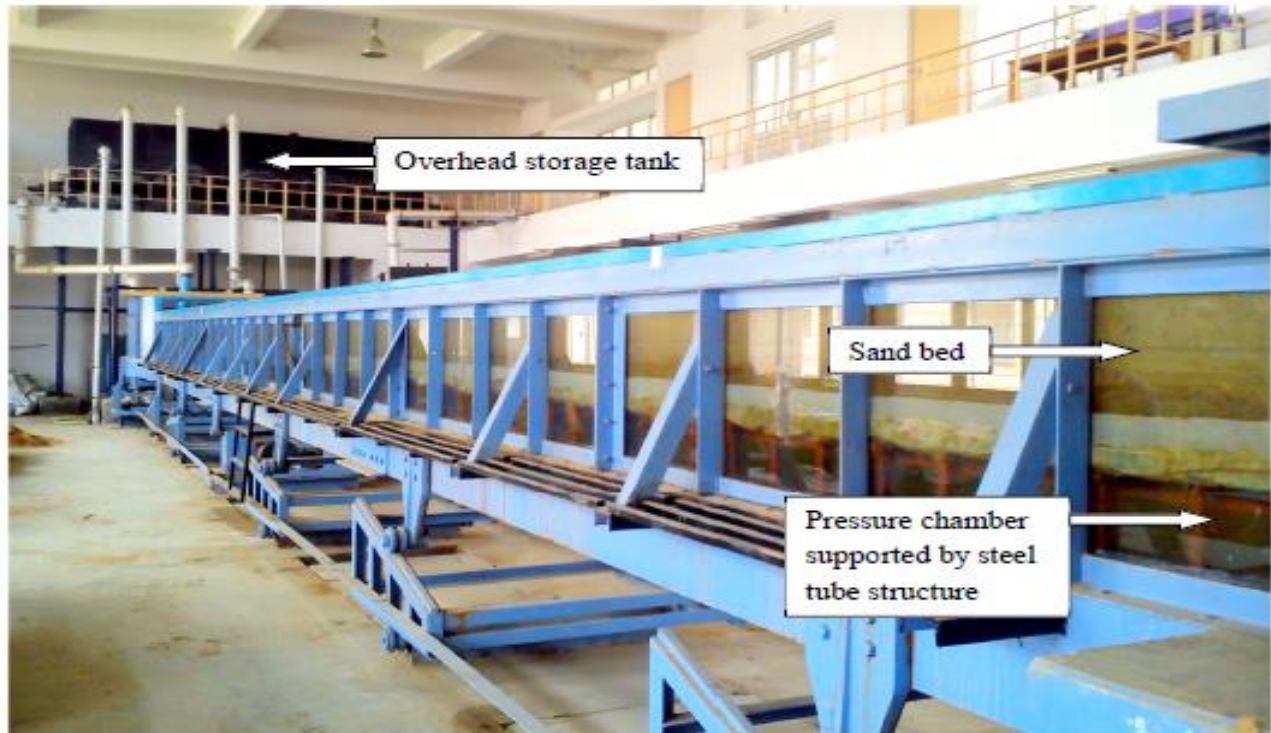


Figure 2.4 Photograph of side view of the experimental flume

2.4 Test Section

Flow in the test section of the flume is significantly affected by the entrance and exit conditions. Strong circulations are present in the flow if it is delivered to the flume directly by means of pipes. In order to avoid the strong circulations present in the flow due to pipes, water was first collected into upstream collection tank of the flume. The level of water rise gradually in the upstream collection tank prior to its introduction into the channel. To ensure the smooth entrance of flow into the channel, couple of baffles was installed in the upstream collection tank just upstream of the channel entrance. Free overfall of flow from the tailgate causes acceleration of flow in the near-bed region just upstream of the tailgate. Figure 2.5 shows the length of test section considered between two sections of the flume. To minimize the effects of flow entrance and exit conditions in the channel, the test section in the present experiments was considered as 8 m length in the middle of the flume (4 m to 12 m from tail gate as shown in Figure 2.5). The flume was long enough to achieve fully developed flow conditions for all the experiments. Measurement of geometry and flow were taken in this proposed test section only.

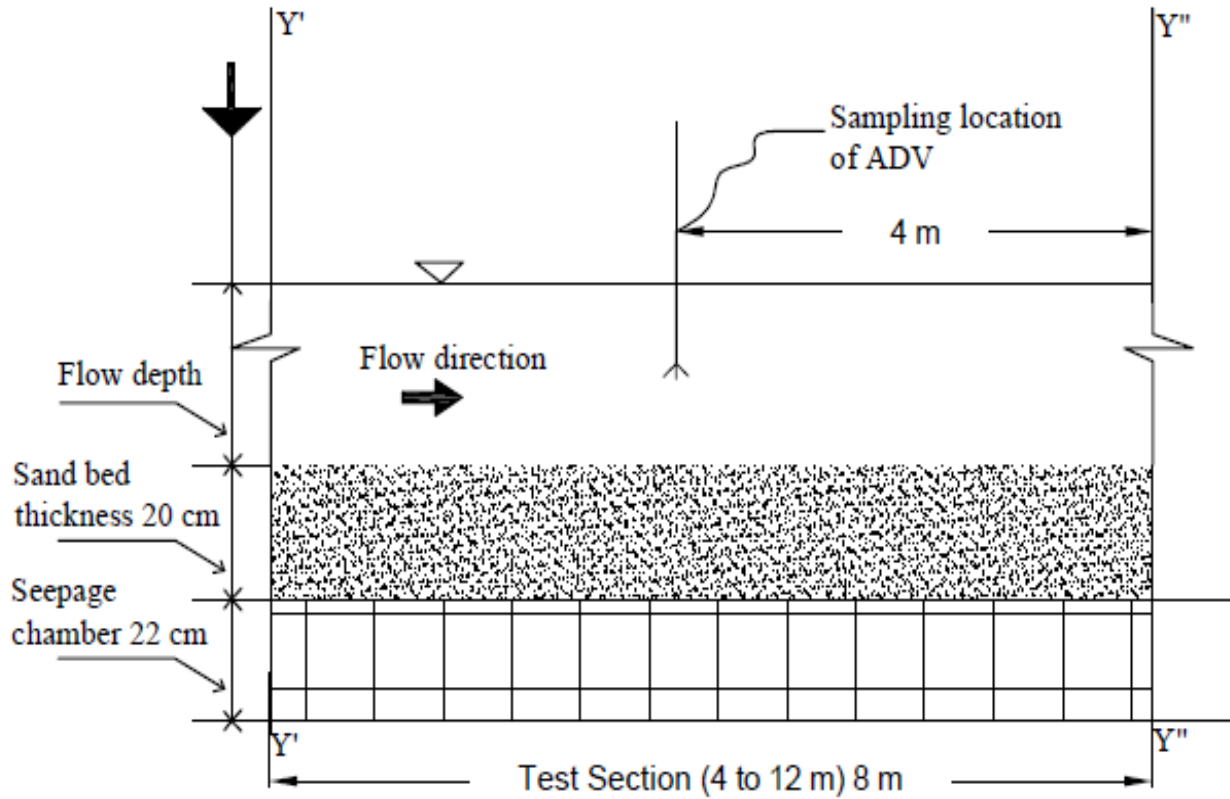


Figure 2.5: Test section considered in the flume

2.5 Main Channel Discharge

A regulating valve was used to supply a controlled amount of water in the main channel connected with the overhead tank. The discharge (Q_0) in the main channel was measured volumetrically with the help of a rectangular notch provided at the downstream of the flume (Figure 2.6). Weir and baffle walls were installed in the downstream collection tank to get smooth flow over rectangular notch. Coefficient of discharge (C_d) for the rectangular notch has been shown in Figure 2.7 which was found to be 0.82, can be calculated as:

$$Q_0 = \frac{2}{3} C_d L_n \sqrt{2g} H^{1.5} \quad (2.3)$$

Where, Q_0 is the discharge over the notch, L_n is the length of the notch ($L_n = 0.5$ m), g is the gravitational constant, and H is the height of water above the notch.



Figure 2.6: Measurement of flow discharge in the main channel discharge

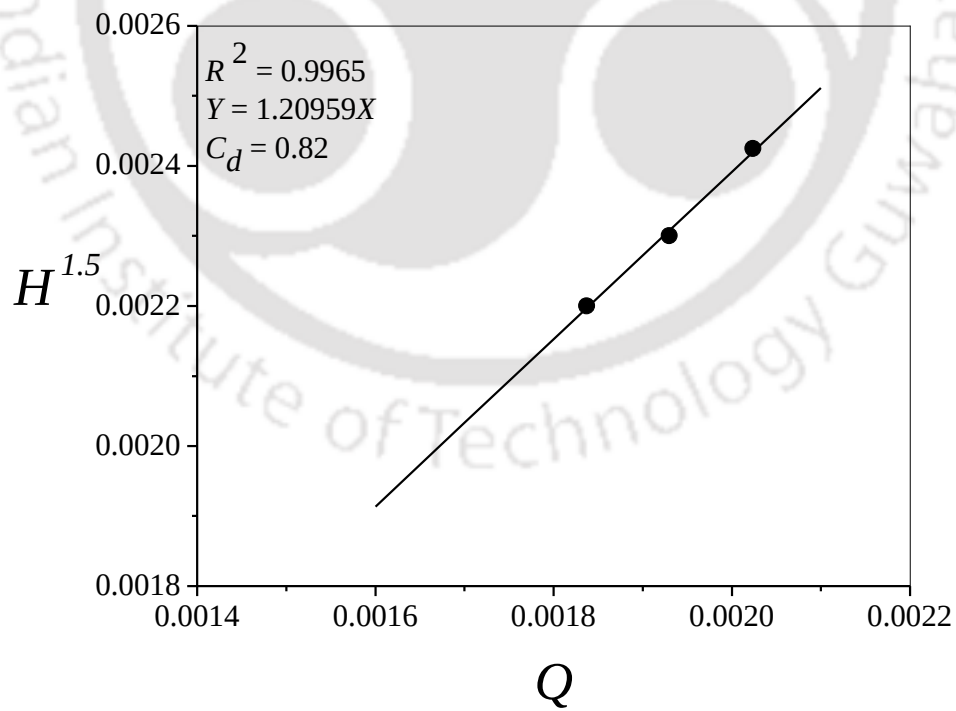


Figure 2.7: Calculation of coefficient of discharge for the rectangular notch

2.6 Seepage Discharge

The seepage discharge (q_s) coming from the seepage chamber was measured with the help of a pair of electromagnetic flow meters installed at the downstream of the flume. The flow meters were connected through the pipes to the seepage chamber. The electromagnetic flow meters are presented in Figure 2.8. The flow meters worked on the principle of Faraday's law of electromagnetic induction. According to the Faraday's law, the voltage induced across any conductor when it moves at right angles through a magnetic field is proportional to the velocity of that conductor.

$$E = f(u_c, B_s, D) \quad (2.4)$$

Where, E is the voltage generated in a conductor, u_c is the velocity of the conductor, B_s is the strength of the magnetic field, and D is the length of the conductor.

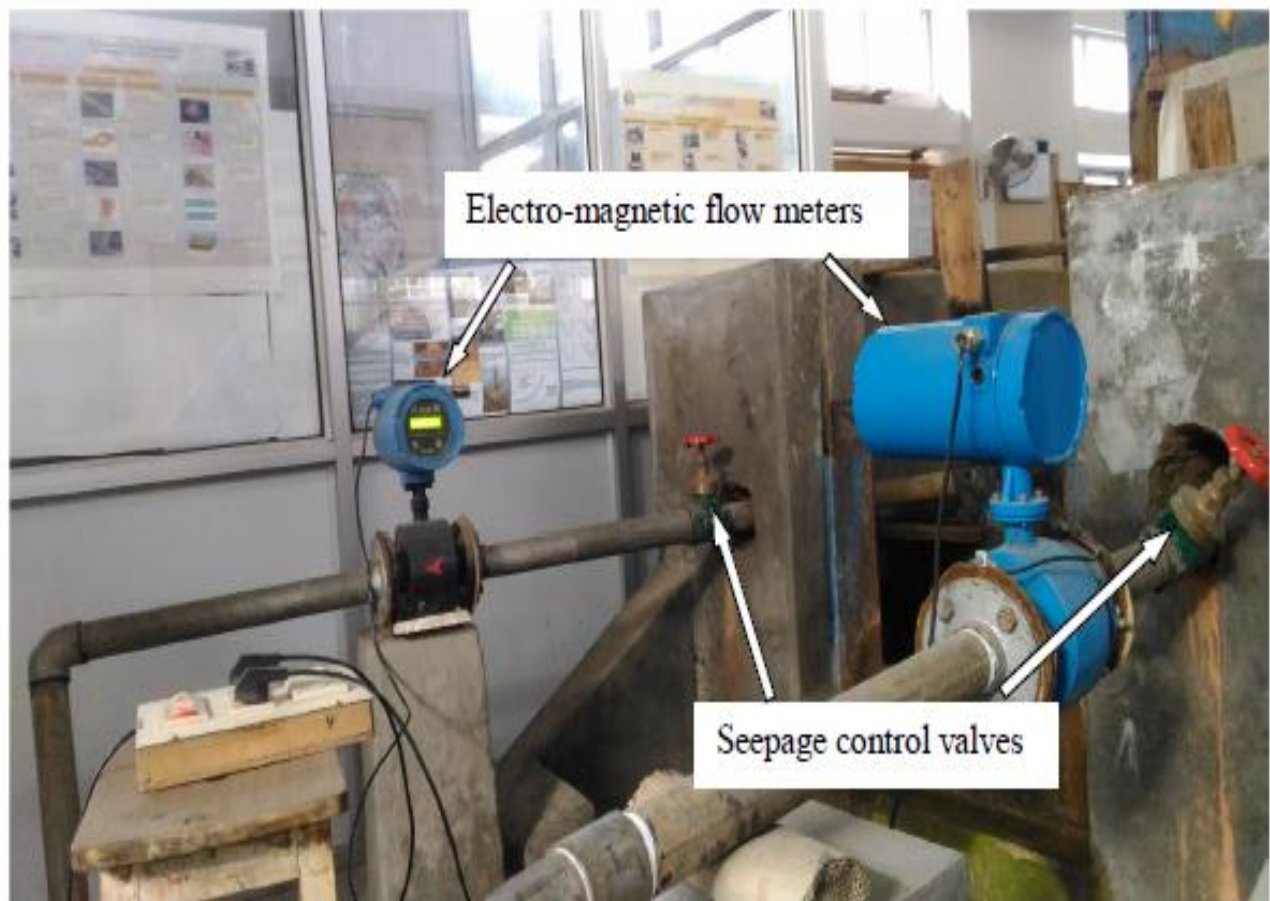


Figure 2.8: Electro-magnetic flow meters for measuring seepage discharge

The fluid being measured must be electrically conductive for the Faraday's law to apply. The basic condition is that the fluid should be electrically conductive for the application of Faraday's law. For measuring the flow discharge, an electro-magnetic field is generated in the whole cross section of the flow pipe. If this magnetic field is considered as the measuring element of the magnetic flow meter, it can be seen that the measuring element is exposed to the hydraulic conditions throughout the entire cross-section of the flow meter. The seepage discharge q_s can be calculated using the following relationship:

$$q_s = A_p u_c \quad (2.5)$$

Where, q_s is the downward seepage discharge (m^3/s), A_p is the area of pipe (m^2) and u_c is the fluid velocity through the flow meter (m/s).

2.7 Flow Depth

The flow depth in the channel was measured with the help of a digital point gauge attached to a moving trolley (Figure 2.9). This arrangement can move in transverse and longitudinal directions to cover whole cross-sectional area along the test section. This was a direct indicating gauge which eliminated observation errors due to vernier and scale reading. It could be set to zero anywhere in the operating range to permit easy relative level checking. The liquid crystal display was easy to read and had a resolution of $\pm 0.01\text{mm}$. A quick-release mechanism permitted rapid changes of position. The depth of flow was defined as a difference between the water surface level and the bed level.

2.8 Water Surface Elevation

Water surface slope in experiments was measured using a Pitot-static tube connected to a digital manometer which was further attached to a moving trolley (Rao, 2005). The digital manometer was powered by two batteries of 12 Volts each connected in series. The outer tube consisting of static pressure holes when connected to digital manometer gave the piezometric height at that point. Water surface slope was measured by moving the trolley in the streamwise direction. Pitot-static tube and digital manometer arrangement are shown in Figure (Figure 2.10).

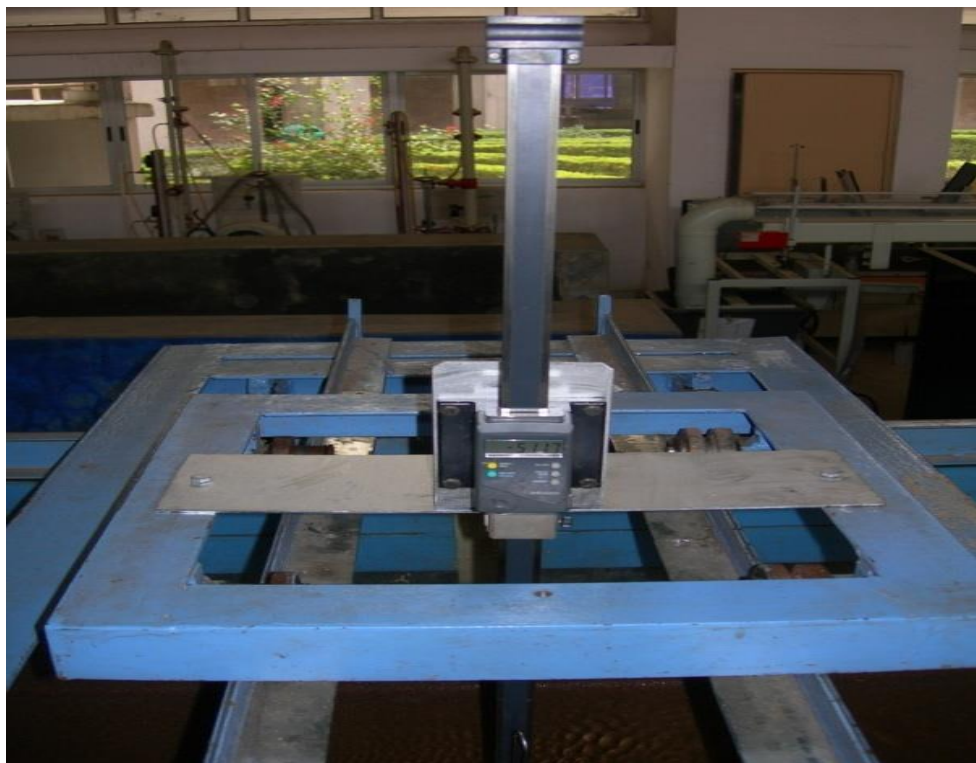


Figure 2.9: Digital point gauge for measuring flow depth

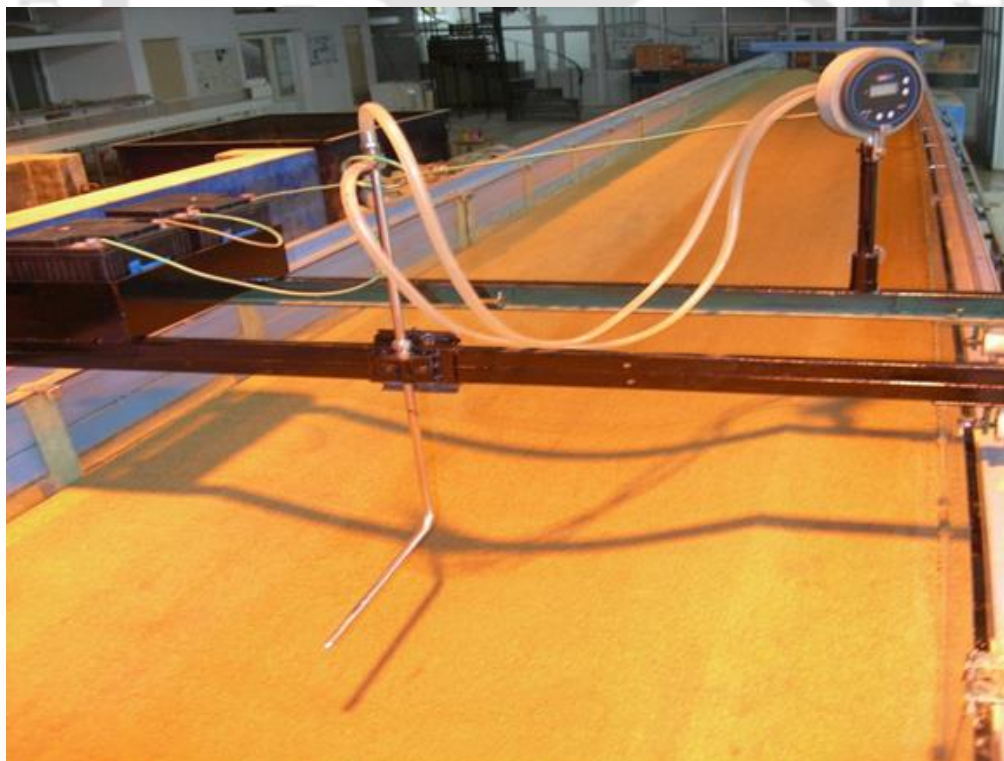


Figure 2.10: Pitot tube and digital manometer assembly for measuring water surface slope

2.9 Bed Slope

Bed slope of the flume was measured with the help of total station (Figure 2.11). A total station is a modern electronic device that combines the ability to simultaneously measure a position horizontally and vertically. It has two components, a machine mounted on a static tripod, and a 'target' prism on a metal staff, which was moved around the site. The machine part of the total station has a lens somewhat like a telescopic rifle-sight with cross-hairs which were focused on the prism. The whole instrument revolved horizontally and the lens swiveled vertically too. The total station was partly based on a principle used in traditional theodolites, where angles were calculated from vertical and horizontal 360-degree scales. It combined these with a device known as an Electronic Distance Measurer or EDM.



Figure 2.11: Total station for measuring bed slope

2.10 Geometric Measurements

The geometry of the main channel was measured using a SeaTek 5 MHz Ultrasonic Ranging System that contains eight transducers attached to an automated moving trolley (Figure 2.12), which moves on a rail at constant speed. The distance between each transducer was 25 mm center to center in transverse direction. The trolley was designed using four high torque DC geared motor connected to the wheels and speed was controlled with the help of Pulse width modulated DC motor speed controller switch in order to match the sampling rate of the transducers. The Seatek instrument uses 5 MHz ultrasound to measure the distance to a target. A transmit pulse of 10 microsecond duration is first sent from the transducer. The pulse travels through the water and is reflected off from a target. The reflected signal then travels back to the transducer. The transducer acts as both a transmitter and receiver. This return signal is then detected by the electronics. Since the speed of sound in water is known, and the time it took for the pulse to travel from the transducer to the target and back is known, the distance to the target can be calculated.

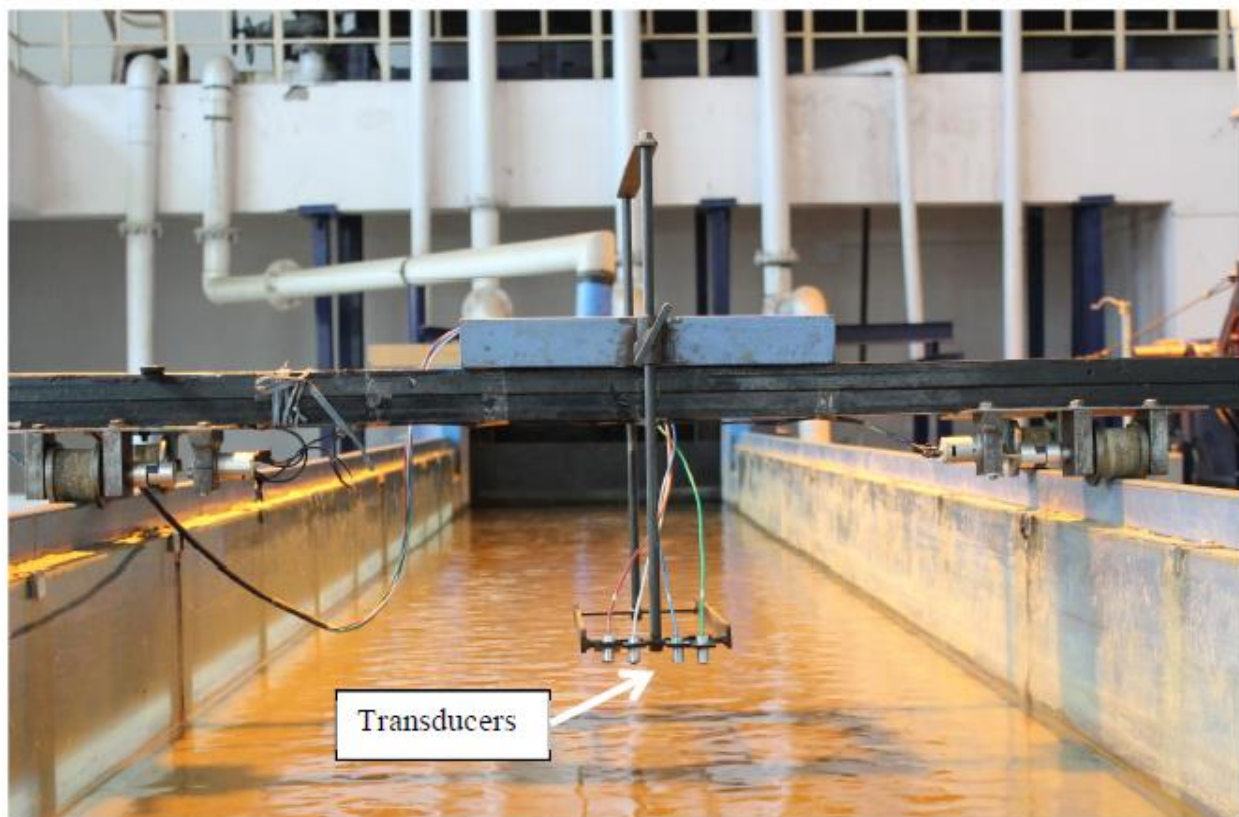


Figure 2.12: Transducers of SeaTek Ultrasonic Ranging System with automated moving trolley

Ultrasonic pulses transmitted by each probe were reflected off the sediment bed or water surface and received again by the probe. The distance d from the probe to the reflecting surface can be obtained as:

$$d = \frac{1}{2} C_s dt \quad (2.6)$$

Where, C_s is the celerity of the ultrasonic waves and dt is the measured time of the transmitted and received signal. The wavelength of 5 MHz sound waves in water is 0.3 mm. The resolution of the system is 0.1 mm; the accuracy (if several pings are processed per return) is +/- 0.2 mm. The limitation of the Ranging System is that it cannot measure the depth less than 3 cm and more than 100 cm.

2.11 Uncertainty Analysis

To obtain the accuracy in the measurements and data collection, uncertainty analysis associated with the measured data has been carried out before performing the experiments. Uncertainty for the measurements of main channel discharge using rectangular notch (Q_0), seepage discharge from the electro-magnetic flow meter (q_s) and geometry measured from the SeaTek Ultrasonic Ranging System were analysed by using several repeated readings from each of these instruments. Transducers for distance measurement were kept over the water surface ensured to touch the water surface and the results are obtained as shown in Table 2.2.

Table 2.2: Uncertainty analysis of the data collection

Statistical parameters	Q_0	q_s	h
Standard deviation	0.00068	0.00036	0.00092
Standard uncertainty	0.03478	0.03768	0.0184

2.12 Instantaneous Flow Measurement

For investigating the turbulent properties of the flow, instantaneous velocities were measured. Instantaneous velocity measurements were performed using a four beam down looking Vectrino+ Acoustic Doppler Velocimeter (ADV) probe manufactured by Nortek (Figure 2.13).

The instrument allowed data collection at a higher sampling rate up to 200 Hz. The Vectrino uses the Doppler Effect to measure the velocity. The Doppler Effect is the change in frequency of a wave (or other periodic event) for an observer moving relative to its source. The pitch of a sound is different when a vehicle sounding a siren approaches, passes and recedes from an observer. Compared to the emitted frequency, the received frequency is higher during the approach, identical at the instant of passing by, and lower during the recession. The change in the pitch will give an idea of how fast the vehicle is moving. The ADV used this principle to measure the velocity of water in three dimensions. Vectrino transmits short pairs of acoustic pulses, listens to their echoes and ultimately measures the change in pitch or frequency of the returned sound.

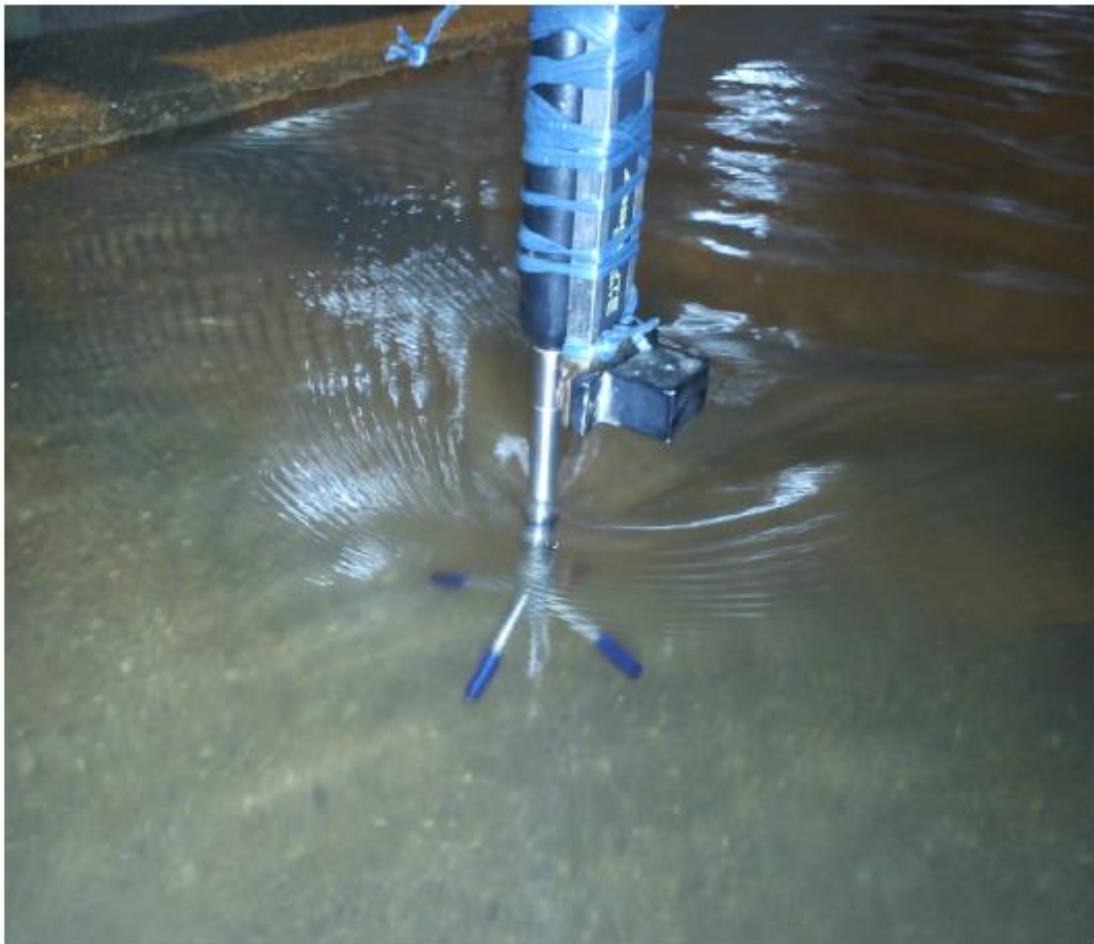


Figure 2.13: Acoustic Doppler Velocimeter for measuring instantaneous velocities

The ADV uses a pulse-to-pulse coherent Doppler methodology to record the sample the flow velocities (**Brumley *et al.*, 1991**). This sound is not actually reflected from fluid particles. It is

rather reflected from the small particles or the seeding particles contaminated in water, which move with same average speed as of water. Consequently, the measured velocity is the velocity of the water. In a separate echo sounder mode, the ADV also allowed the measurement of the distance of the central transmitter from the boundary. The instrument collects data in a cylindrical remote sampling volume of user adjustable height located 5 cm below the central transmitter, thereby; the data could not be collected in the top 5 cm zone from the water surface (Figure 2.14).



Figure 2.14: ADV and the remote sampling volume (Image source: Vectrino user guide)

The software called Vectrino+ developed by Nortek was used for collecting the instantaneous velocities in the computer system (Figure 2.15). A sampling rate of 200 Hz was used for the data acquisition. It worked with an acoustic frequency of 10 MHz having an adjustable cylindrical sampling volume of 6 mm diameter and 1–4 mm height. The height of the sampling volume was set at 4 mm when the measurement location was away from the bed and 1 mm when very near to the bed so that the sampling volume did not touch the particles on the bed surface. The sampling

length in the near-boundary flow was set with a lowest height of 1 mm. In recent study (**Dey et al., 2017**), a sampling length of 1 mm was found to be adequate to capture the correct descriptions in the shear layer and near-boundary zones. Velocity measurements were carried out in a vertical profile and at each location, instantaneous velocity samples were collected for the duration of 5 minutes. A sampling time of 300 s was found to be sufficient to achieve the statistically time-independent time-averaged velocity. Velocity data was collected in the test section at the center line of the channel cross section at a distance of 8 m from the downstream end of the flume. Regarding the coordinate system, z-axis ($z = 0$) was chosen as the reference of bed surface and thus it was considered positive in the vertically upward direction. The x-axis was associated with centerline of the channel and $x = 0$ indicated the measurement location and was taken as positive in the stream wise direction. The transverse axis y was taken positive in the right. Therefore, the data measuring location had coordinates (0, 0, z).

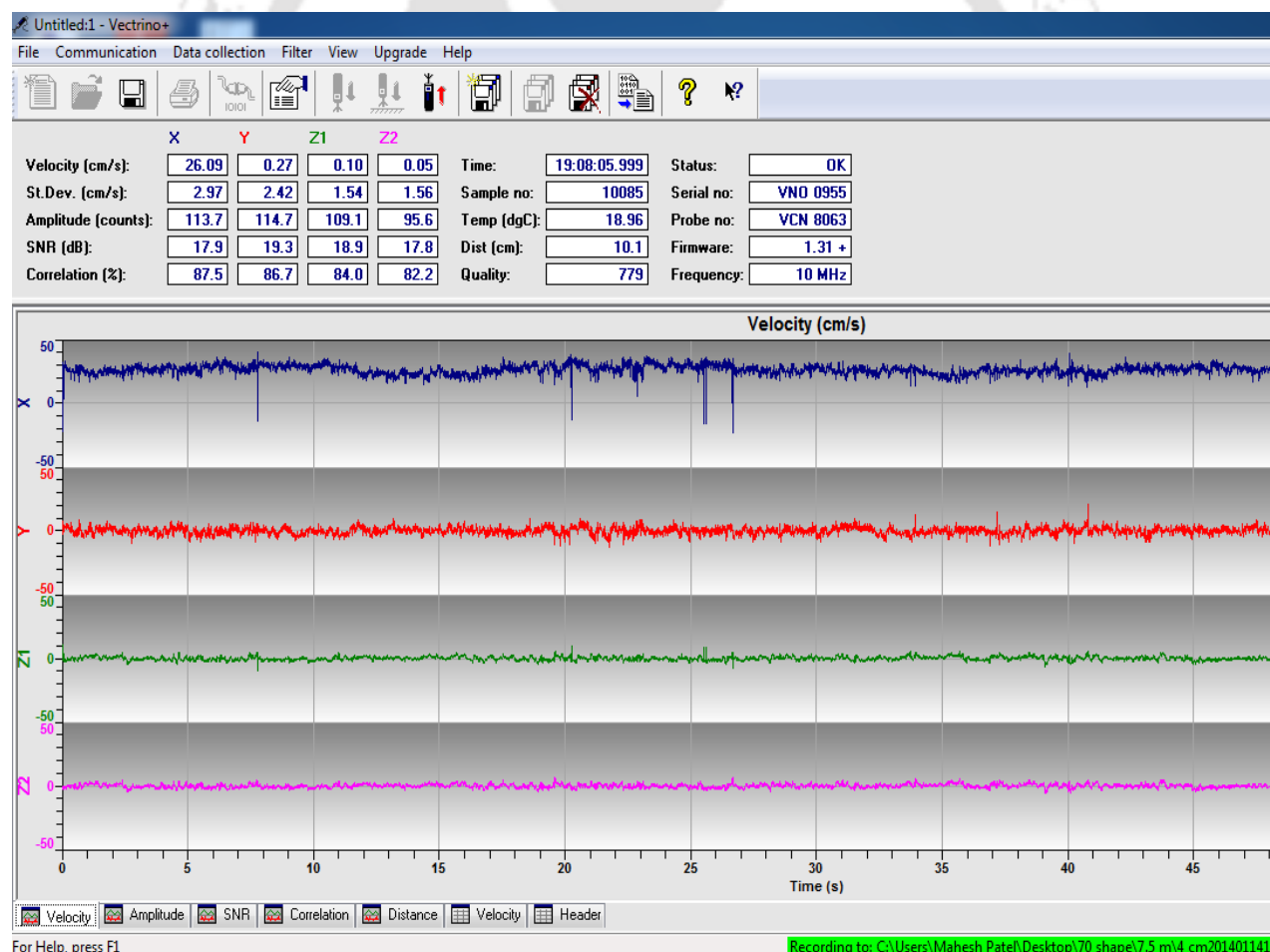


Figure 2.15: Vectrino+ software for data collection

To analyze uncertainty associated with the ADV measurements, 10 pulses were collected very near to the channel bed ($z \sim 10$ mm) each pulse had the duration of 5 minutes, where z is the distance of measurement point from the channel bed (positive in the vertically upward direction). Table 2.3 shows the values obtained from uncertainty analysis, where u , v , and w are the time-mean velocities in the streamwise, transverse, and vertical directions, respectively. Here, the term standard uncertainty of the mean describes the standard deviation of the mean for a set of several repeated pulses of instantaneous velocities. Standard uncertainty is calculated as:

$$S_u = \frac{S_d}{\sqrt{n}} \quad (2.7)$$

Where, S_d is the standard deviation and n is the number of measurements in the set. The data shown in Table 2.3 are within $\pm 5\%$ error for the time-averaged velocities and rms quantities, corroborating the capability of 200 Hz frequency of measurements by Vectrino (**Dey et al., 2012**)

Table 2.3: Uncertainty associated with ADV data

	u	v	w	$(\overline{u'u'})^{0.5}$	$(\overline{v'v'})^{0.5}$	$(\overline{w'w'})^{0.5}$
Standard deviation, S_d	4.31×10^{-3}	9.62×10^{-4}	4.33×10^{-4}	1.05×10^{-3}	9.32×10^{-4}	3.44×10^{-4}
Standard uncertainty, S_u	1.36×10^{-3}	3.04×10^{-4}	1.37×10^{-4}	0.33×10^{-3}	2.95×10^{-4}	1.09×10^{-4}
Uncertainty %	0.33	0.061	0.85	0.091	0.072	0.041

In all the experiments, signal to noise ratio (>15) and correlation amongst transmitted and received signals (>70) were recommended as a cutoff value. In the region very close to the channel bed, correlations of the samples were reduced approximately by 5% (**Dey et al., 2012**). The measured pulses of instantaneous velocities contained some spikes because of interference between transmitted and received signals. These data were needed to be post processed or filtered. Acceleration thresholding method (**Goring and Nikora, 2002**) is used for removing the spikes in the velocity data. Threshold values (1-1.5) were selected by trial and error in such a way that the velocity power spectra in the inertial subrange yielded a satisfactory fit with

Kolmogorov “-5/3 scaling law” (Lacey and Roy, 2008). Velocity power spectra for the no seepage and seepage runs are depicted in Figure 2.16 (at $z \sim 10$ mm), where F_{uu} is power spectra for the streamwise component of the velocity, f is frequency in Hz, and z is distance from bed surface. It is observed from Figure 2.16 that for both no seepage and seepage case, velocity power spectra for filtered data is in good agreement with the Kolmogorov’s -5/3 law in the inertial sub range. Further, close examination of Figure 2.16 indicates that power spectra for no seepage and seepage runs are similar suggesting that spectra are not influenced by the seepage.

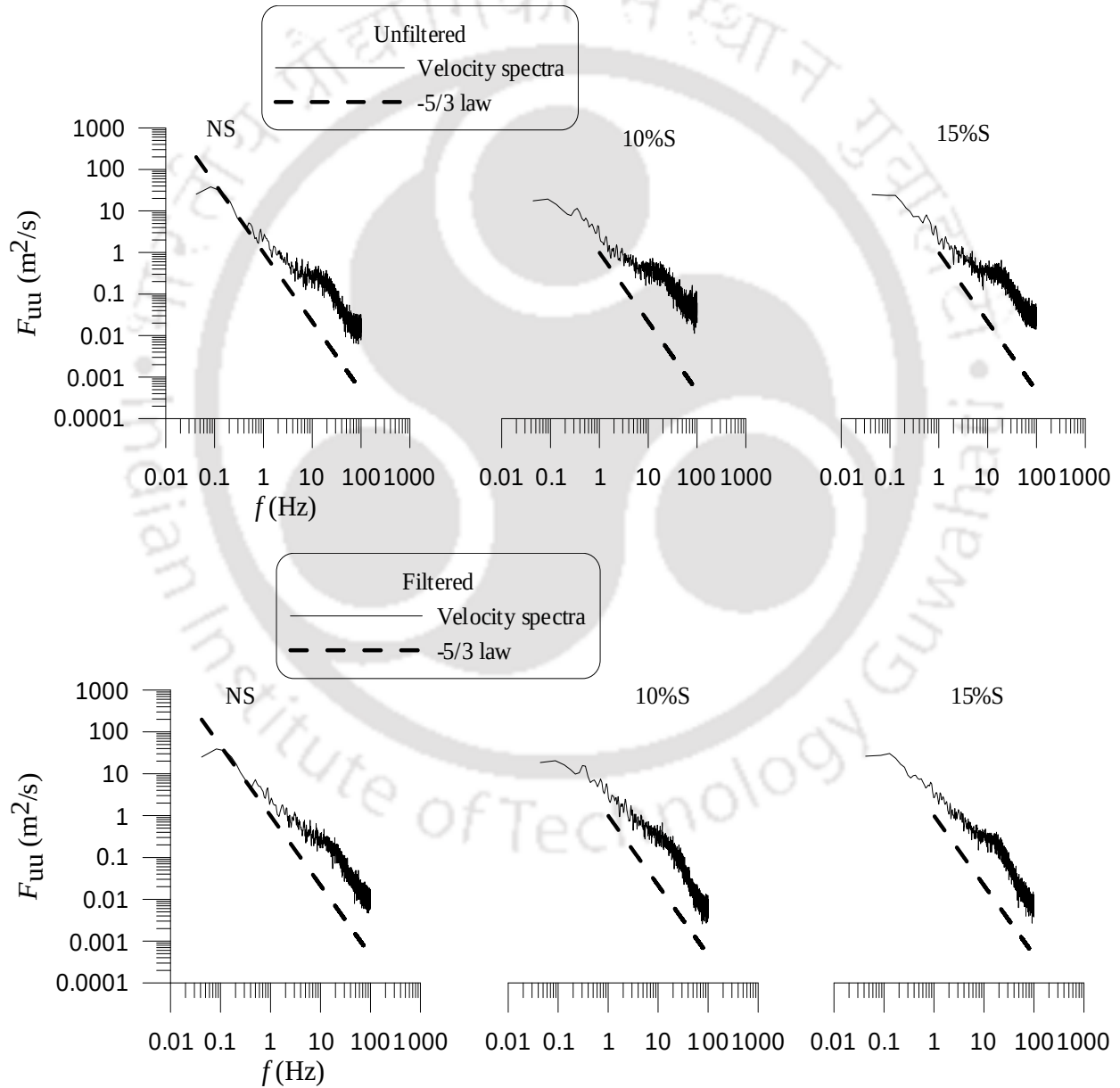


Figure 2.16: Streamwise velocity power spectra before and after spike removal

2.13 Temperature and Kinematic Viscosity

Average temperature of water during an experiment was recorded by Vectrino+ ADV which had a temperature sensor (thermistor), located inside the probe head. The corresponding value of kinematic viscosity (m^2/s) was determined from standard tables.

2.14 Experimental Methodology

The experimental framework was designed for two types of experiments: no seepage and seepage experiments. In the no seepage (NS) run, water is introduced to the channel by gradually opening a valve located at the overhead tank till required, inflow discharge and corresponding flow depth are registered. In no seepage experiment, i.e. particles were on the movement without providing or extracting water through the channel bed. At this stage, measurement of inflow discharge, water surface slope, and geometry of the cross section were recorded. Consequently after conducting the experiment with no seepage, some percentage of downward seepage (some percentage of no seepage inflow discharge) was applied by controlling the electromagnetic flow meters installed at the downstream section of the flume and it was called as seepage experiment. In seepage experiments, i.e. water was extracted in the downward direction from the non-uniform sand bed channel through the channel boundary, without any increment of inflow discharge. Detailed description of both types of experiments is given below.

2.14.1 No Seepage Experiments

The non-uniform sand bed channel was prepared with the help of a wooden channel (Figure 2.17). Special care was taken to prepare the channel to ensure the uniform geometry of channel cross section along the length of the channel (Figure 2.18). In the wooden channel, the levelling and measuring of the bed was done with the aid of a smoothing board sliding between the side rails. In the no seepage experiment, discharge was slowly introduced to the channel so that the average value of the shear stress increase gradually until the hydrodynamic forces of the flow were greater as compared to the resistive forces of the bed particles (on the bed surface). At this state, the sediment particles were at the mobile condition. Particles were eroded from the bed and deposited in the adjacent sections in the form of bed features such as sheet layers till the whole cross section was deformed.



Figure 2.17: Steel frame for preparing flat non-uniform sand bed channel in the flume



Figure 2.18: Snapshot of the channel before performing the no seepage experiments

These experiments were continued for several hours until the channel geometry and physical characteristics of bed-features reached in the equilibrium condition depending upon sediments

property such as non-uniform sands. No seepage experiments were performed on non-uniform sand beds in order to observe the effect of flow on non-uniform sand bed channel.

The average value of the shear velocity is evaluated as:

$$u_{*0} = \sqrt{gR_{h0}S_{f0}} \quad (2.8)$$

Here, subscripts '0' represents no seepage experimental conditions. Where, g is gravitational acceleration, R_{h0} is hydraulic radius, and S_{f0} is friction slope and it can be calculated as:

$$S_{f0} = S_{w0}(1 - F_0^2) + S_0F_0^2 \quad (2.9)$$

Where, F is Froude number and S_w is water surface slope. Measurements of cross-sectional geometry, water surface slope, inflow discharge (Q_0), and flow depth (h) were taken during the experiment. The cross-sectional geometry along the length of the main channel was measured using ultrasonic ranging System.

2.14.2 Seepage Experiments

For investigating the influence of downward seepage on a non-uniform sand bed channel, water was extracted through the channel bed in the form of downward seepage after achieving the no seepage discharge. It was observed that the sediment bed reached its highly mobile condition. Particles were eroded from the bed and deposited in the adjacent sections in the form of bed features such as sheet layers till the whole cross section was deformed. These experiments were continued for several hours until the channel geometry and physical characteristics of bed-features reached its equilibrium condition depending upon sediments property such as non-uniform sands. Seepage experiments were performed on non-uniform sand beds in order to observe the effect of downward seepage flow on sand bed channel.

Seepage experiments are examples of spatially varied flow in nature. The friction slope of spatially varied flow can be given as **(Rao *et al.*, 2011)**:

$$S_{fs} = S_{ws}(1 - F_s^2) + S_0F_s^2 + 2\rho_w u_s V_s / \gamma h_s \quad (2.10)$$

Here, subscript 's' stands for seepage experiments. In Equation (2.10), S_w is water surface slope, and S_0 is bed slope, u_s is the average velocity of flow:

$$u_s = \frac{Q_s}{A} \quad (2.11)$$

A is the cross-sectional area, Q_s is average discharge:

$$Q_s = Q_0 - q_s \left(1 - \frac{X}{L} \right) \quad (2.12)$$

Q_0 is discharge for no seepage run, q_s is the seepage discharge over the reach length L of the channel, and X is the distance from the tail gate. F_s in Equation (2.12) is the Froude number:

$$F_s = \frac{u_s}{\sqrt{gR_{hs}}} \quad (2.13)$$

where, g is gravitational acceleration, R_{hs} is hydraulic radius, $R_{hs} = As/p_s$; p_s is wetted perimeter. The bed slope S_0 is considered to be the same in all of the runs in a given set, both under no seepage and seepage conditions; the water surface slope with seepage S_{ws} is found to be fairly linear in the test reach. In Equation (2.10), V_s is the seepage velocity through the sand bed of length L :

$$V_s = \frac{q_s}{p_s L} \quad (2.14)$$

Shear stress τ_s for seepage experimental conditions can be found as:

$$\tau_s = \gamma R_{hs} S_{fs} \quad (2.15)$$

The assumption of hydrostatic distribution taken in Equation (2.15) is reasonable in case of rectilinear, quasi-steady and quasi-uniform flows (**Francalanci et al., 2008**). **Francalanci et al. (2008)** also argued that by its very definition the Shields number assumes the pressure distribution of the water in which sediment grains are immersed to be hydrostatic.

2.14.2.1 Criteria to Consider the Seepage Velocity

In the Section 1.3, studies suggested that alluvial channels can have a wide range of seepage (approximately 10-50% of main channel discharge) depending upon the field specific conditions. These studies are performed on a field i.e. on a large scale, thereby; exact scaled model cannot be used in the experimental study. Therefore, the aim is to give the appropriate amount of the seepage in the present experimental study. **Dey et al. (2011)** suggested the criteria to consider the seepage velocity laboratory on the basis of natural system. They defined that the magnitude of seepage velocity can be taken as 1% or less of the average stream flow velocity. Furthermore, many experimental studies were performed on different seepage flow rates in terms of seepage velocities to investigate the influence of seepage in alluvial channels. For example, seepage velocities were considered as 0.0 to 0.88 mm/s by **Richardson et al. (1985)**, 0.55 mm/s to 1.5 mm/s by **Faruque and Balachandar (2011)**, and 0.4 mm/s to 7.3 mm/s by **Cao and Chiew (2014)** in their laboratory studies. In accordance with the ranges suggested for volumetric losses of water due to seepage and corresponding seepage velocities, the downward seepage was maintained at 10% and 15% of the main channel discharge for non-uniform sand bed which are in accordance with the literature.

2.14.3 Experimental Program

The experimental program comprises a set of experiments on a non-uniform sand bed channel.

Table 2.4: Summary of the experimental program for the present study

Median sand size, d_{50} (mm)	Bed slope, S_0	Flow depth h (m)	Discharge, Q_0 (m^3/s)	Seepage discharge. q_s (% Q_0)	Number of Runs
0.50	0.001	0.105-0.119	0.0346-0.0538	0,10%,15%	30
0.50	0.0005	0.113-0.126	0.0346-0.0538	0,10%,15%	30
0.425	0.001	0.112-0.117	0.0402-0.0488	0,10%,15%	15
0.425	0.0005	0.116-0.123	0.0402-0.0488	0,10%,15%	15
Total number of runs					90

Width of the channel was fixed 1 m for all the experiments. Two types of sediment beds were used in order to investigate the effect of downward seepage on physical characteristics of

sediment. Seepage discharge and velocities were provided in accordance with the literature. Details of the experimental program are listed in Table 2.4.

2.15 Boundary Layer Development

The concept of boundary layer flow introduced by **Prandtl (1904)** is a popular field in fluid mechanics for engineers, physicists and mathematicians. Boundary layer is that portion of flow zone where motion of fluid is affected by the solid boundary. The thickness of boundary layer is defined as the distance from the sand bed at which flow velocity is 99% of free stream velocity. The boundary layer may be laminar at the upstream end, gradually thickens up to a certain point in the channel length at which the flow is in the stage of developing flow and beyond that point, the flow becomes fully developed flow. The flow details within the boundary layer are important for problems in aerodynamics together with the development of a wing stall, the heat transfer occurring in high speed flight, the skin friction drag of a body and the performance of a high speed aircraft inlet (**Schlichting, 1968**). The boundary layer flow can be laminar or turbulent, subsequent in radically different classes of profile shapes. **Prandtl (1952), Batchelor (1967) and Schlichting (1979)** provide thoroughly descriptions of the boundary layer concept. The behaviour of a solid moving relative to a flow cannot be accurately described without an understanding of the boundary layer. When the bed particle moves relative to the flow, the boundary layer occurs very close to the bed surface as a result of no slip condition and viscosity (**Prandtl 1904**). **Rohr et al. (1998)** have suggested that the thickness of the boundary layer may be linked to the relative intensity of bioluminescence nearby a swimming dolphin. **Allen (1961)** achieved a qualitative description around the flow zone and possibly the growth of boundary layer of fish by means of Schlieren technique. **Anderson (2001)** investigated the normal and tangential velocity profiles of the flow boundary layer surrounding live swimming fish by means of digital particle tracking velocimetry, DPTV. **Ardiclioglu and Ozturk (2006)** investigated the boundary layer development in smooth surface open channel flow. In their study, boundary layer thickness is proposed both by numerical and experimental method. In-situ measurements have become sufficient to investigate turbulent quantities in the atmospheric boundary layer because of remote sensing technologies (**Poulos et al. 2002; Kunkel and Marusic, 2006; Drobinski et al. 2007**). The boundary layer development is considered in the framework of a changeover of a well-developed boundary layer subject to abrupt change of wall roughness, also raised to as an

internal boundary layer, (**Cheng and Castro, 2002; Belcher *et al.*, 2003**). **Castro (2007)** checked the development of boundary layer in a wind tunnel over different roughnesses at relatively high Reynolds numbers. **Kervella *et al.* (2009)** studied boundary layer development in different geometric configuration and suggests some important areas of sediment transport modification. **Tomas *et al.* (2011)** experimentally investigate boundary layer development over a rough surface and proposed an empirical equation for boundary layer thickness. In recent study, **Devi *et al.* (2016)** studied the boundary layer development in the vegetative channel under the application of downward seepage.

The drawback of previous studies is that their work is restricted to the channel with local seepage (small seepage zone length as compared to the flume length), which does not satisfy the actual field condition as well as the natural rivers. In such a small seepage zone length, it may be difficult to achieve the fully developed flow condition. It is clear that the velocity and other flow profiles in the seepage affected alluvial channel which do not approach the fully developed flow condition has introduced the error due to a small seepage zone length used. Probably the small seepage zone length was not sufficient to capture the real shape of flow profiles because of which a large seepage zone length should be used. The seepage zone length is important in the study of the hydrodynamics of the channel because sediment transport rate increases with increase seepage zone length (**Cao and Chiew, 2014**). The occurrence of fully developed flow condition depend upon the flow developing length at which velocity depth profiles approximately remain same in the flow direction. The selection of test sections in the channel depends upon the length of the channel at which fully developed flow condition is achieved. Only in that test sections, we will achieve the correct result of flow profiles such as velocity, Reynolds shear stress, turbulent intensity etc.

Considering all the factors mentioned in the above, we have setup the experiments with large seepage zone length and to investigate the fully developed flow condition in the channel subjected to downward seepage. Although many studies have investigated boundary layer development in smooth and rough surface open channel flows, the boundary layer development of flow in flat sand bed comprised of non-uniform sediment with downward seepage remain unexplored. Since riverbeds are usually comprised of non-uniform sediment mixture (**wu *et al.*, 2004**), the present chapter therefore focuses on the influence of downward seepage on the flow

boundary layer over a mobile rough boundary (made of non-uniform sand of size 0.5mm). Also the present chapter develops the empirical equation for boundary layer thickness and flow development length under the application of downward seepage. The author hopes that this chapter will help investigators in the field of boundary flow and especially researchers to avoid the errors in the future.

2.15.1 Mean Velocity Field

Boundary layer development was investigated for the main flow discharge ($Q_0=0.0402 \text{ m}^3/\text{s}$) and median grain size ($d_{50}=0.5 \text{ mm}$). Flow velocities were measured at 12 longitudinal sections ($x=0.5, 1.5, 2.5, \dots, 11.5\text{m}$) along the flow developing zone. Mean stream wise (u) and vertical (w) velocities were calculated as:

$$\left. \begin{aligned} u &= \frac{1}{n} \sum_{i=1}^n U_i \\ w &= \frac{1}{n} \sum_{i=1}^n W_i \end{aligned} \right\} \quad (2.16)$$

Where, U_i and W_i are the instantaneous velocities in the stream wise and vertical directions respectively and n is the number of samples. Figure 2.19-2.21 shows the variations of time averaged stream wise velocity u with vertical distance z in flows subjected to no seepage and seepage for 10% and 15% at each measurement station ($x=0.5-11.5\text{m}$). Because of the limitation of water depth in the flume, the growth of boundary-layer is associated with a slight increase in the surface velocity to maintain the flow rate due to the increasing displacement thickness in the boundary layer. Mean stream wise velocity profiles fitted to log law region given by Equation (2.17) can be used to determine the roughness length z_0

$$\frac{u}{u_*} = \frac{1}{k} \ln \left(\frac{z + \Delta z}{z_0} \right) \quad (2.17)$$

Where, Δz is depth of virtual bed level from the bed surface, z_0 is zero velocity level, k is von Karman constant. The values of z_0 were also determined from the relationship of Nikuradse grain roughness height as $K_s=30z_0$ (**Jiménez 2004**). The effective roughness height is supposed to be equal to $K_s = 2d_{50}$ (**Van Rijn, 1984**).

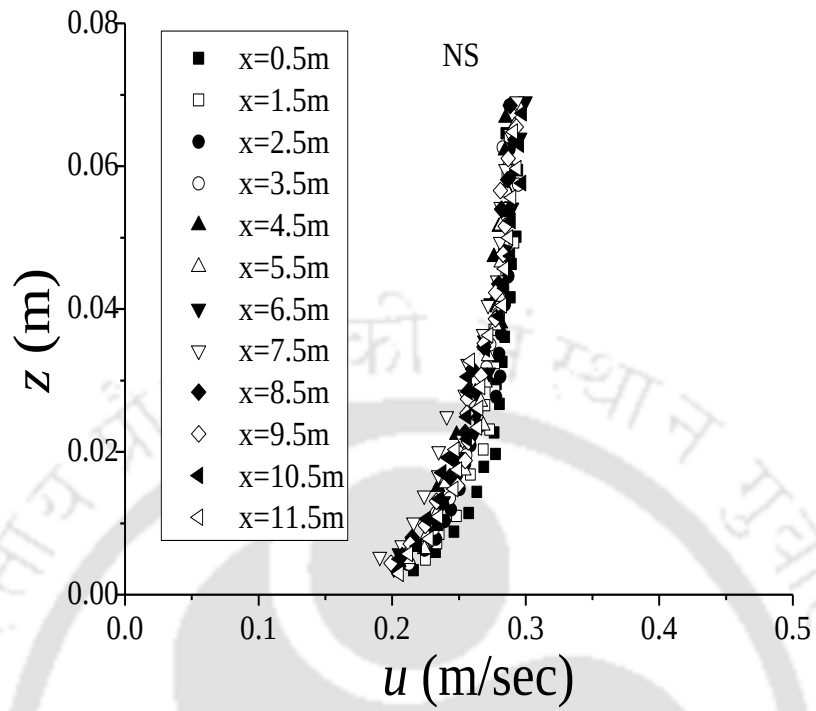


Figure 2.19: Time average velocity profiles at each measurement position for NS

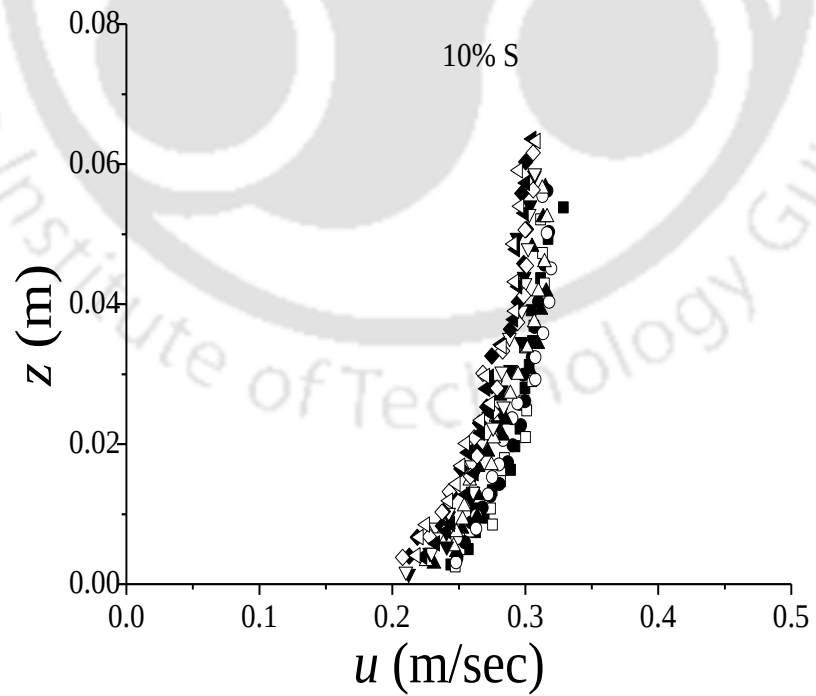


Figure 2.20: Time average velocity profiles at each measurement position for 10% S

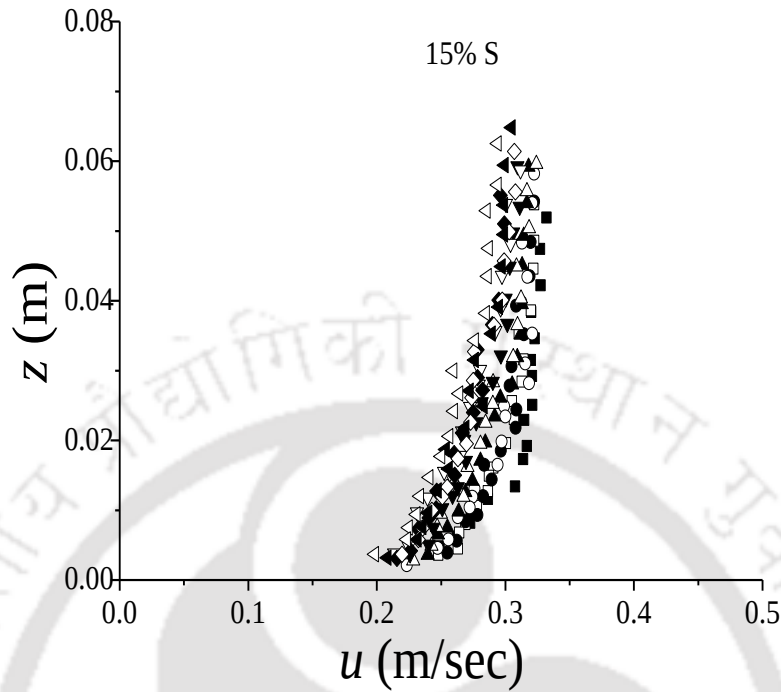


Figure 2.21: Time average velocity profiles at each measurement position for 15% S

2.15.2 Length of Flow Developing Zone

Figure 2.22-2.24 shows the measured velocity profiles with respect to flow depth and channel length for no seepage and seepage flow and gives the boundary layer thickness along the centreline of the developing flow. Close examination of the velocity profiles in Figure 2.22-2.24, reveals that the vertical distribution of the velocities remains almost unchanged and therefore the flow can be assumed as fully developed flow when $x \geq 6.5$ m for no seepage and $x \geq 7.5$ m for 10% and 15% seepage flows. That is the length of the flow developing zone is $L_f = 6.5$ m for no seepage run and $L_f = 7.5$ m for seepage runs. The measured velocity profiles of the no seepage flow shows that the length of the boundary layer development become equal to $56h$ where h is the flow depth. The result is similar to the finding of previous researchers (**Ardiclioglu and Ozturk, 2006; Kirgoz and Ardiclioglu, 1997**). However with the application of seepage, development length is increased, which is equal to $65h$. The result depicts that development length is increased to 16% with the application of seepage as compared to no seepage flow. The reason behind this observation is that in presence of downward seepage, the stream wise velocity

near the bed increases resulting in the formation of a more uniform velocity distribution. Figure 2.22-2.24 shows that the boundary layer thickness δ gradually increases and at the end of the developing zone, δ becomes approximately constant. In Figure 2.25, the dimensionless length of the flow developing zone L_f/h is plotted against the ratio Re/F , based on the flow parameters given in Table 1. In Figure 2.25, a simple expression for boundary development length was given which was determined based on no seepage and seepage flow data. From the experimental data in Figure 2.25, the correlation between the dimensionless length of the developing zone with Reynolds number and Froude number may be approximated by the following simple expression:

$$\frac{L_f}{h} = 271.86 - 0.0005 \frac{Re}{F} \quad (2.18)$$

Equation (2.18) is suggested for predicting the length of the flow developing zone of open channel flow within the present experimental conditions.

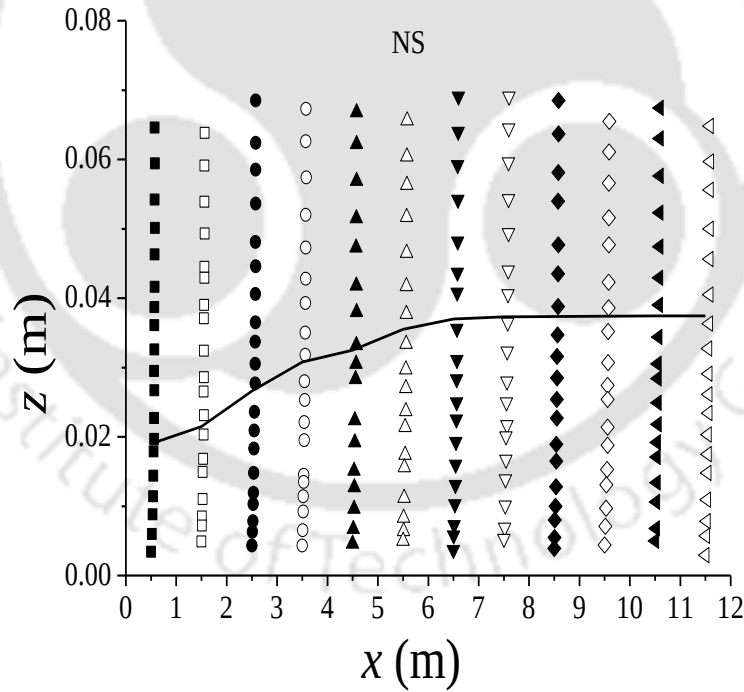


Figure 2.22: Variation of boundary layer thickness along flow axis for no seepage conditions

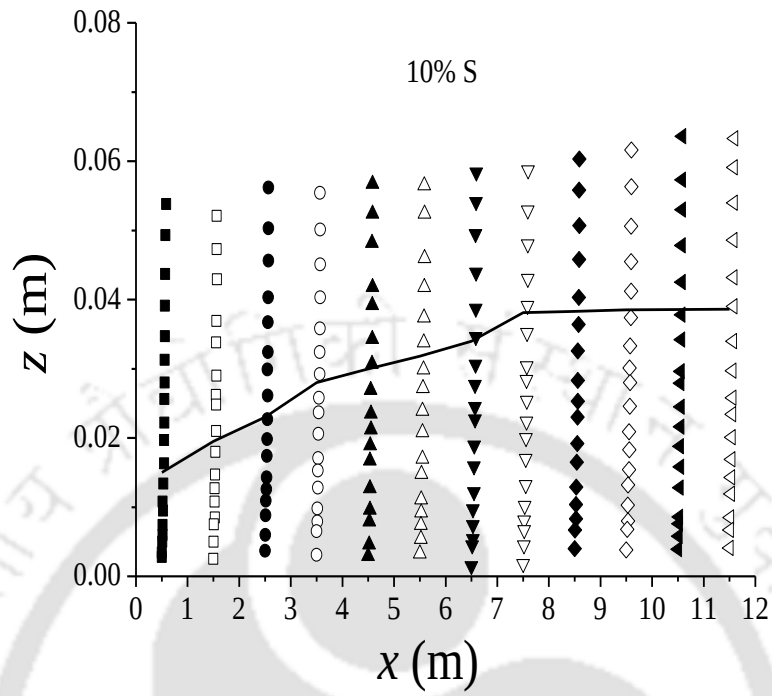


Figure 2.23: Variation of boundary layer thickness along flow axis for 10% seepage conditions

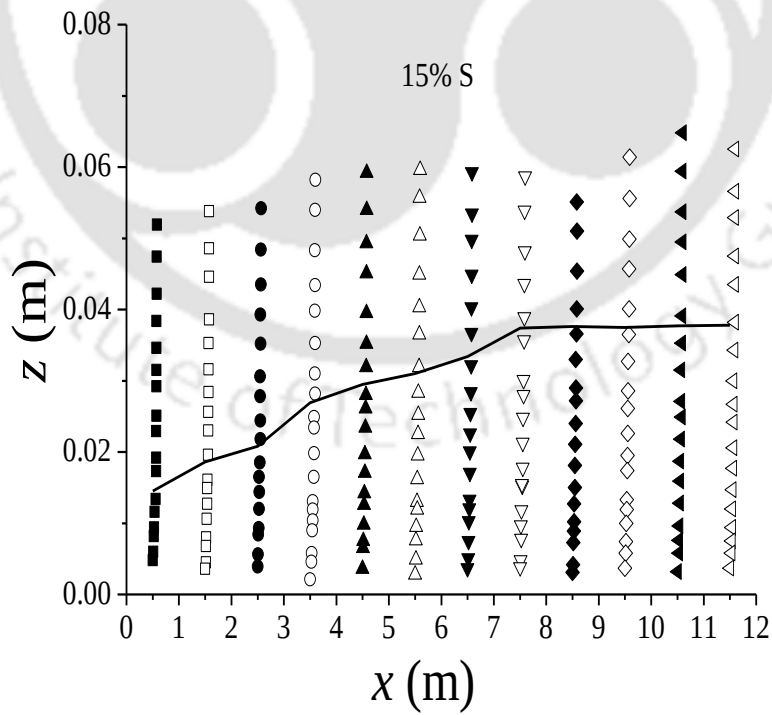


Figure 2.24: Variation of boundary layer thickness along flow axis for 15% seepage conditions

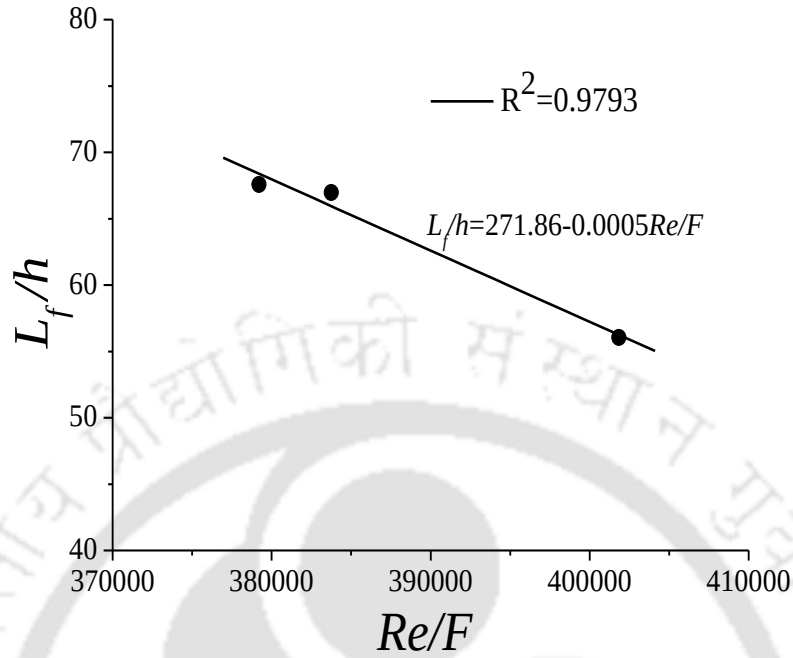


Figure 2.25: Variation of dimensionless length of flow developing zone with Re/F

2.15.3 Vertical Development of Boundary Layer

The boundary-layer height, δ , has been determined as the height where time average velocity reaches 99% of the free stream velocity magnitude. The boundary-layer height based on this definition and normalised by roughness height (z_0) is shown in Figure 2.26. Figure 2.26 pointed out that boundary layer height based on the no seepage mean velocity is consistently higher than the height based on the seepage velocity profile. Overall, for all the cases subjected to no seepage and seepage flow, decrement of boundary layer thickness occurs in the following sequence: Plane bed without seepage (NS) $\rightarrow$ Plane bed with seepage (10%S) $\rightarrow$ Plane bed with seepage (15%S). As the velocity is increased and achieved uniform velocity distribution with the application of seepage, the boundary layer thickness corresponding to plane bed without seepage is greater as compared to plane bed with seepage. From the experimental data in Figure 2.26, the correlation between the dimensionless boundary layer thicknesses with channel longitudinal section may be approximated by the following simple expression:

$$\left. \begin{aligned} \frac{\delta}{z_0} &= 0.0385 \left(\frac{x}{z_0} \right)^{0.8} + 481.48 \text{ for NS} \\ \frac{\delta}{z_0} &= 0.0396 \left(\frac{x}{z_0} \right)^{0.8} + 364.21 \text{ for S} \end{aligned} \right\} \quad (2.19)$$

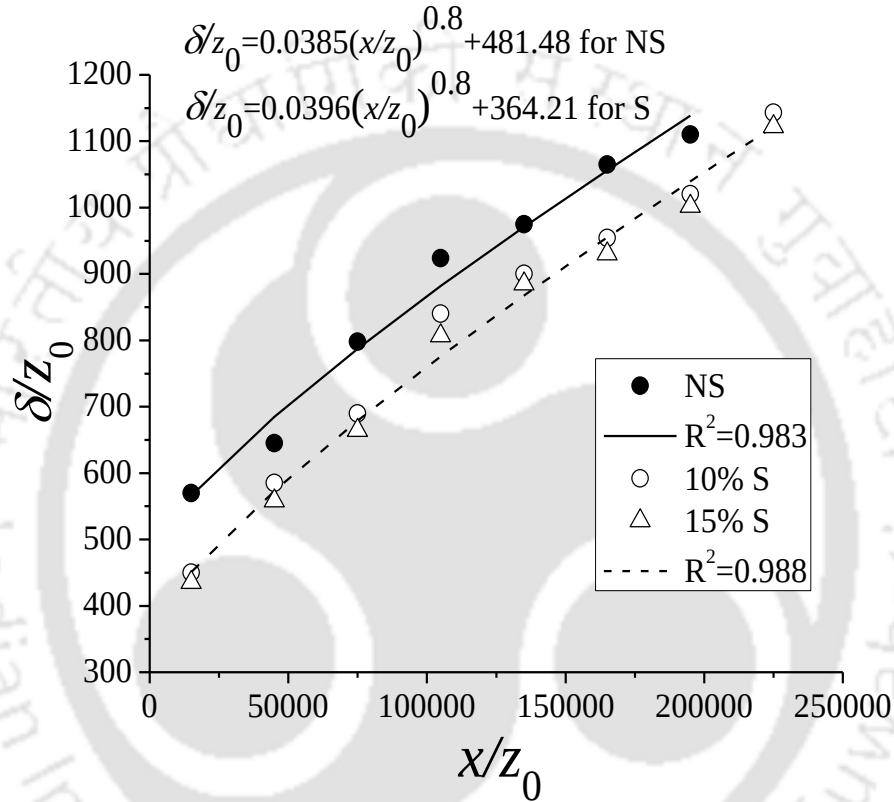


Figure 2.26: Variation of Boundary-layer Thickness (δ) normalized by the roughness height (z_0) subjected to no seepage and seepage flow.

Equation (2.19) is suggested for predicting the boundary layer thickness of no seepage and seepage flow within the present experimental conditions. As expected, this law differs from the classic smooth surface turbulent boundary-layer growth rate that predicts a power law exponent of 0.85 (**White 1991**). Our power-law exponent is equal to the well-known 4/5th power law, however, starting with **Elliot's model (1958)**, to predict the growth of an internal boundary layer developing after a sudden change of roughness. Many experimental studies of the increasing internal boundary layer (**Antonia and Luxton, 1971; Pendergrass and Arya, 1984**) as well as

numerical studies (**Lin et al., 1997**) have yielded exponents near to 0.8. **Townsend's (1966)** theoretical analysis shows that, for a large change in friction velocity, from a very smooth to a very rough surface, the flow behaves essentially as that of a boundary layer developing below a uniform free-stream velocity. However, the coefficient and depth of the boundary layer measured is less than those predicted by **Elliot's theory (1958)**, which is valid for small changes in friction velocity.

2.16. Conclusions

The main goal of this chapter is to provide the details of experimental setup and methodology and also to investigate boundary layer development using the measurements of velocity profiles in non-uniform sand bed channel subjected to no seepage and seepage flows. The experiment presented herein reproduces the development of a boundary layer in a water flume over a non-uniform sand bed channel at high Reynolds numbers. The velocity measurements were performed via a three-dimensional acoustic Doppler velocimeter (ADV) technique in the vertical symmetry plane of the flow. Measured velocity profiles are used to determine the boundary layer thickness and development length. For no seepage and seepage flow conditions, the maximum development length was obtained at $x=6.5$ m and $x=7.5$ m respectively. Therefore, it can be accepted that the flow was fully developed beyond that point. Here, no seepage flow gives higher thickness of boundary layer and smaller boundary layer development length values as compared to seepage flows. The boundary layer development varies between 56 to 65 times of the water depth for all flow conditions. The boundary-layer growth was established on the basis of the mean longitudinal velocities yielding 0.8 power law in agreement with previous results for the growth of an internal boundary layer. However, it appears that coefficient is smaller than that for a typical internal boundary layer. The present chapter introduces empirical equations in order to develop boundary layer thickness and flow development length in seepage affected alluvial channel under present experimental conditions.

3 Structure of Turbulence over Non-uniform Sand Bed

Channel with Downward Seepage

3.1 Overview

Fluvial processes in the natural environment are comprised with aggradation or degradation of the channel bed and banks. Sediment movement can be characterized by the resistance offered by the sediment particles due to specific weight of particles. In this regard, for each sediment size, there is specific bed shear stress to start the movement of bed particle that is called the threshold (critical) shear stress. In alluvial channels, bed feature develop whenever the bed shear stress caused by flow exceeds from its critical value. In this regard, turbulent flow plays a major role in fluvial processes and bed features phenomenon. Earlier studies suggested that the bed features development is a function of the coherent turbulent flow structures in sedimentary environments, where sweeps (bursting events) govern the initiation of bed features in the near-bed region (**Williams and Kemp, 1971; Raudkiv and Witte, 1990; Best, 1992; Nelson *et al.*, 1995; Raudkivi, 1997**). In particular, development of hairpin vortices near the channel bed causes the generation of multiple bursting events such as ejections and sweeps, moreover, impact of micro turbulent sweeps ascribe to create flow parallel ridge or defects over the rough boundary (**Best, 1992**). **Venditti *et al.* (2005)** observed that the initiation of bed defects and their length are linked to integral scales of flow.

Although many studies (Section 1.4) have investigated flow hydrodynamics or turbulent characteristics with seepage in case of uniform sand, but the flow characteristics of flat sand bed composed of non-uniform sediment with downward seepage remain unexplored. Since riverbeds are usually composed of non-uniform sediment mixture, the present study therefore focuses on the influence of downward seepage flow on the turbulent flow characteristics over a mobile rough boundary (made of non-uniform sand of size 0.5mm). In this chapter, a brief introduction of turbulent flow characteristics and its importance in fluvial system are discussed. Acoustic

Doppler Velocimeter measurements were taken at a location where flow and turbulence quantities were uniform over a horizontal range. The experimental result will deliver important information related to the turbulence characteristics, such as velocity, Reynolds shear stress, anisotropy, correlation coefficient, turbulence diffusivity, mixing length, von Kármán constant, third order correlations, fourth order correlations, turbulent energy budget and conditional Reynolds shear stresses. The present chapter experimentally investigates probability distribution function (PDFs) of turbulent parameters such as velocity, Reynolds shear stress, bursting events for both no seepage and seepage runs. Also, the present chapter develops the theoretical expression for the PDFs of turbulent intensity which is not available in the existing literature.

3.2 Turbulent Fluvial Flows

State of fluid flows in natural environment is characterized in two categories i.e. laminar flow and turbulent flow. Laminar state is rarely observed in environment, occurs in low flow velocity that is envisaged as layered flow in which fluid layers smoothly slide over each other, thus maintaining a differential flow velocity in between the layers. As most of the flow in natural channels is always turbulent in nature in which turbulence in the fluid flow is defined by irregular, diffusive, dissipative, and chaotic motion of fluid particles. Velocities of fluid particles vary rapidly in space and time. Further, turbulence in most of the geophysical flows is exceedingly complex because of intrinsic instability in such flows. Instability in such flows causes formation of eddies at different scales depending upon boundary condition. These eddies ‘cascade’, splitting into smaller and smaller eddies until their energy is dissipated through viscosity (**Tennekes and Lumley, 1972**). Indeed, turbulence is intricate in nature, which cannot be defined straightforwardly. However, one can show its characteristic structures from the measurement of instantaneous velocity data. The most important characteristic of turbulence is its randomness and intermittency, which create a deterministic approach potentially difficult. Thus, one has to describe the turbulence through statistical analysis of temporal velocity data. Important parameters of turbulent flow characteristics are included such as first order analysis is magnitude of mean of velocity fluctuations, second order is the variance of a sampled instantaneous velocity series, third order analysis is higher-order moments such as skewness and kurtosis (which describes the temporal distribution of this variance), cross-correlations of velocity fluctuations which describe fluid momentum transfer, and parameters (integral scales of

flow) describing the scales or sizes of eddies present. In this regard, statistical analyses of instantaneous velocities are used to describe turbulent flows in different scenarios.

3.2.1 Velocity Decomposition

Turbulence has been evaluated traditionally by decomposing the instantaneous velocity samples (U_i, W_i) into the time-averaged velocity (u, w) and fluctuating components of velocity (u'_i, w'_i) (Figure 3.1). This process is known as Reynolds decomposition (**Tennekes and Lumley, 1972**) and it requires a steady state or quasi-steady flow.

$$\left. \begin{aligned} U_i &= u + u'_i \\ W_i &= w + w'_i \end{aligned} \right\} \quad (3.1)$$

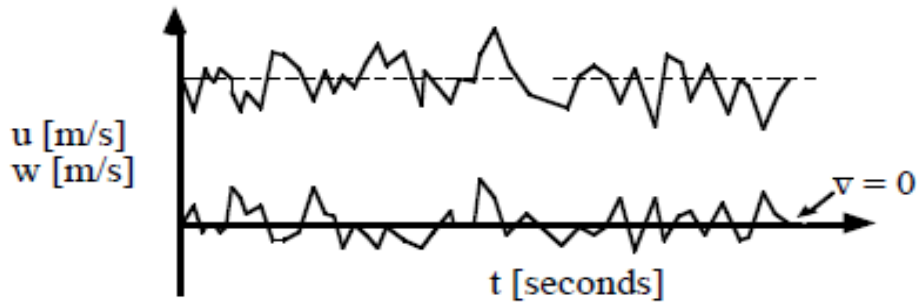


Figure 3.1 Velocity recorded signal at a point

Time averaged streamwise (u) and vertical (w) velocities have been evaluated as:

$$\left. \begin{aligned} u &= \frac{1}{n} \sum_{i=1}^n U_i \\ w &= \frac{1}{n} \sum_{i=1}^n W_i \end{aligned} \right\} \quad (3.2)$$

Where n is the number of samples. Averaging fundamentally results in a loss of information. Indeed, it is the purpose of averaging to remove ‘non-essential’ details, while leaving only the core information. With respect to this, **Reynolds (1895)** proposal should be recalled that it would be impossible to understand turbulence in detail, and it would thus be more useful to consider

only statistical properties of the turbulent flow. Figure 3.1 depicts the loss of information due to averaging. Time averaged velocity can be similar for different fluctuating velocities and also it cannot define the scale of fluctuations in the flows. Therefore, standard deviation of the set of ‘random’ or fluctuating velocities can be used to define the level or strength of turbulence in flow. Turbulence strength is the root mean square (RMS) velocity in streamwise (vertical) direction u_{rms} (w_{rms}) and it can be evaluated as:

$$\left. \begin{aligned} u_{rms} &= \sqrt{\frac{1}{n} \sum_{i=1}^n (U_i - u)^2} \\ w_{rms} &= \sqrt{\frac{1}{n} \sum_{i=1}^n (W_i - w)^2} \end{aligned} \right\} \quad (3.3)$$

For flows over irregular boundaries, such as that of mobile or deformable sand bed, the time-averaged velocity is dependent upon the spatial and temporal position over the bed. Turbulent fluctuations at a point are given by the deviations from the local time averaged velocity:

$$U_i(x, y, z, t) = u(x, y, z, t) + u'_i(x, y, z, t) \quad (3.4)$$

Variations in streamwise velocities near the channel bed lead to the formation of a shear mixing layer. In this regard, fluid layers or molecules move in very irregular paths in turbulent flow, causing an exchange of momentum from one portion of fluid to the other, and hence, the turbulent shear stresses (τ_{turb}) are generated also known as the Reynolds shear stresses (τ_{uw}). The Reynolds shear stresses can be given by time-averaging of the Navier- Stokes equation.

$$\left. \begin{aligned} \tau_{turb} &= \tau_{uw} = -\rho_w \overline{u'w'} \\ u'w' &= \frac{1}{n} \sum_{i=1}^n (U_i - u)(W_i - w) \end{aligned} \right\} \quad (3.5)$$

Where, ρ_w is the density of water. In turbulent flow both viscosity and turbulence contribute to shear stress. The total shear stress (τ_{total}):

$$\tau_{total} = \tau_{visc} + \tau_{turb} = \rho_w \nu \frac{du}{dz} + (-\rho_w \overline{u'w'}) \quad (3.6)$$

Where, τ_{visc} is the shear stress due to viscosity and ν is the kinematic viscosity.

3.2.2 Turbulence in 3-Dimensional Flow Field

The level of turbulence in a sampled velocity is traditionally been described by the average magnitude of velocity fluctuations, commonly the RMS $\sqrt{u_i'^2}$, or standard deviation of the velocity sample (Equation 3.3). This is commonly expressed as the non-dimensional turbulence intensity ($\hat{\sigma}_u$) of streamwise velocity components, and the turbulent kinetic energy (TKE). Semi-empirical universal functions for ($\hat{\sigma}_u$) and non-dimensional TKE in open-channel flow are proposed by **Nezu and Nakagawa (1993)**, based on the k- ϵ turbulence model:

$$\left. \begin{aligned} \hat{\sigma}_u &= \left(\frac{\sqrt{u'u'}}{u_*} \right) = D_u \exp^{(-C_u z^+)} \\ TKE &= \frac{1}{2} (\overline{u'^2} + \overline{v'^2} + \overline{w'^2}) \end{aligned} \right\} \quad (3.7)$$

where:

z^+ : is the non-dimensional flow depth.

D_u and C_u : are the empirical parameters set to 2.3 and 1, respectively, as suggested by **Nezu and Nakagawa (1993)**.

These values of empirical parameters are suitable for the flat and smooth bed throughout the flow depth except very near to the boundary, where viscous effects are dominant and side currents are absent. Turbulence intensities in the near-bed region are reduced by the effect of increasing roughness height. With increasing relative roughness, **Bayazit (1976)** observed significant reduction in turbulence intensities. These empirical parameters are not universal constant and are dependent on the bed roughness. **Sukhodolov et al. (1998)** have calculated new sets of constants for flows over rough bed.

The distribution of turbulence is further reflected by the higher moments of velocity fluctuations which aid in the identification of infrequent velocity fluctuations of large magnitude associated with sweep and ejection events (**Grass, 1971**). The third moment of turbulent velocity fluctuations or the coefficient of skewness $Sk(u_i)$ of a velocity time series describes the asymmetry in the probability density function of turbulent fluctuations. The coefficient of

skewness is an important factor to describe infrequent velocity fluctuations of large magnitude such as bursting events (**Nezu and Nakagawa, 1993**) can be defined as:

$$Sk(u_i) = \frac{\overline{u_i'^3}}{(\overline{u_i'^2})^{3/2}} \quad (3.8)$$

Grinvald and Nikora (1988) reported from investigations in flumes and rivers at moderate velocity that the skewness coefficient for velocity in the streamwise direction is usually positive near the bed, becomes negative in the intermediate region, and again positive near the water surface, with absolute values generally less than 1. Measurements of **Nikora and Smart (1997)** in a gravel-bed river similarly show negative skewness coefficient for velocity in the streamwise direction over the majority of the flow depth. These negative values of skewness coefficient reflect the transport of turbulent energy from the bottom region to the upward direction (**Nikora and Smart, 1997**).

Experiments were performed on both the sands and both the slope (Table 2.4). Results from all experiments were found to be consistent. Therefore, in the results section, in order to avoid the duplicity and congestion, turbulent characteristics presented here have been taken for the experiments on the non-uniform sand of median grain size 0.5 mm and longitudinal bed slope of 0.001. Measurements were carried out along the centerline of the channel on a section located at distance of 8 m from the downstream end.

3.3 Time Averaged Velocity

At a point, the time averaged streamwise velocity was obtained from the decomposition of instantaneous velocity samples into the time averaged velocity and fluctuating component of the velocity. Figure 3.2 shows the variations of time averaged stream wise velocity (u) with normalised vertical distance ($=z/h$) in flows subjected to no seepage and seepage for 10% and 15%. It is interesting to note in Figure 3.2 that the velocity increased with the application of downward seepage as compared to no seepage for a particular discharge. Further, quantitative analysis suggests that at very near to the bed ($z/h=0.03$), velocity increases on an average value of 2.36% with the application of 10% seepage. It is also observed that in the lower flow region

($z/h=0.03$), the velocity for 15% seepage case is slightly higher with an average increasing value of 0.40% than the velocity for 10% seepage.

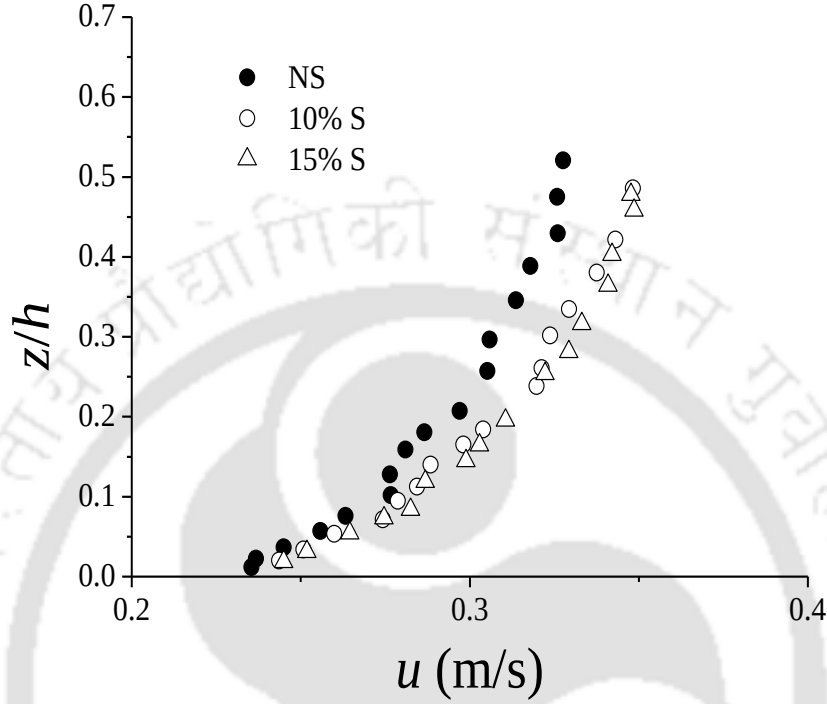


Figure 3.2: Vertical profiles of time averaged stream wise velocity

Experimental data has fitted to the **Deshpande and Kumar (2016a)** log law as expressed in following non-dimensional form.

$$\frac{u}{u_*} = \frac{1}{k} \ln(z^+ + \Delta z^+) - \frac{1}{k} \ln(\varepsilon^+) \quad (3.9)$$

Where, $z^+ = z/d_{50}$, $\Delta z^+ = \Delta z/d_{50}$, Δz is depth of virtual bed level from the bed surface, $\varepsilon^+ = z_0/d_{50}$, z_0 is zero velocity level, k is von Karman constant. Here, the log law has been fitted in the inner layer ($z/h < 0.2$) of the velocity profiles in the same manner as has been suggested by **Deshpande and Kumar (2016a)**. In seepage condition, there is strong exchange of momentum in between the flow and sand bed due to which the modified logarithmic law for no seepage condition as shown in Figure 3.3 does not hold for seepage condition. As a result, the modified logarithmic law has been proposed for both 10% and 15% seepage condition as shown in Figure 3.3.

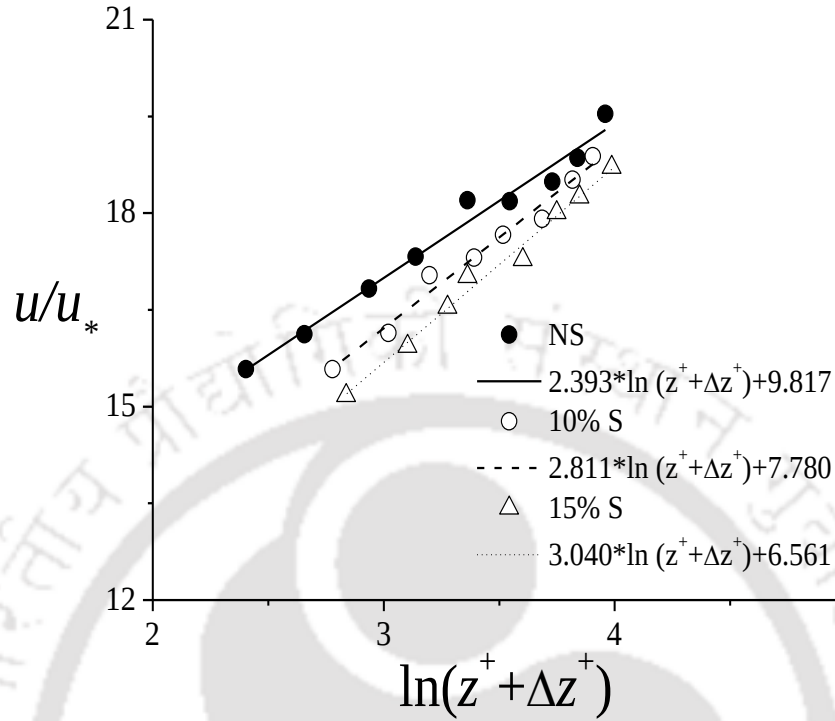


Figure 3.3: Logarithmic law for velocity profiles

Average values of the von Karman's constant, depth of the virtual bed level and the zero velocity level are tabulated in Table 3.1. Value of the k for no seepage condition has been observed to be slightly higher than the universal value (0.41). The downward seepage influenced the bed particles to move rapidly and value of k drastically decreased to 0.355 and 0.328 with 10% S and 15% S respectively. Further, from the regression equation shown in Figure 3.3, it is observed that with the application of seepage, the depth of virtual bed level and the zero velocity level from the bed surface increased suggesting an exposure of increased velocity component in the stream wise direction to the particles on the bed surface.

Table 3.1: Coefficient value observed from log law equation

Flow condition	u_* (mm/s)	k	Δz (mm)	z_0 (mm)
NS	15.25	0.418	3.03	8.258×10^{-3}
10% S	16.14	0.355	4.25	31.58×10^{-3}
15% S	16.60	0.328	5.04	57.78×10^{-3}

3.4 Reynolds Stresses

Reynolds stresses provide crucial information on the characteristics of flows with and without seepage. Reynolds stresses are the components of a symmetric second order tensor where the diagonal components are called the Reynolds normal stresses (RNS) and the off diagonal components are called the Reynolds shear stresses (RSS) (**Pope, 2000**).

3.4.1 Reynolds Shear Stresses (RSS)

RSS defines the real time momentum flux of the streamwise velocity in the vertical direction caused by the fluctuating velocity field. Figures 3.4 show the vertical variations of Reynolds shear stress (RSS) in flows subjected to no seepage and seepage runs where the vertical ordinate (z) has been made non-dimensional with the flow depth (h). The shear velocity $u_* [= (\tau_0 / \rho_w)^{0.5}]$ where τ =boundary shear stress that are used to normalise the flow and turbulence quantities has been evaluated by the linear projection of the RSS profile on the channel bed which is $\tau_0 = (\tau_{uw})_{z=0}$ as given by **Nezu (1977)**. It has been assumed here that the shear stress caused by viscosity is negligible compared to the shear stress due to RSS (**Nezu, 1977**). Figure 3.5-3.7 shows that the variation of normalised RSS $\tau_{uw}^+ = \tau_{uw} / u_*^2$ for both no seepage and seepage runs is reasonably consistent with the traditional linear law of RSS for free-surface flows with zero pressure gradients. It can be seen from Figure 3.4 that RSS increases towards the channel bed are associated with the provided momentum from the main flow to maintain sediment particle motion overcoming the bed resistance and attain a maximum value in between $0.05 < z/h < 0.2$ and then again decrease towards the boundary because of the presence of a roughness sub layer in the near bed region. The distance of occurrence of maximum RSS from the channel bed increased under the action of downward seepage indicating an increase in the thickness of sublayer. In the present experiments, the profiles of Reynolds shear stress distribution were found to be similar for both no seepage and seepage runs but the magnitude is higher in both inner and outer layer with the application of seepage. The maximum value of τ_{uw} is about 13.80 % greater with 10%S and about 14.30 % greater with 15%S, as compared to that with no seepage. This increase in the RSS caused the subsequent increase in the shear velocities by 5.83% and 8.85% in case of 10%S and 15%S respectively as compared to that with no seepage.

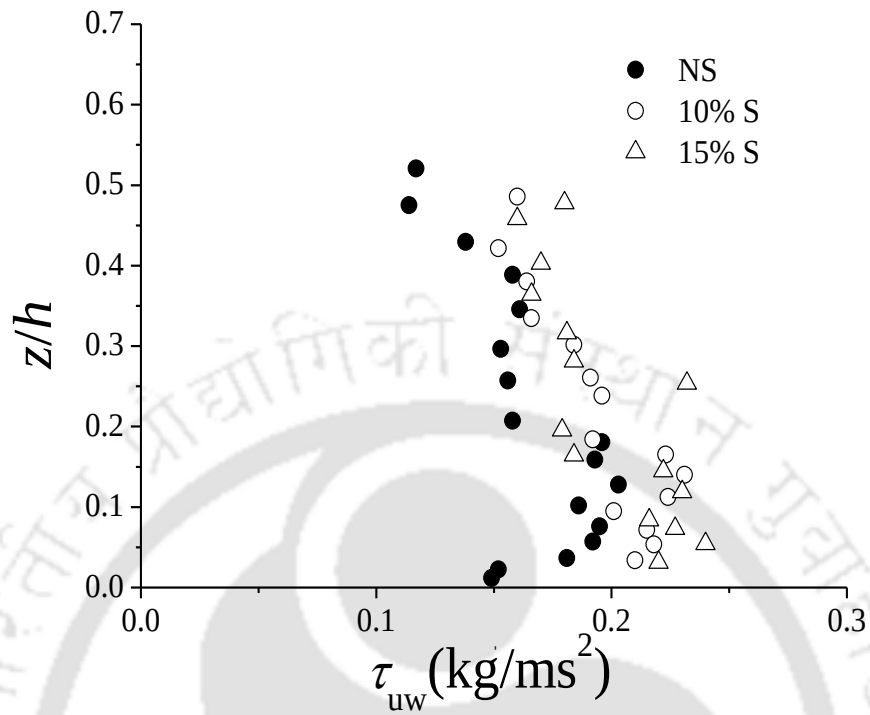


Figure 3.4: Vertical profiles of Reynolds shear stresses

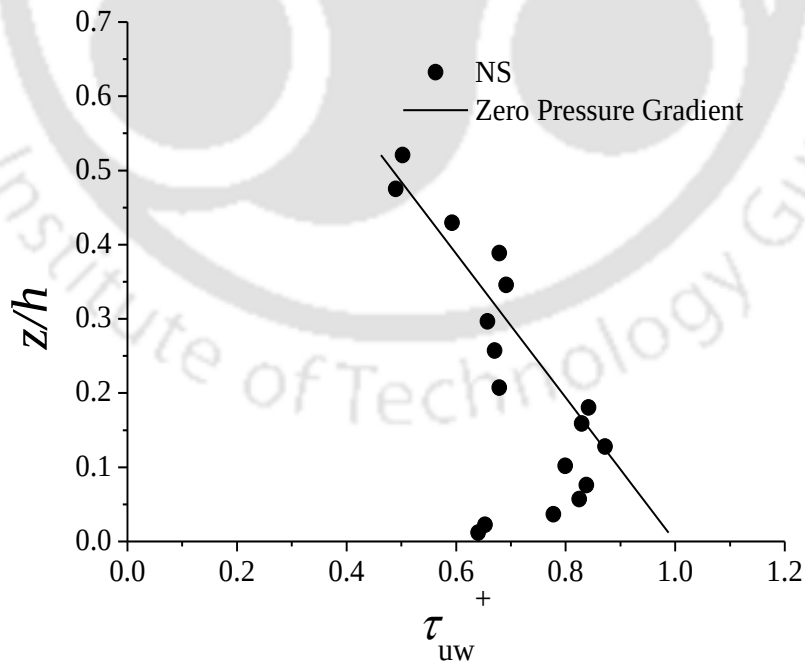


Figure 3.5: Vertical profiles of non-dimensional RSS for no seepage run

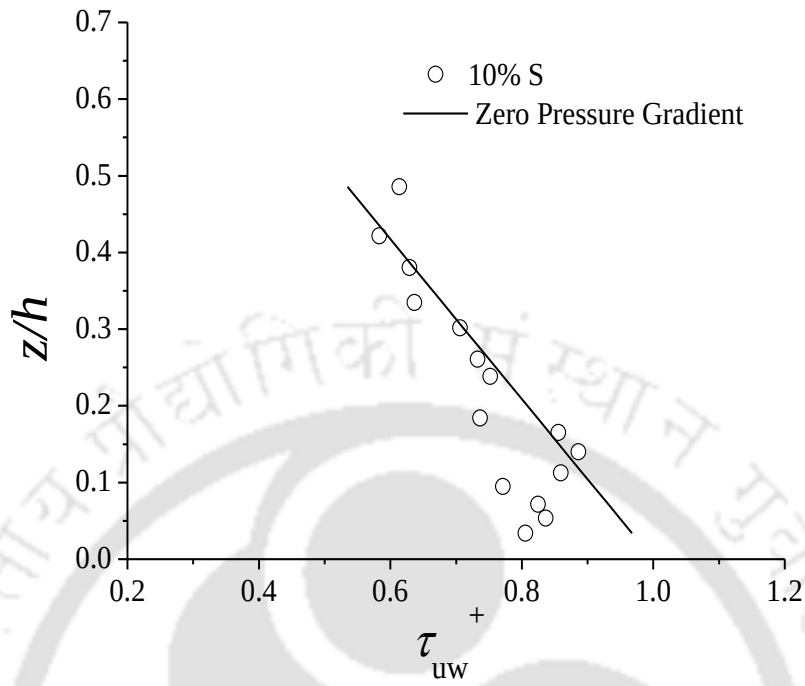


Figure 3.6: Vertical profiles of non-dimensional RSS for 10% seepage run

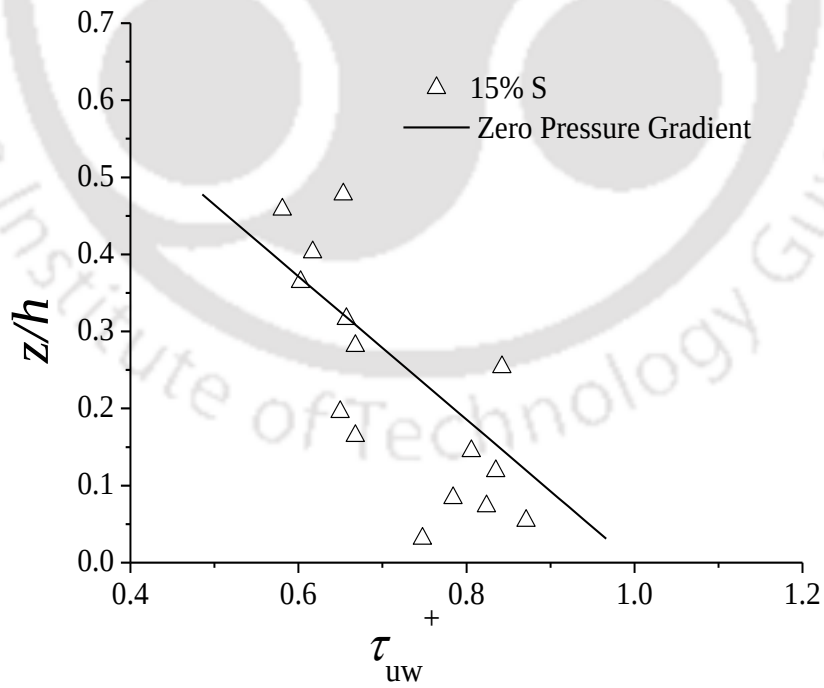


Figure 3.7: Vertical profiles of non-dimensional RSS for 15% seepage run

3.4.2 Reynolds Normal Stresses (RNS)

Vertical distributions of Reynolds normal stresses are also called the turbulence intensities. Strength of turbulence in the flow is defined by the turbulent intensity, which can be calculated by the square root of the second moment of velocity. The vertical profiles of stream wise turbulence intensity ($\sigma_u = \sqrt{u'u'}$) and vertical turbulence intensity ($\sigma_w = \sqrt{w'w'}$) are shown in Figure 3.8 and Figure 3.9 respectively. In general, the profiles of σ_u and σ_w with downward seepage are greater than the profiles of σ_u and σ_w with no seepage. This observation is in conformity with previous researchers (Nezu, 1977; Antonia *et al.*, 1988, 1995; Krogstad and Kourakine, 2000; Kim and Sung, 2003; Deshpande and Kumar, 2016a). Vertical profiles of turbulence intensities are evident within the near bed flow zone. As compared to no seepage, the degree of damping increases by an average value of 10% for σ_u and 9.8% for σ_w with 10%S and increases with 15%S in average by 15% for σ_u and 10.3% for σ_w . It is found that the strength of turbulence increases under the action of downward seepage.

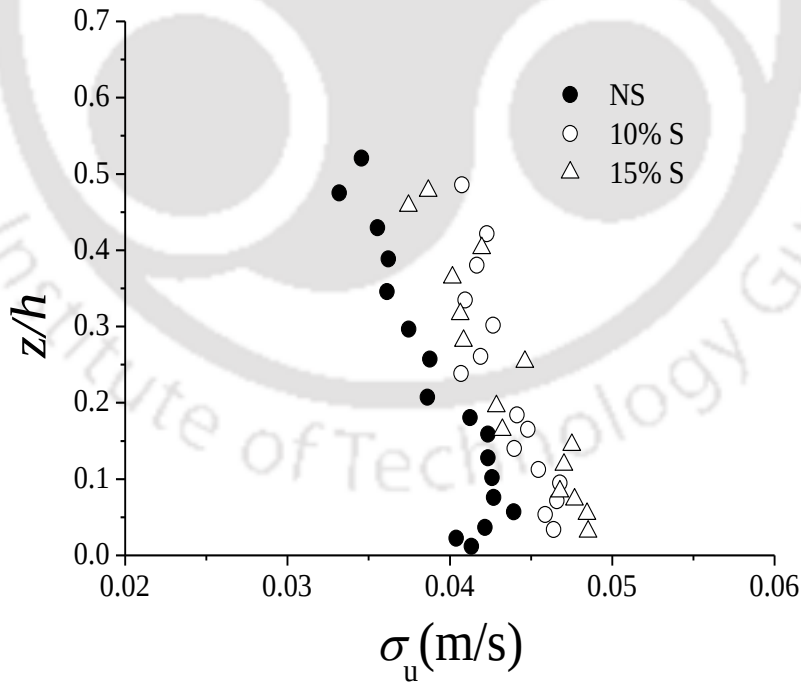


Figure 3.8: Vertical profiles of streamwise turbulence intensities

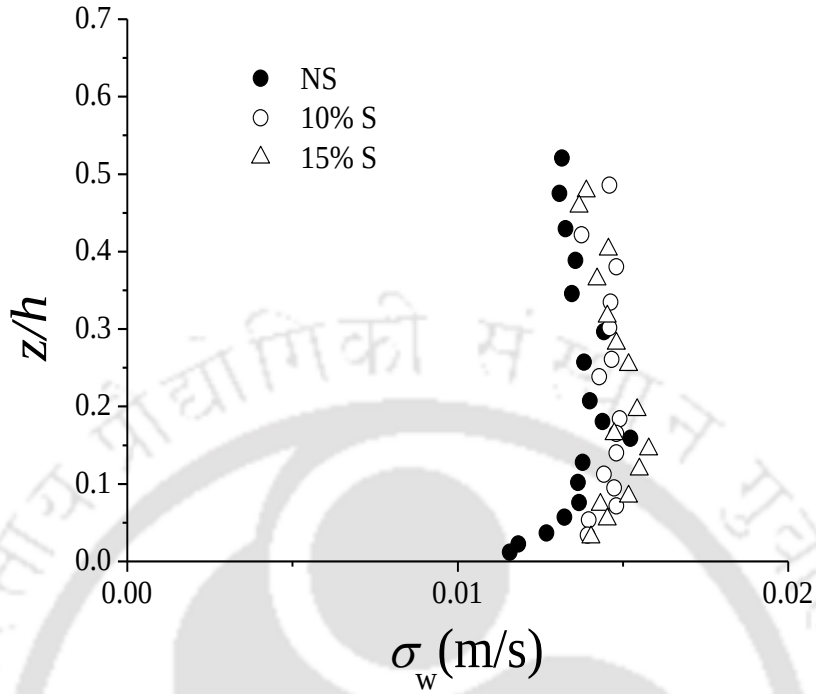


Figure 3.9: Vertical profiles of vertical turbulence intensities

Stream wise turbulence intensities (σ_u) and vertical turbulence intensities (σ_w) have been made non-dimensional with u_* in a way that $\hat{\sigma}_u = \sigma_u/u_*$ and $\hat{\sigma}_w = \sigma_w/u_*$. Several laws have been suggested to describe the vertical distribution of turbulence intensities in uniform open channel flows (**Nezu and Nakagawa, 1993; Nikora and Goring, 1998**). Since these laws provided by them are not being based on the non-uniform sediment mixture due to which time averaged turbulent intensities in stream wise direction for no seepage and seepage runs shown in Figure 3.10-3.12 is not fitted to the universal laws provided by **Nezu and Nakagawa (1993)** and **Nikora and Goring (1998)**. Regression analysis has been used to derive new empirical constants in Equation (3.10) for flows over non-uniform beds with no seepage and seepage affected channel. The value of empirical constants is tabulated in Table 3.2.

$$\hat{\sigma}_u = D_u \exp^{(-C_u z/h)} \quad (3.10)$$

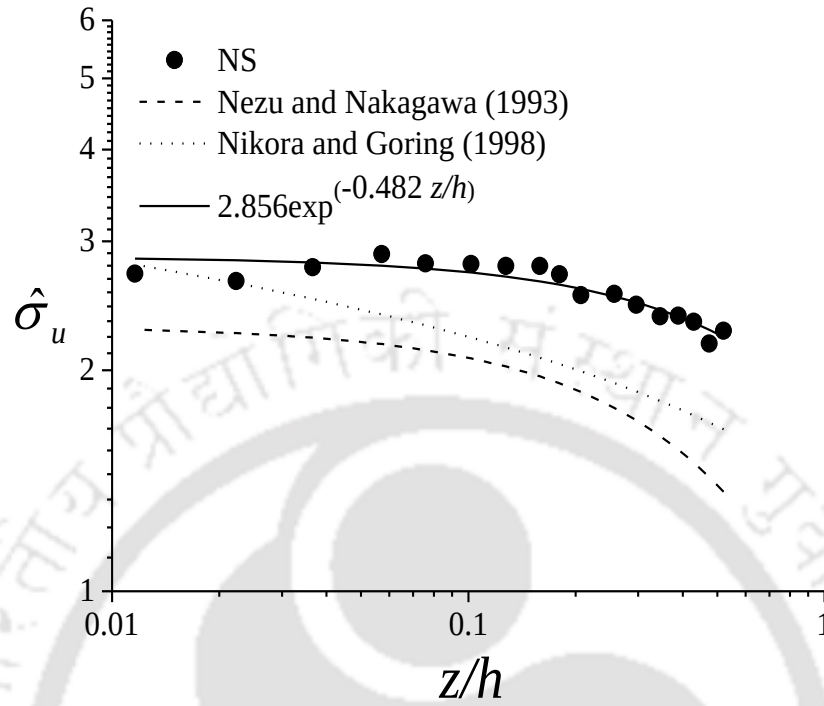


Figure 3.10: Comparison of streamwise turbulent intensities with existing literatures for NS

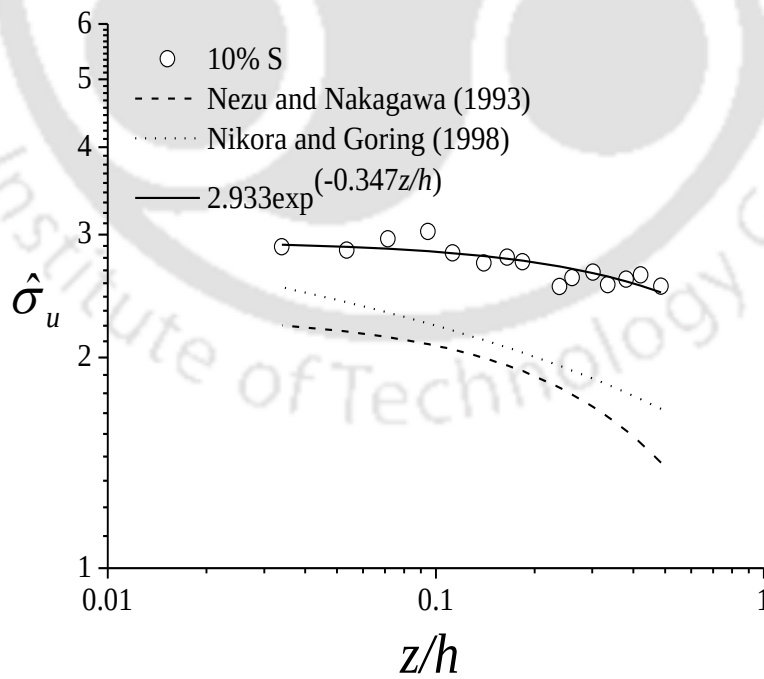


Figure 3.11: Comparison of streamwise turbulent intensities with existing literatures for 10% S

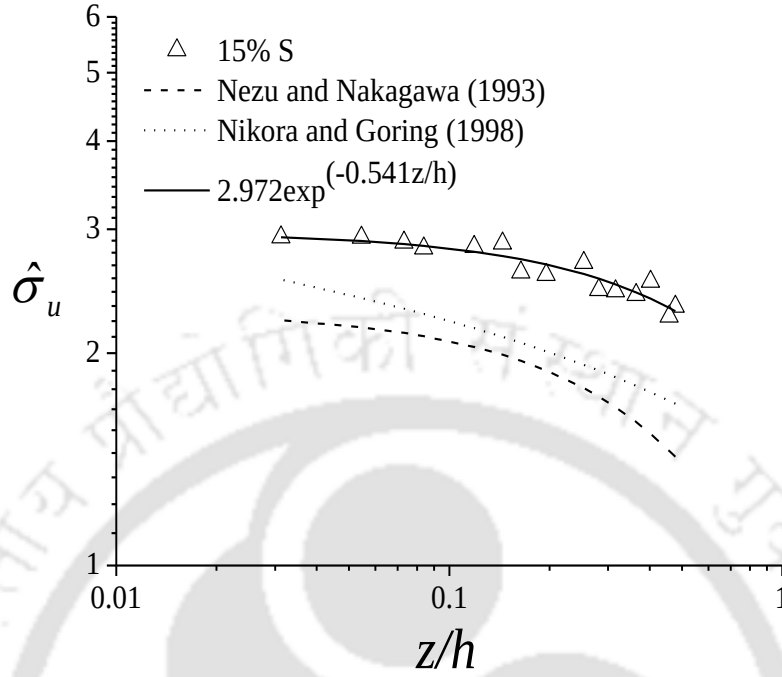


Figure 3.12: Comparison of streamwise turbulent intensities with existing literatures for 15% S

Table 3.2: Values of empirical constants for streamwise turbulent intensity

Flow Condition	D_u	C_u
NS	2.856	-0.482
10%S	2.933	-0.347
15%S	2.972	-0.541

The degree of flow anisotropy is measured by the ratio $\hat{\sigma}_w/\hat{\sigma}_u$. Figure 3.13 shows the vertical distribution of $\hat{\sigma}_w/\hat{\sigma}_u$ plotted against z/h , from which it is observed that the flow is highly anisotropic as $\hat{\sigma}_w/\hat{\sigma}_u < 1$. The $\hat{\sigma}_w/\hat{\sigma}_u$ ratio is approximately 0.28 near the boundary irrespective of the flow condition, i.e. for both no seepage and seepage runs, varying almost linearly with flow depth. In the previous studies, **Nezu and Nakagawa (1993)** predicted that the ratio is universal and has a constant value of 0.55 for smooth boundary throughout the flow depth while **Dey and Raikar (2007)** found the value of 0.6 for mobile gravel boundaries. On the contrary, the value of $\hat{\sigma}_w/\hat{\sigma}_u$ in the present result remain less than those observed by **Nezu and**

Nakagawa (1993) and **Dey and Raikar (2007)**. However, the influence of seepage on the profiles of $\hat{\sigma}_w/\hat{\sigma}_u$ is not apparent.

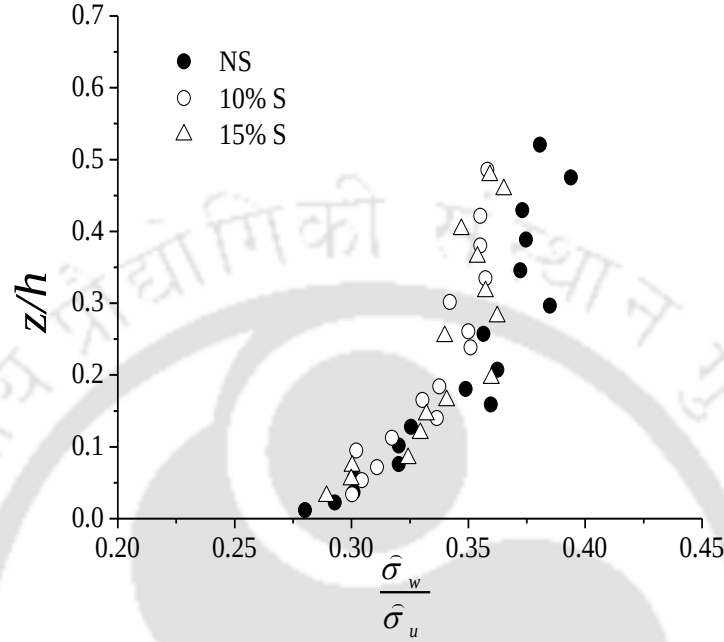


Figure 3.13: Vertical profiles of flow anisotropy

3.5 Correlation Coefficient

The correlation coefficient $r_{uw} \left[= -\overline{u'w'} / (\overline{u'u'} \times \overline{w'w'})^{0.5} \right]$ is the ratio of Reynolds shear stress to the product of turbulence intensities in stream wise and vertical direction. Figure 3.14 depicts the correlation coefficient as a function of flow depth for both no seepage and seepage condition. In the flow subjected to no seepage condition, the correlation coefficient decreases with the increase of z/h varying from $r_{uw} \sim 0.34$ near the boundary to $r_{uw} \sim 0.25$ at $z/h = 0.52$. These values are smaller than those for flows over smooth and rough boundaries with no seepage. **Hinze (1975)** and **Schlichting (1979)** found $r_{uw} = 0.4-0.5$ for smooth boundary over most of the extent of the boundary layer while **Dey and Raikar (2007)** observed that the value of r_{uw} is almost constant at $r_{uw} = 0.43$ for $z/h < 0.6$ for weekly mobile gravel boundaries but it decreases towards the free-surface. The influence of seepage on r_{uw} is prominent as r_{uw} with downward seepage is decreased

as compared to that with no seepage. Change of r_{uw} with downward seepage is due to relative modification of $-\overline{u'w'}$ over $(\overline{u'u'} \times \overline{w'w'})^{0.5}$ in the presence of downward seepage.

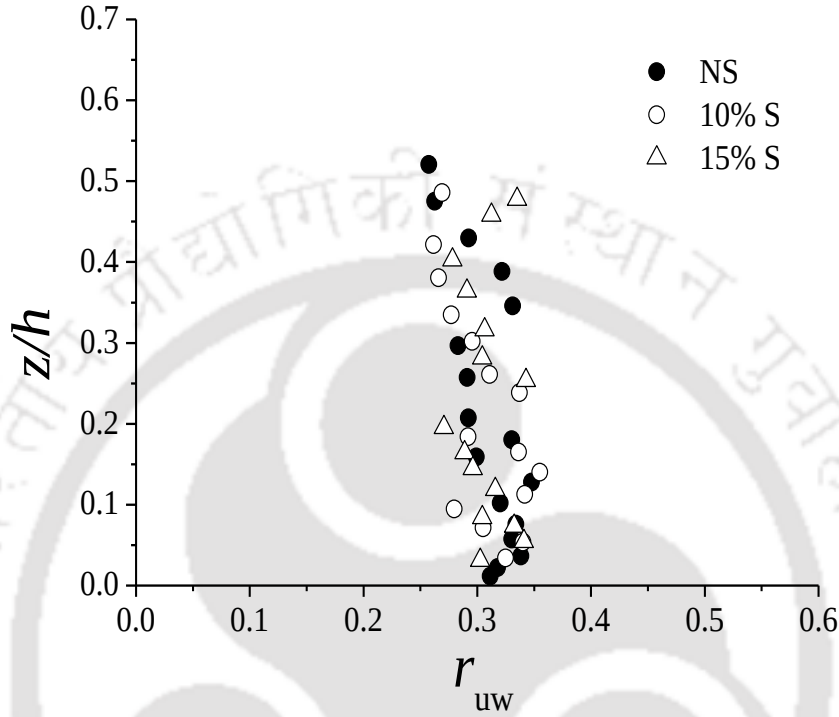


Figure 3.14 Vertical profiles of correlation coefficient

3.6 Turbulence Diffusivity

The dependency of turbulence diffusivity or the eddy viscosity $\varepsilon_f \left[= -\overline{u'w'} / \left(\frac{du}{dz} \right) \right]$ with the flow depth subjected to no seepage and seepage condition is plotted in Figure 3.15. In flows with no seepage and seepage runs, the turbulence diffusivity increases with flow depth in response to the turbulent generation achieving peak at $z/h \sim 0.3$ then it decrease with the flow depth. At $z/h \leq 0.3$, increase in ε_f is due to apparent decrease in velocity gradient and decrease in ε_f at $z/h > 0.3$ may be due to possibility of larger eddies. Within the near bed flow layer ($z/h \leq 0.1$), ε_f significantly decreases in the presence of downward seepage and further decreases with the increase of downward seepage. This decrement of turbulence diffusivity is the effect of the increase of the velocity gradient du/dz in flows induced by the downward seepage.

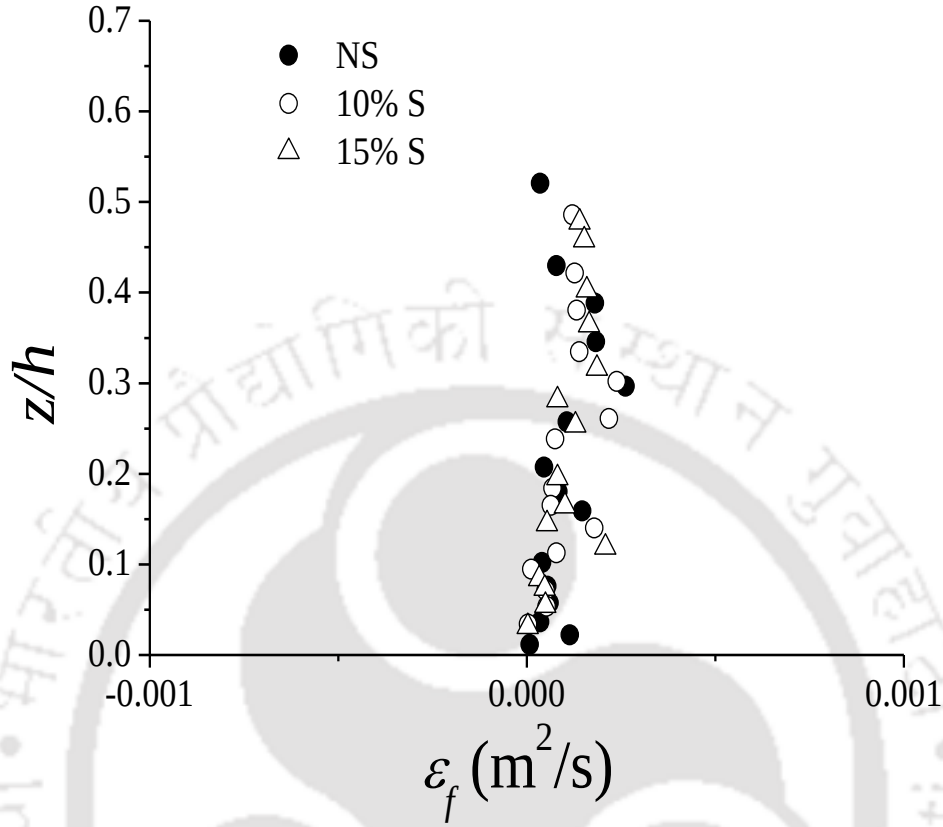


Figure 3.15 Vertical profiles of turbulence diffusivity

3.7 Turbulent Mixing Length

Another criterion to examine the nature of turbulence in flows over seepage affected alluvial channel is to look into the spatial distributions of the Prandtl mixing length l defined by (Krogstad and Antonia, 1999).

$$l^+ = (\tau_{uw}^+)^{0.5} / (\partial u^+ / \partial z^+) \quad (3.11)$$

Where, $l^+ = l/h$, $u^+ = u/u_*$ and $z^+ = z/h$. In order to determine mixing length l^+ , the measured time averaged velocity curves were used to calculate the velocity gradients $\partial u^+ / \partial z^+$ and the values of τ_{uw}^+ were obtained directly from the measured RSS distributions. The distribution of non-dimensional Prandtl mixing length as a function of non-dimensional flow depth has been shown in Figures 3.16, 3.17 and 3.18 for both no seepage and seepage runs. In general, the

distribution pattern of l is almost similar to that of ε_f having some noise in the distributions with the application of seepage. Within the inner flow layer ($0 < z/h < 0.2$), there is an acceptable linear fit between l^+ and z^+ defined by $l^+ = kz^+$ where k is von Karman constant. The slope of this linear fit k decreases with the effect of seepage suggests that the traversing length of turbulent eddy decreases, which further signifies the increase in sediment transport in the channel under the action of downward seepage. It is also noticed from Figure 3.16-3.18 that value of von Karman constant observed from the Prandtl mixing length is almost similar as value observed from the velocity logarithmic law profile (Figure 3.3) for both no seepage and seepage cases. Also, the present experimental data points collapse satisfactorily with the curves defined by the theoretical equation $l^+ = kz^+(1 - z^+)^{0.5}$ given by **Nezu and Nakagawa (1993)**.

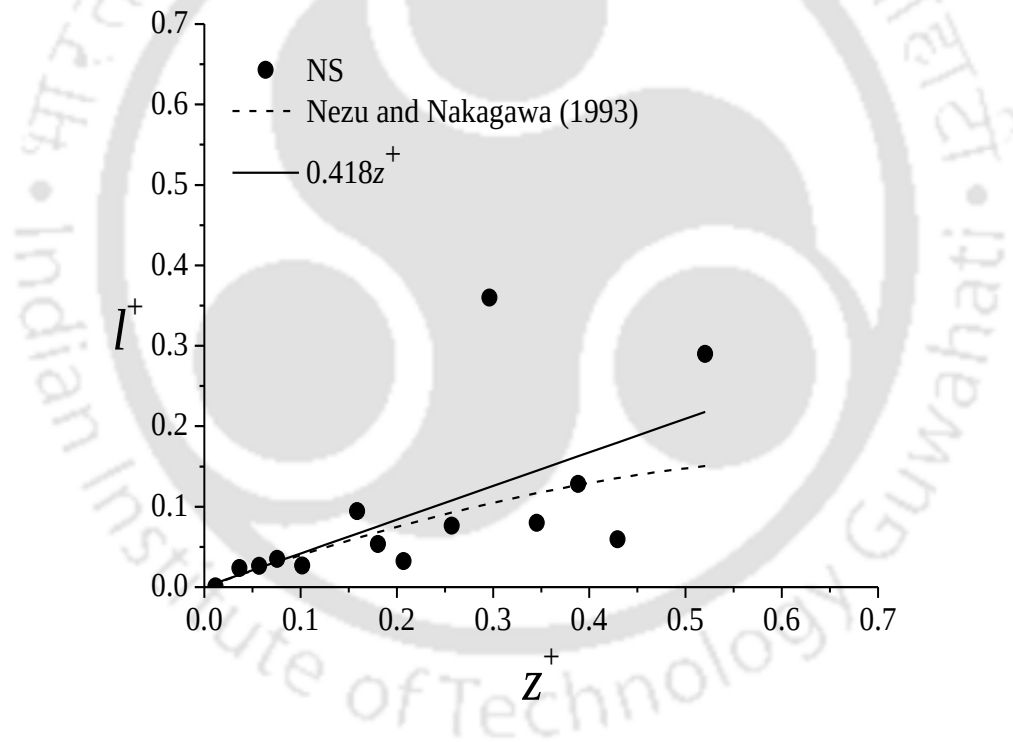


Figure 3.16: Distribution of Prandtl mixing length of flow subjected to no seepage (NS)

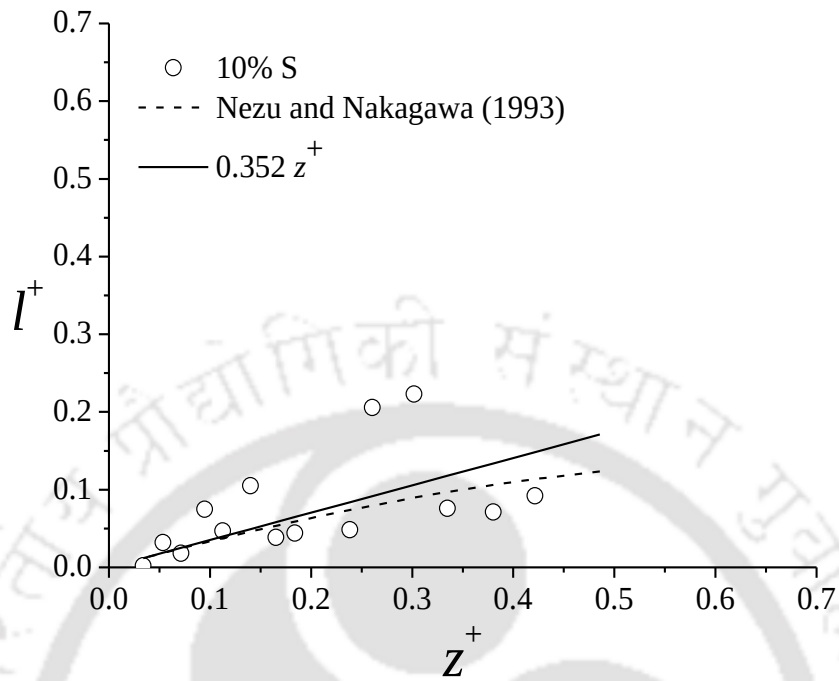


Figure 3.17: Distribution of Prandtl mixing length of flow subjected to 10% seepage (10%S)

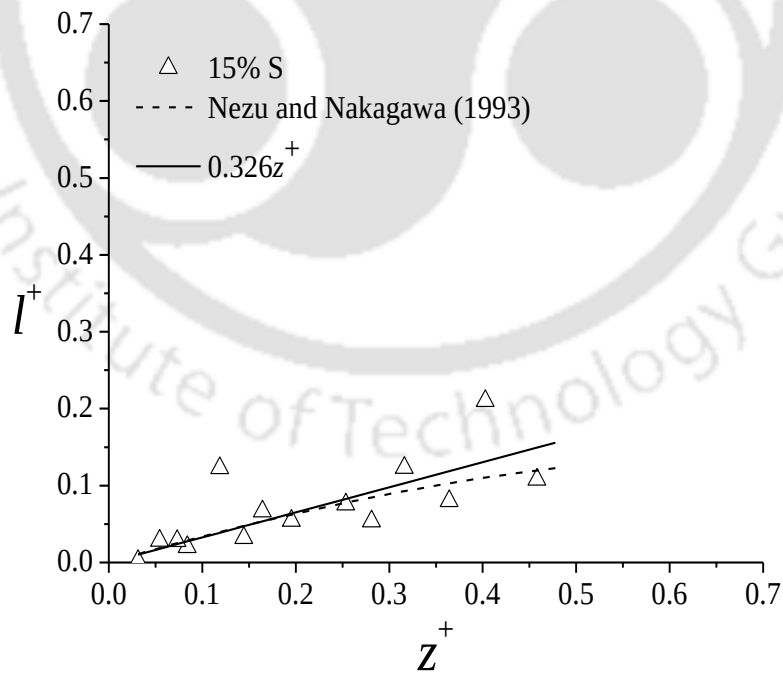


Figure 3.18: Distribution of Prandtl mixing length of flow subjected to 15% seepage (15%S)

3.8 Quadrant Analysis

The Reynolds shear stress at any given point is the summation of different types of bursting events. To quantify the bursting events, it is necessary to plot velocity fluctuation u' and w' on $u'w'$ plane (**Lu and Willmarth, 1973**). The hyperbolic hole region H is introduced to differentiate the larger contributions to $-\overline{u'w'}$ from each quadrant leaving the smaller u' and w' corresponding to more quiescent periods. The hole region H also represents the threshold level which is determined by the curve $|u'w'| = H \left(\overline{u'^2} \right)^{0.5} \left(\overline{w'^2} \right)^{0.5}$ (**Nezu and Nakagawa, 1993**). Hence, depending upon the relative sign of velocity fluctuations u' and w' , the bursting events are defined by four quadrants such as outward interactions ($i=1, u'>0, w'>0$), ejections ($i=2, u'<0, w'>0$), inward interactions ($i=3, u'<0, w'<0$) and sweeps ($i=4, u'>0, w'<0$). Graphical representation of four quadrants and hole region has been depicted in Figure 3.19. Pairs of (u', w') in Cartesian coordinate system are shown in Figures 3.20-3.22 for no seepage and seepage experiments, respectively, at $z/h \sim 0.3$.

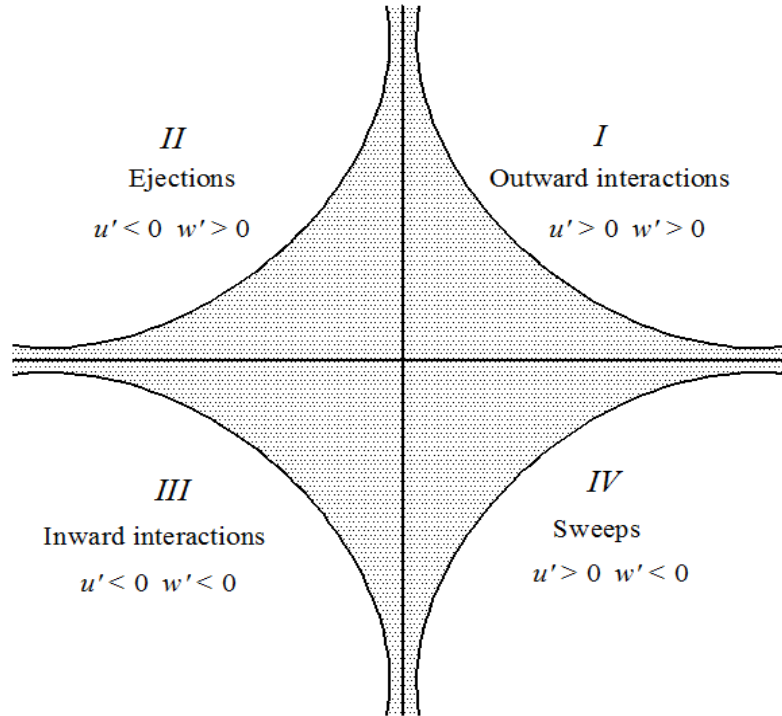


Figure 3.19: Graphical representation of quadrants and hole region

At any point, the contribution to the total Reynolds stress through the different forms of momentum transfer can be calculated as:

$$\langle u'w' \rangle_{i,H} = \lim_{T \rightarrow \infty} \frac{1}{T} \int_0^T u'(t)w'(t)I_{i,H}[u'(t)w'(t)]dt \quad (3.12)$$

Where t is time, T is sampling duration, $I_{i,H}$ is the indicator function defined as

$$I_{i,H}[u'(t)w'(t)] = \begin{cases} 1, & \text{if } (u', w') \text{ is in quadrant } i \text{ and if } |u'w'| \geq H \left(\overline{u'^2} \right)^{0.5} \left(\overline{w'^2} \right)^{0.5} \\ 0, & \text{otherwise} \end{cases} \quad (3.13)$$

The quadrant analysis provides the estimation of stress fraction transported by the contribution is

$$S_{i,H} = \frac{\langle u'w' \rangle_{i,H}}{\overline{u'w'}} \quad (3.14)$$

Here, $S_{i,H}$ is positive for $i=2$ and 4 (Q2 and Q4) and $S_{i,H}$ is negative for $i=1$ and 3 (Q1 and Q3).

Hence, sum of contributions from bursting events is unity that is $\sum_{i=0}^{i=4} [S_{i,H}]_{H=0} = 1$.

Conditional statistics through the quadrant analysis provides essential information of the coherent structure of turbulent flow in the near wall region. It has been observed through the flow visualization techniques (**Kline et al., 1967**) that relatively slow moving fluid parcel in the region very near to the wall, drifts away from the wall. At some distance, it moves violently away from the wall, oscillates, breaks down and merges in the flow layers. This process in which the fluid parcel moves slowly away from the wall is called ejection. In order to maintain continuity, high speed fluid moves towards the wall which is called sweeps. These combined events are called bursting and provide characteristic behavior to the turbulence. **Grass (1971)**, **Nakagawa and Nezu (1977)**, and **Raupach (1981)** have observed that largest contribution to the Reynolds shear stress come from both ejections and sweeps throughout the boundary layer. Although ejections dominate in most of the thickness of boundary layer, sweeps are prevalent in the near boundary region. Thus, the roughness characteristics and sediment transport are associated mainly to the sweep events which are dominant in contribution to the momentum transfer in the region close to the boundary.

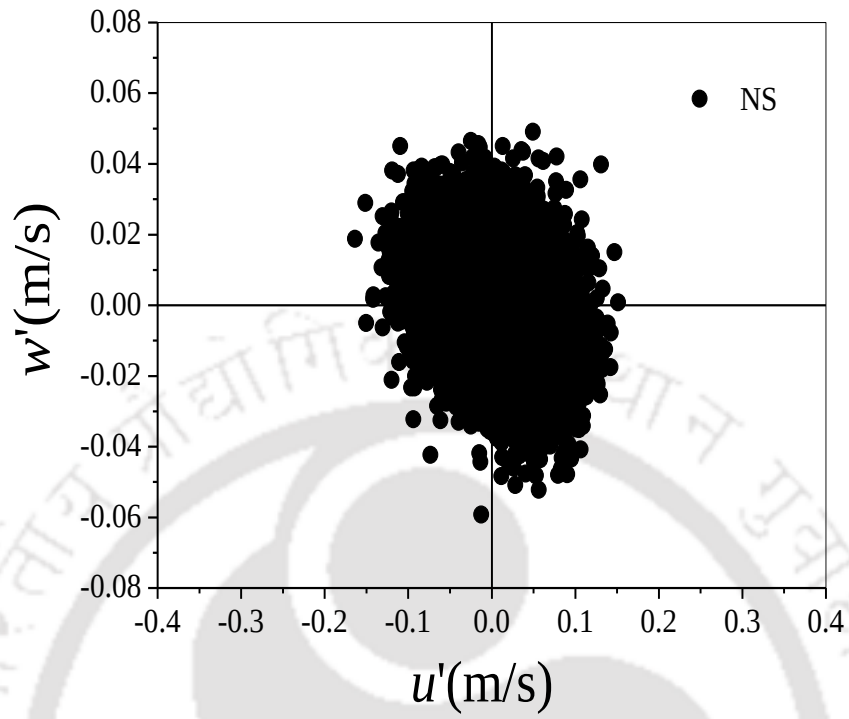


Figure 3.20: (u', w') for no seepage experiment

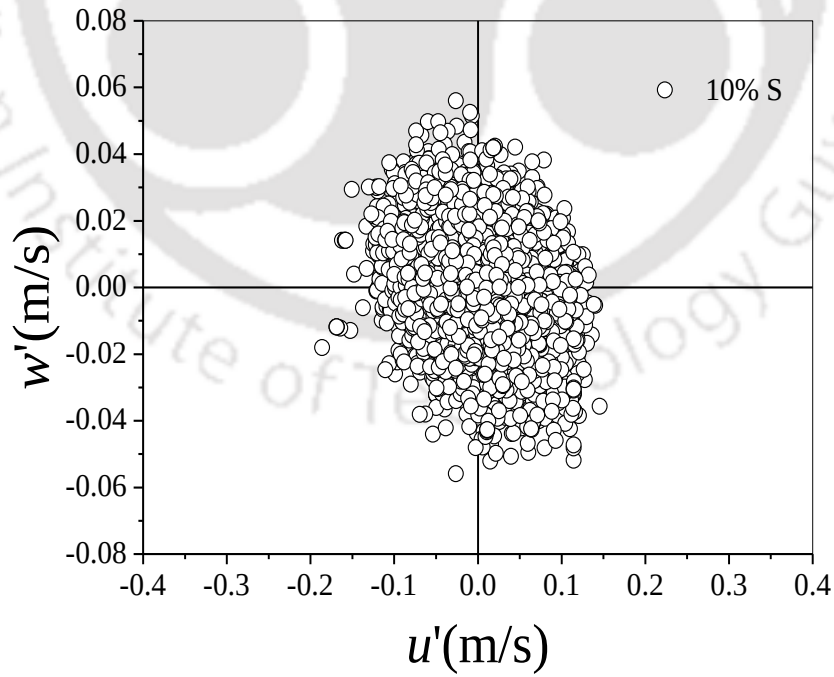


Figure 3.21: (u', w') for 10% seepage experiment

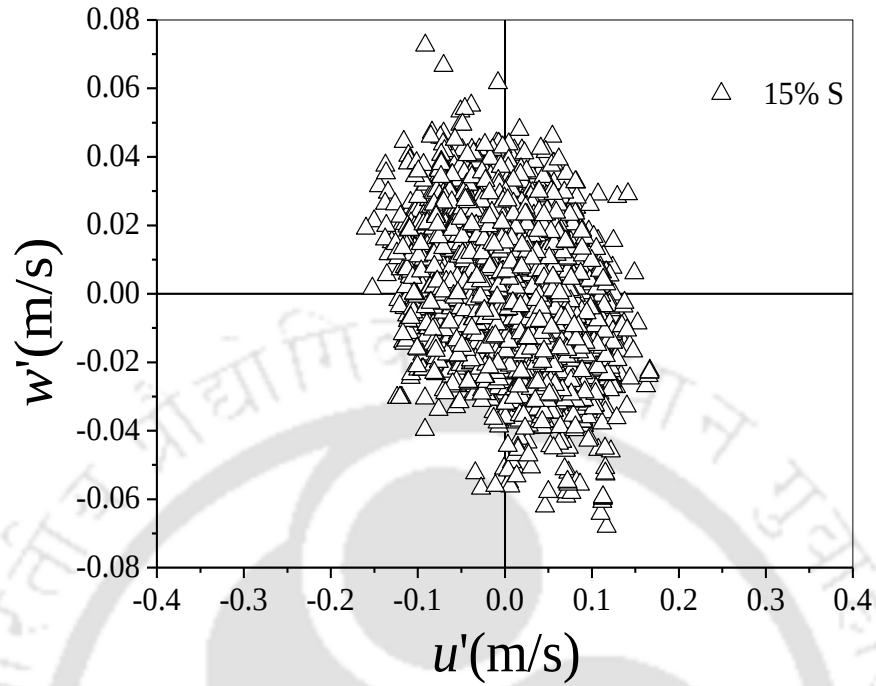


Figure 3.22: (u', w') for 15% seepage experiment

Figure 3.23-3.26 shows the variation of fraction contribution $S_{i,H}$ with hole size H for each of the four quadrants at $z/h=0.036$ (near the boundary) subjected to no seepage and seepage experiment. Governing events are Q2 and Q4 for no seepage flow while Q1 and Q3 rarely contribute towards the production of RSS. It was also examined from Figure 3.23-3.26 that the fractional contribution from Q1 and Q3 events vanishes for $H>7$ and $H>4$ respectively. On the other hand, contribution from the Q2 and Q4 events vanishes for $H>9$ and $H>10$ respectively. However, $S_{i,H}$ is slightly affected with seepage in such a way that variations of $S_{i,H}$ with H concludes that Q2 and Q4 events are lesser and higher respectively than that with no seepage. As a downward seepage generate the withdrawal of momentum in the near bed region which causes an increase in Q4 events and decrease of Q2 events, resulting in arrival of a high speed fluid parcel due to acceleration of fluid in stream wise direction. In associated with seepage flow, the feebly contributions from both Q1 and Q3 events cease for $H>8$ (10%S case) and $H>6$ (15%S case) while Q2 and Q4 events dominate even for $H > 11$ with sweeps still being the largest contributor towards the production of RSS.

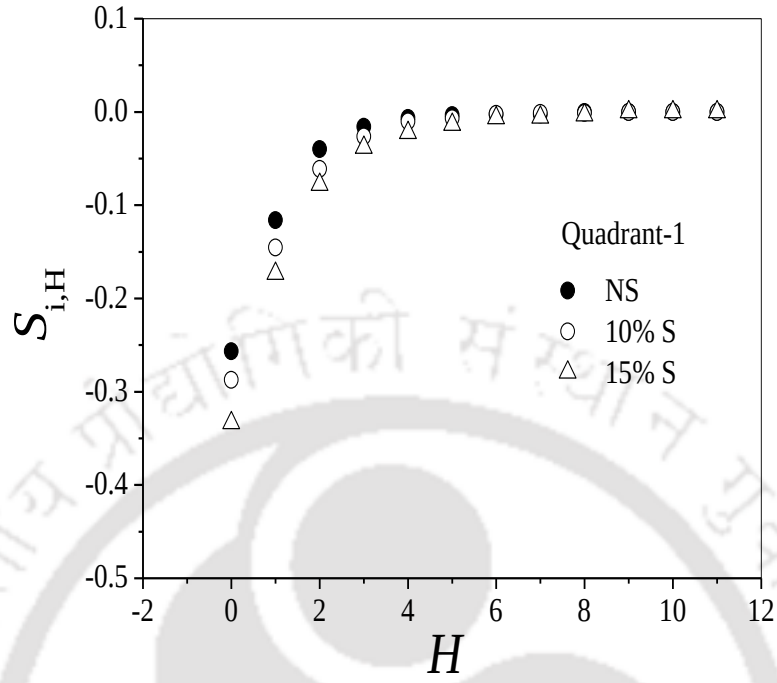


Figure 3.23: Variations of $S_{i,H}$ with H for outward interactions (Quadrant-1)

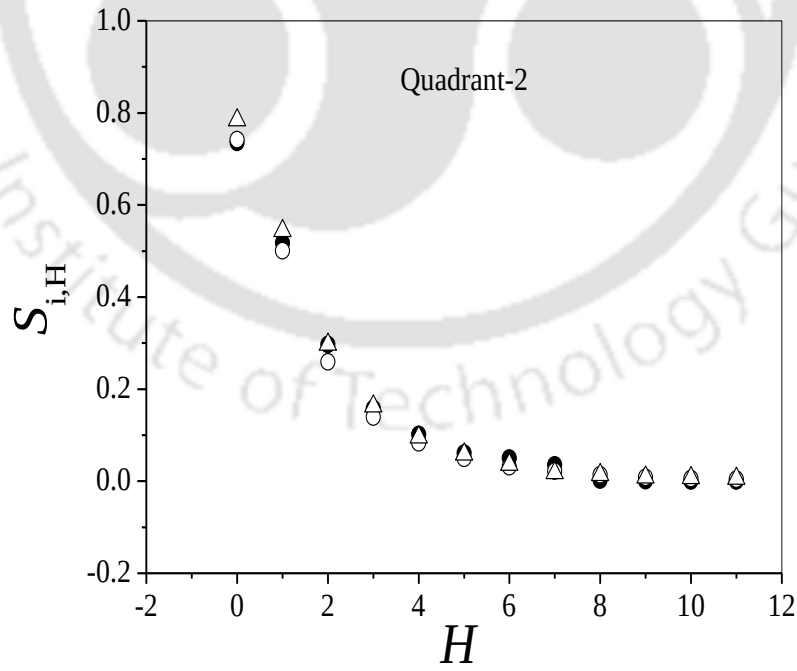


Figure 3.24: Variations of $S_{i,H}$ with H for ejections (Quadrant-2)

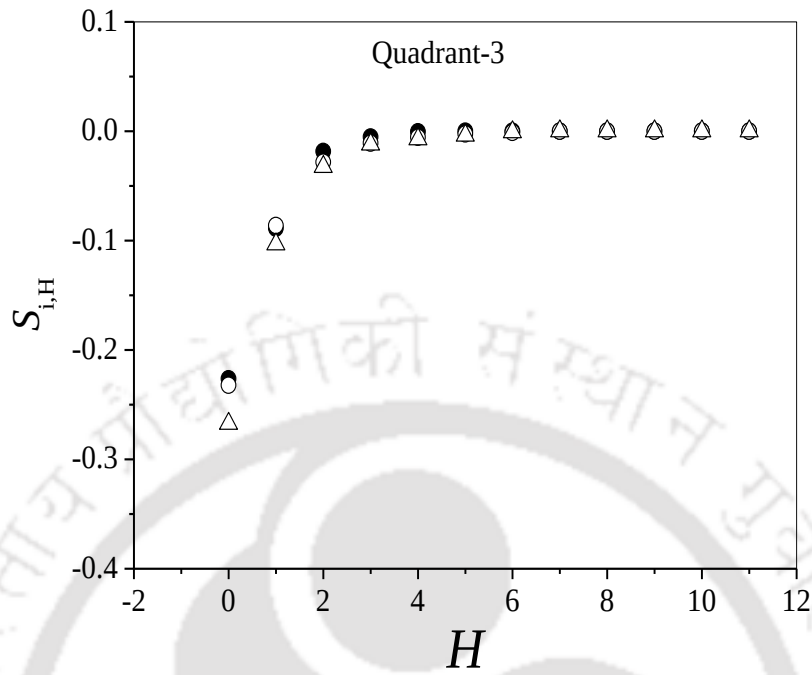


Figure 3.25: Variations of $S_{i,H}$ with H for inward interactions (Quadrant-3)

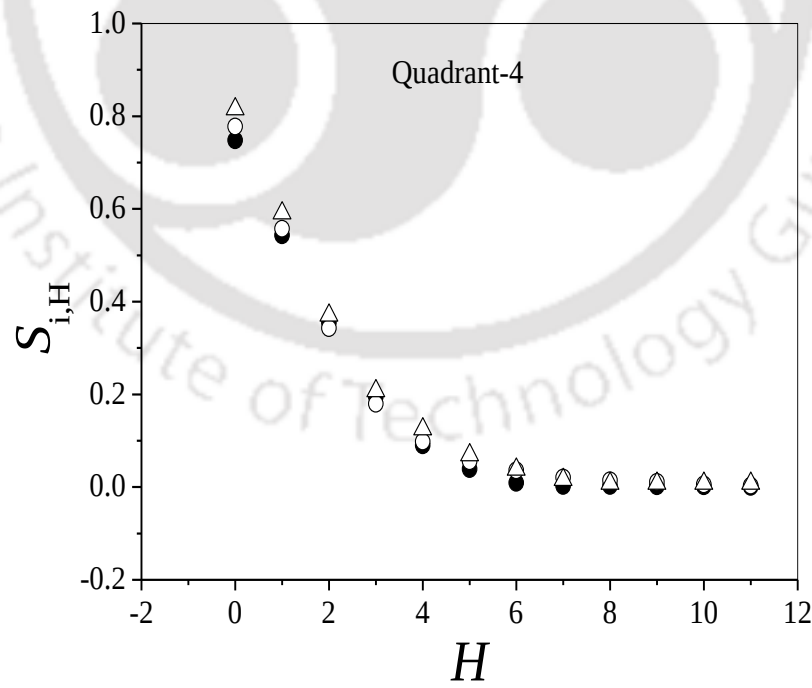


Figure 3.26: Variations of $S_{i,H}$ with H for sweeps (Quadrant-4)

The vertical distributions of $S_{i,H}$ in flows subjected to no seepage and seepage are examined. The most frequent events $H=0$ and $H=2$ is considered associated with the use of u' and w' in flows subjected to no seepage and seepage cases (Figure 3.27-3.32). In Figure 3.27-3.32, the contributions from Q2 and Q4 events are positive while those from Q1 and Q3 events are negative. For $H=0$, primary contribution is from Q2 and Q4 events and feebly contribution from Q1 and Q3 towards the RSS production. The divergence in between Q2 and Q4 events is accomplished with the large scale of eddies. This nature is in conformity with flows over rough and smooth beds (Lu and Willmarth, 1973; Raupach, 1981; Cellino and Lemmin, 2004; Deshpande and Kumar, 2016a). In Figure 3.27 with $H=0$, Q2 and Q4 events in near bed ($z/h \sim 0.03$) contribute about 73.5% and 74.7% respectively towards the production of RSS while Q1 and Q3 events contribute minimal as 25.6% and 22.6% respectively. The dominance of sweep events over ejection extends nearly up to $z/h \sim 0.057$. While away from the bed ($z/h > 0.057$), ejection events dominate in contributing to the Reynolds shear stress production. At this junction point of flow layer ($z/h \sim 0.057$), average contributions from ejections and sweeps are 74% and 77%, respectively. For 10%S (Figure 3.28), within the near bed surface ($z/h \sim 0.03$), Q2 and Q4 events contribute about 74.2% and 77.7% respectively while Q1 and Q3 events contribute minimal as 28.7% and 23.2% respectively. With the increase of seepage to 15% at near the bed ($z/h \sim 0.03$) as shown in Figure 3.29, contribution from Q4 and Q2 events becomes 78.4% and 81.55% respectively while feebly contribution from Q1 and Q3 becomes 33.3% and 26.7% respectively. The result predicts that the contribution from all bursting events to RSS production in the near bed flow zone increases with the increase of percentage of seepage as compared to that with no seepage run. Further the present result suggests that dominants of sweep events over the ejection events at near the bed increases progressively with the further increase of downward seepage. The thickness of the zone of sweep dominance increases from $z/h \sim 0.057$ with no seepage to $z/h \sim 0.09$ and $z/h \sim 0.12$ in the seepage run of 10%S and 15%S. These increased contributions from sweep events and thickening of the zone of sweep dominance in the seepage run increase momentum exchange from the flow to bed particles due to which sediment transport is increased with the application of downward seepage as compared to no seepage. Hence, for downward seepage, sweep is the major events in the near bed flow zone. With the application of downward seepage, the high speed fluid parcel becomes more persistently preserving near to the bed as compared to that with no seepage. Compared to the RSS

contributions from more extreme events occurring for hole size $H=2$ for both no seepage and seepage run (Figure 3.30-3.32), the most energetic events Q2 and Q4 have distinct behaviours throughout the flow depth, since the divergence between Q2 and Q4 events gets stronger with z/h . However, there remains a consensus that there remains a similar predominating feature of Q2 and Q4 events within the wall shear layer while contributions from Q1 and Q3 events remain insignificant.

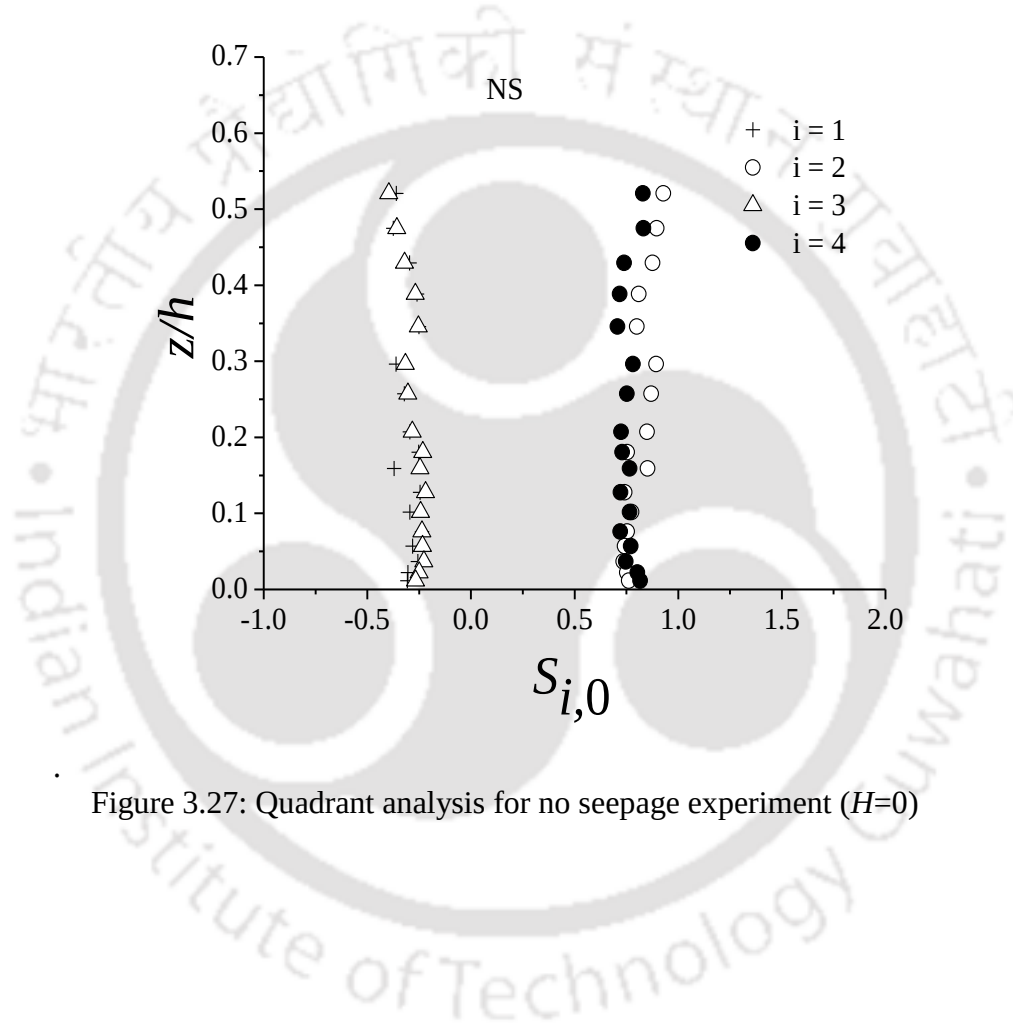


Figure 3.27: Quadrant analysis for no seepage experiment ($H=0$)

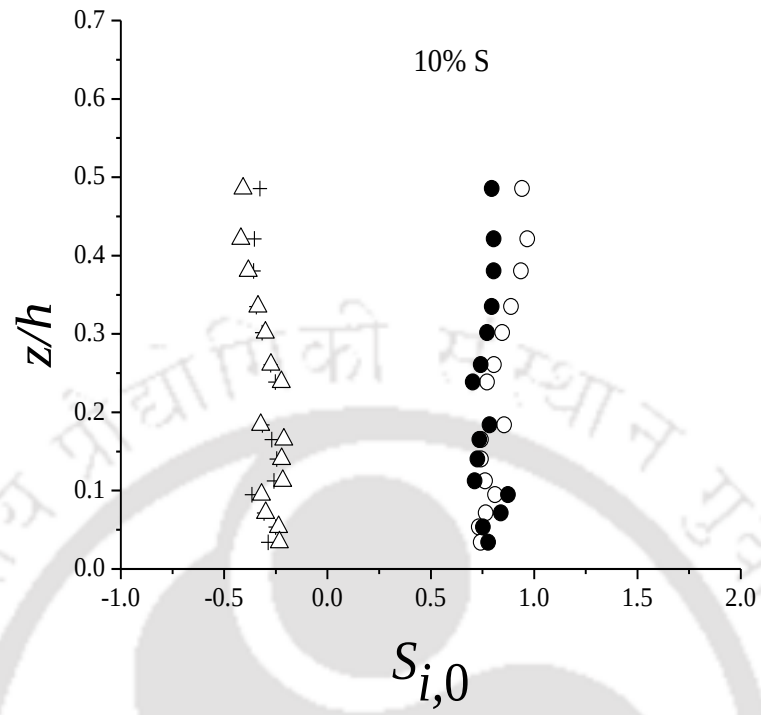


Figure 3.28: Quadrant analysis for 10% seepage experiment ($H=0$)

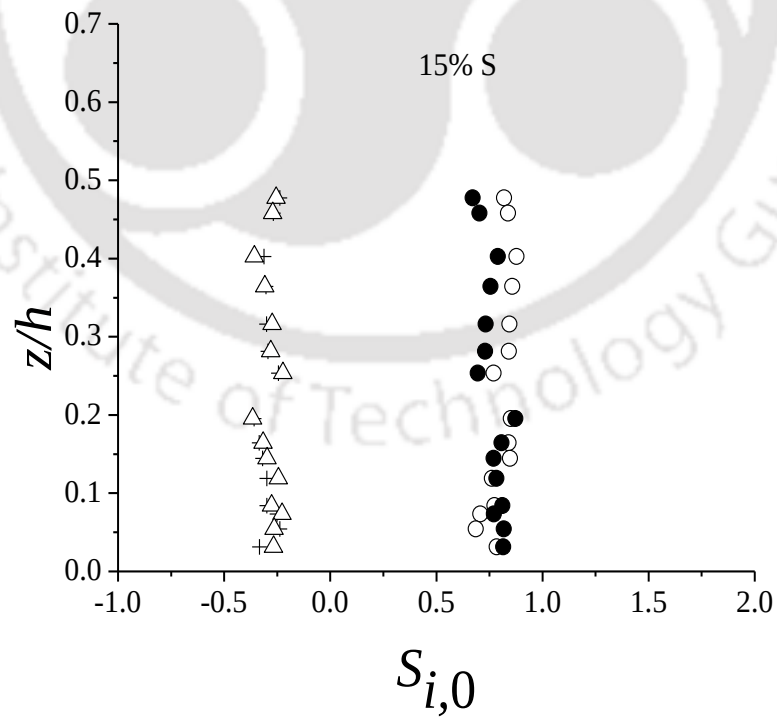


Figure 3.29: Quadrant analysis for 15% seepage experiment ($H=0$)

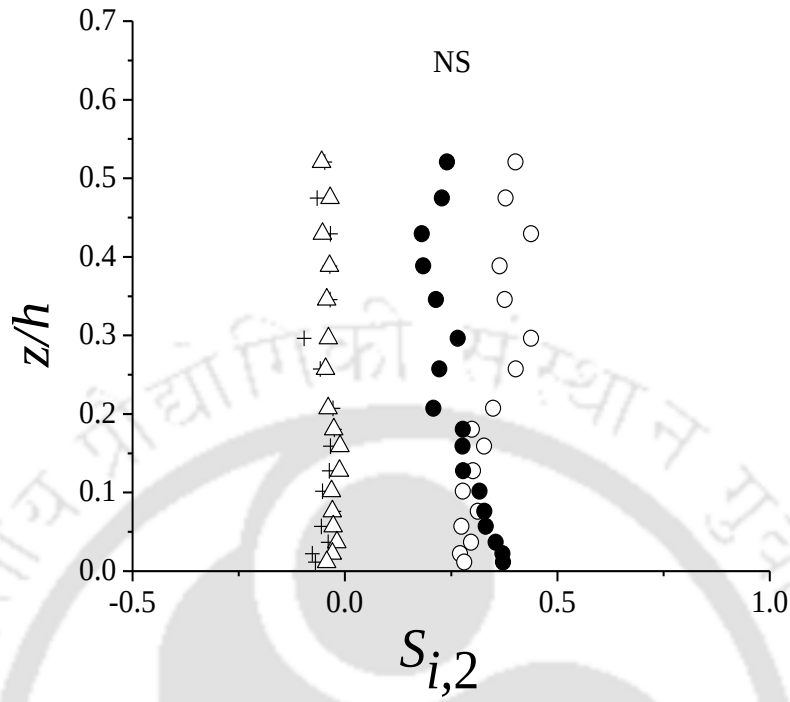


Figure 3.30: Quadrant analysis for no seepage experiment ($H=2$)

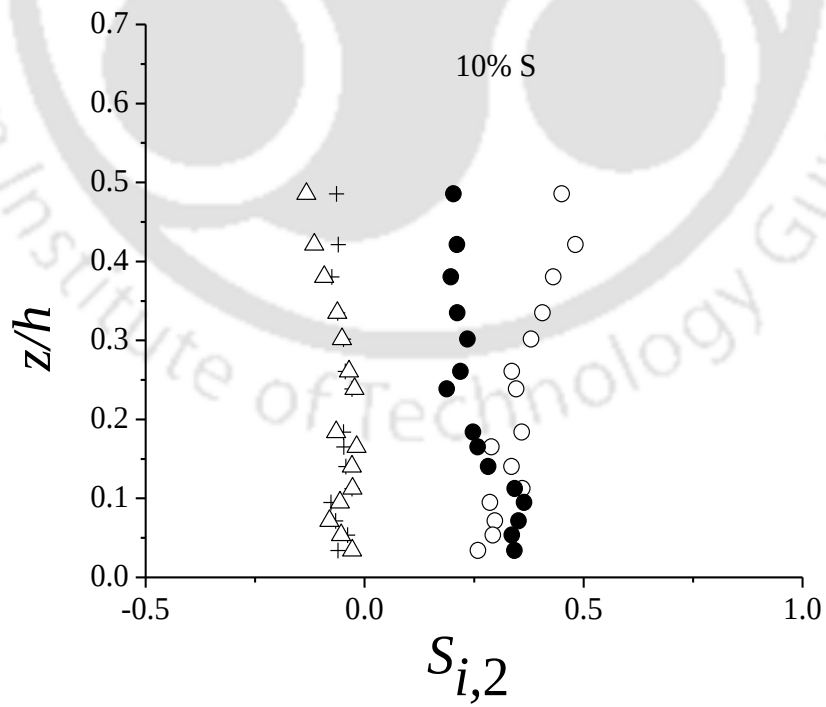


Figure 3.31: Quadrant analysis for 10% seepage experiment ($H=2$)

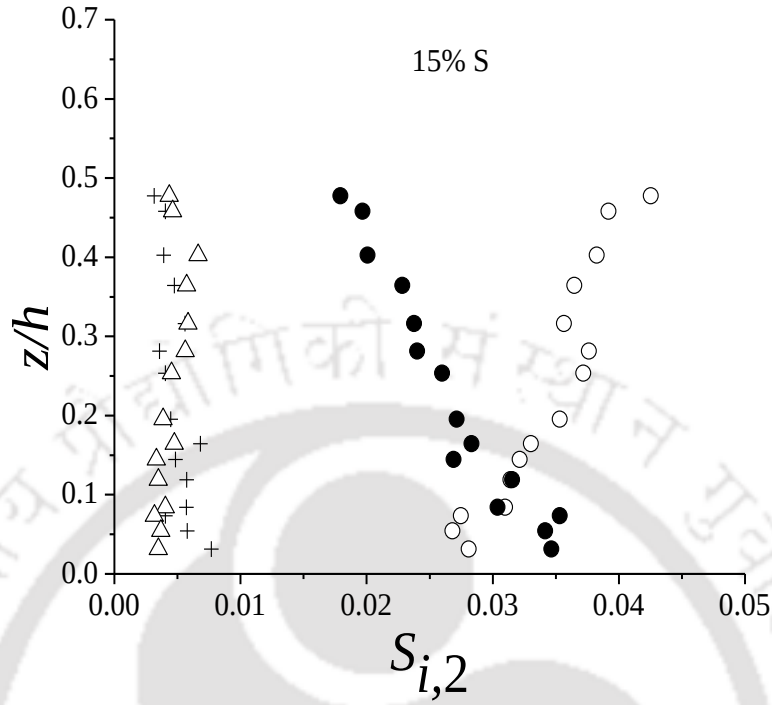


Figure 3.32: Quadrant analysis for 15% seepage experiment ($H=2$)

The probability $P_{i,H}$ of occurrence of the bursting events is expressed as (Cellino and Lemmin, 2004):

$$P_{i,H} = \frac{\int_{t=0}^{t=T} I_{i,H} dt}{\int_{t=0}^{t=T} [I_{1,H} + I_{2,H} + I_{3,H} + I_{4,H}] dt} \quad (3.15)$$

Where $I_{i,H}$ is an indicator function described by the Equation (3.13). The vertical variations of $P_{i,H}$ in flows subjected to no seepage, 10% S and 15% S are plotted in Figure 3.33-3.38. The probabilities are calculated for hole region $H=0$ and $H=2$. However for $H=0$ and $H=2$ subjected to no seepage case, values of $P_{1,H}$ and $P_{3,H}$ for Q1 and Q3 events overlap bearing small probabilities. Considering $H=0$, $P_{2,0}$ and $P_{4,0}$ for Q2 and Q4 events are dominant events in the near bed surface for no seepage runs. But, the dominance of $P_{2,H}$ changes to $P_{4,H}$ with an increase in hole size ($H=2$). Therefore, the strongest bursting events are $P_{4,2}$ (Q4 events) while the most frequent ones are $P_{2,0}$ (Q2 events). Interestingly, $P_{i,H}$ ($i=1, 3$) and $P_{4,0}$ remain almost invariant of z/h ; whereas $P_{2,0}$ and $P_{4,2}$ decrease with z/h . Although there is no significant difference observed

in the profiles of $P_{i,H}$ for no seepage and seepage, near boundary values of $P_{4,H}$ ($H=0,2$) are greater associated with the smaller of $P_{2,H}$ ($H=0, 2$) with seepage than that with no seepage.

In order to study the time scales of Q2 and Q4 events, the occurrences of bursting events Q2 and Q4 in a sample were counted by keeping a hole size H and calculating the number of changeover of the series of (u', w') in the applicable quadrant. The mean time of occurrence of Q2 and Q4 events t_E and t_S respectively were thus obtained. Figures 3.39-3.42 show the vertical profiles of normalised time scales $T_E (=t_E u^*/y)$ and $T_S (=t_S u^*/y)$ in flows subjected to no seepage and seepage for $H=0$ and 2. Generally for $H=0$, T_E increases with z/h while T_S decreases with z/h . But the phenomenon has been changed for $H=2$ in case of no seepage and seepage runs. It suggests that the time of occurrence of stronger Q4 events is less than that of stronger Q2 events. It is evident from Figures 3.39-3.42 that both T_E and T_S in seepage are more persistent than those in no seepage.

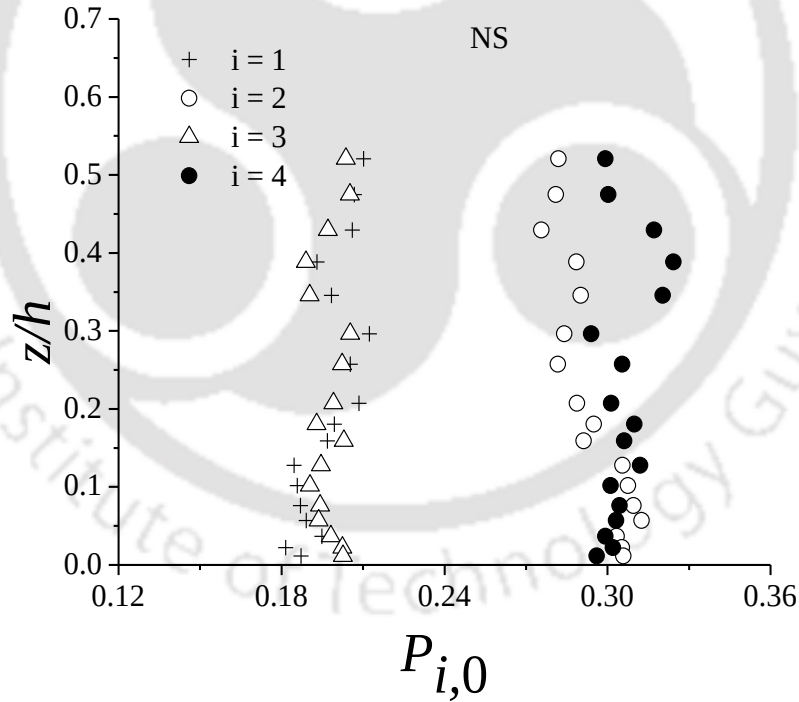


Figure 3.33: Vertical profiles of $P_{i,H}$ in flows subjected to NS for $H=0$

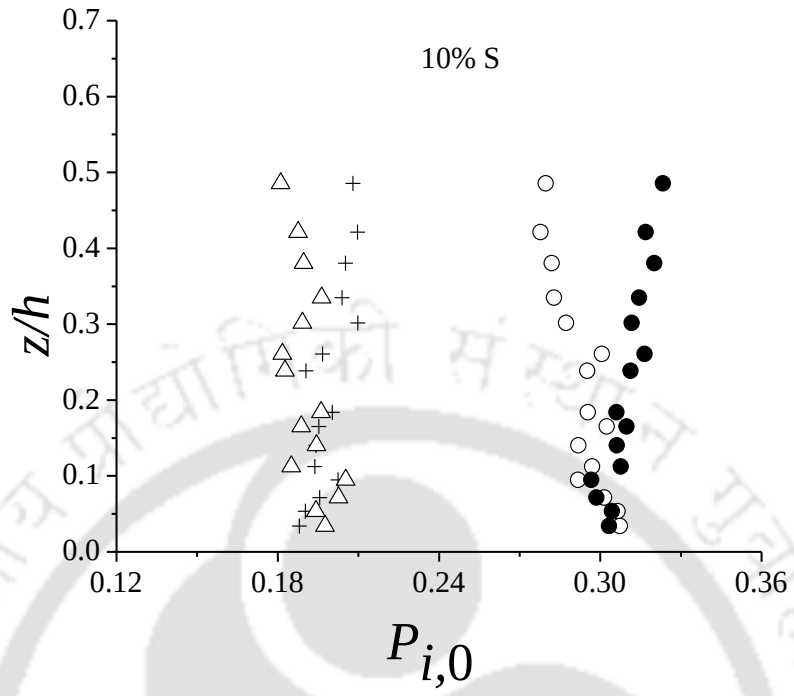


Figure 3.34: Vertical profiles of $P_{i,H}$ in flows subjected to 10% S for $H=0$

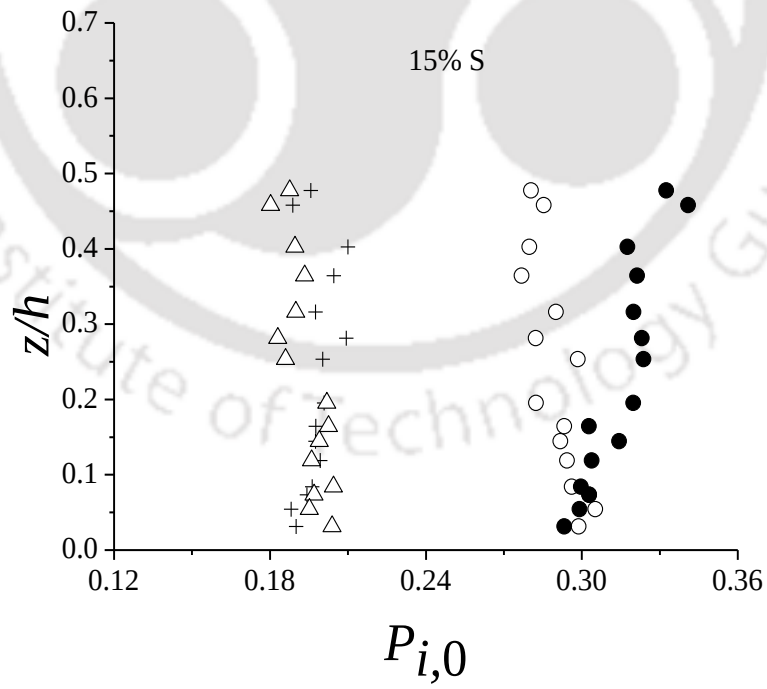


Figure 3.35: Vertical profiles of $P_{i,H}$ in flows subjected to 15% S for $H=0$

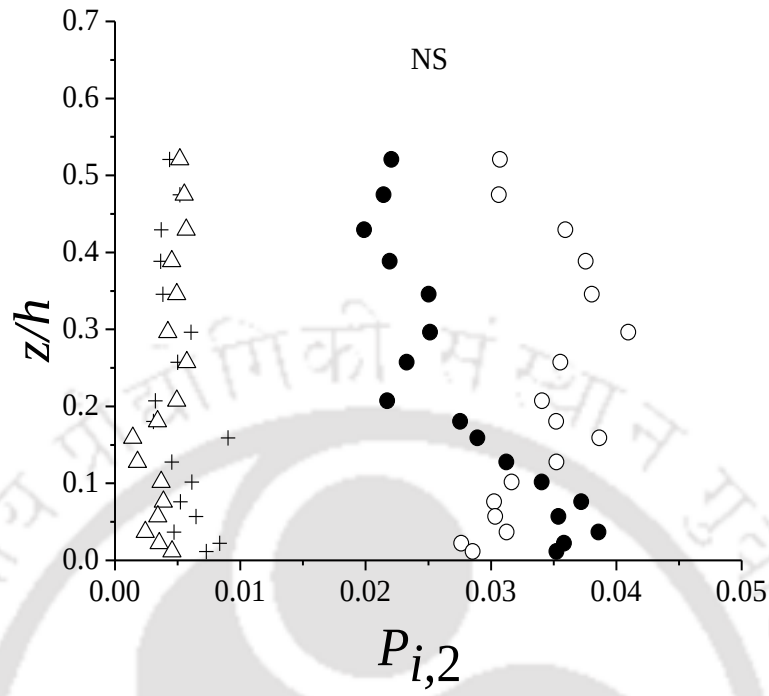


Figure 3.36: Vertical profiles of $P_{i,H}$ in flows subjected to NS for $H=2$

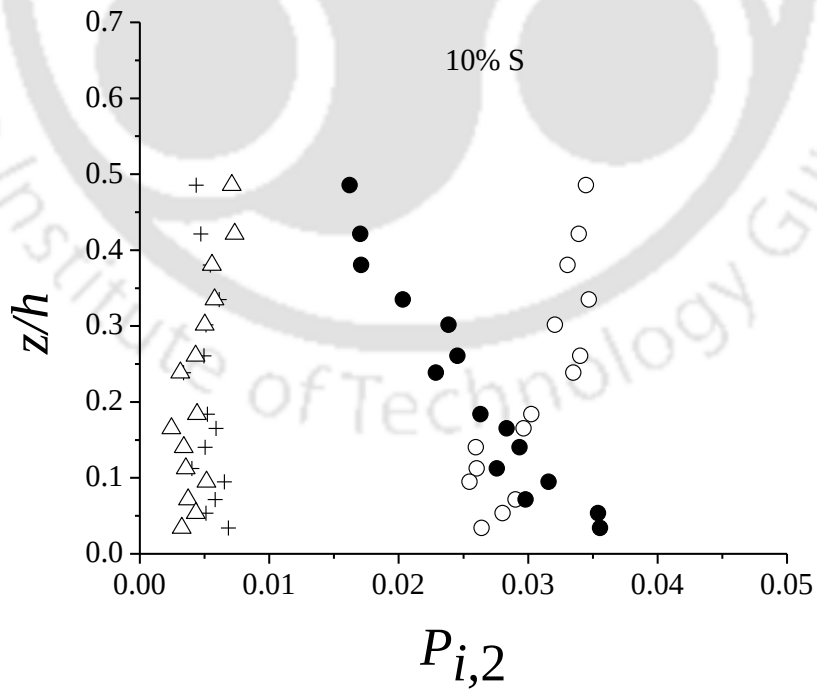


Figure 3.37: Vertical profiles of $P_{i,H}$ in flows subjected to 10% S for $H=2$

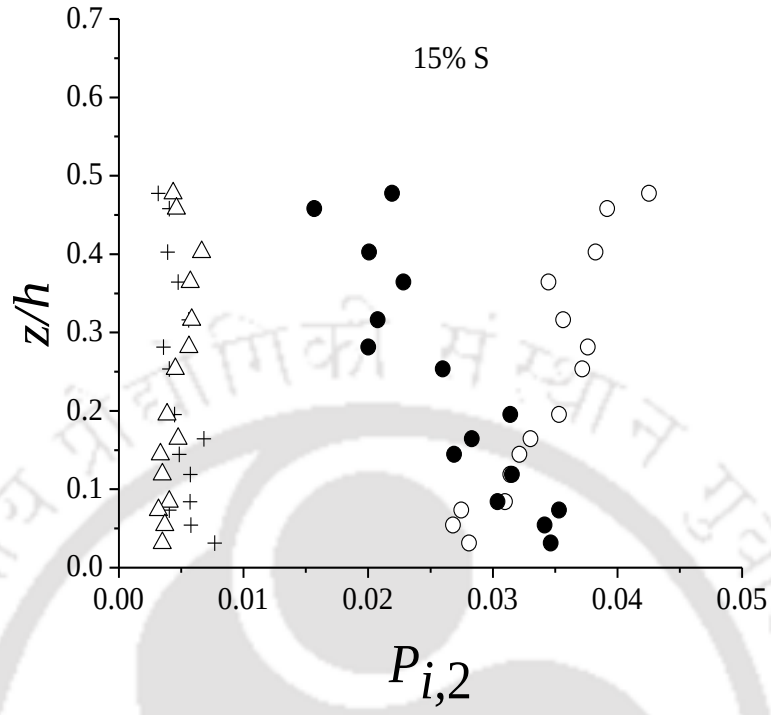


Figure 3.38: Vertical profiles of $P_{i,H}$ in flows subjected to 15% S for $H=2$

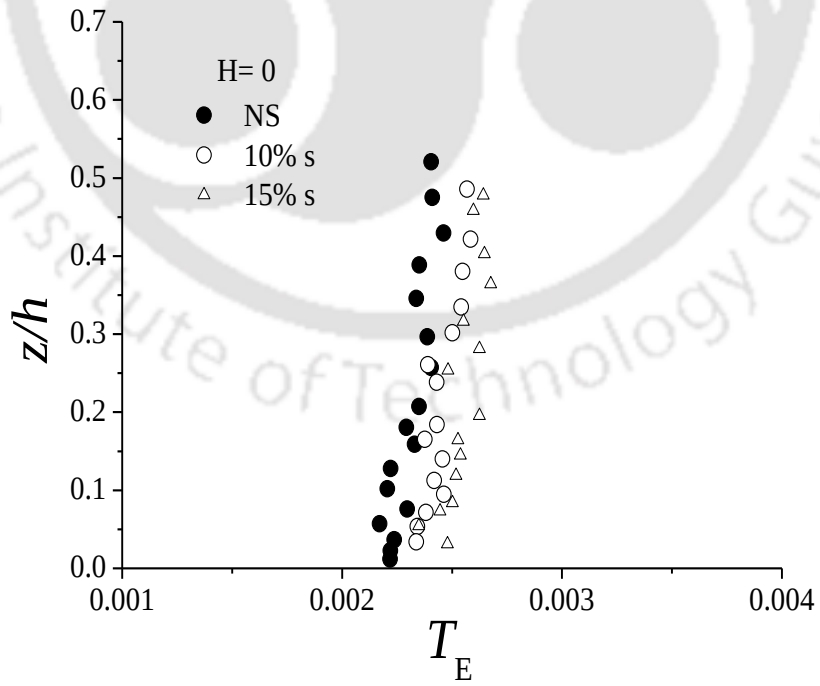


Figure 3.39: T_E as a function of flow depth for $H=0$ subjected to NS, 10% S and 15% S.

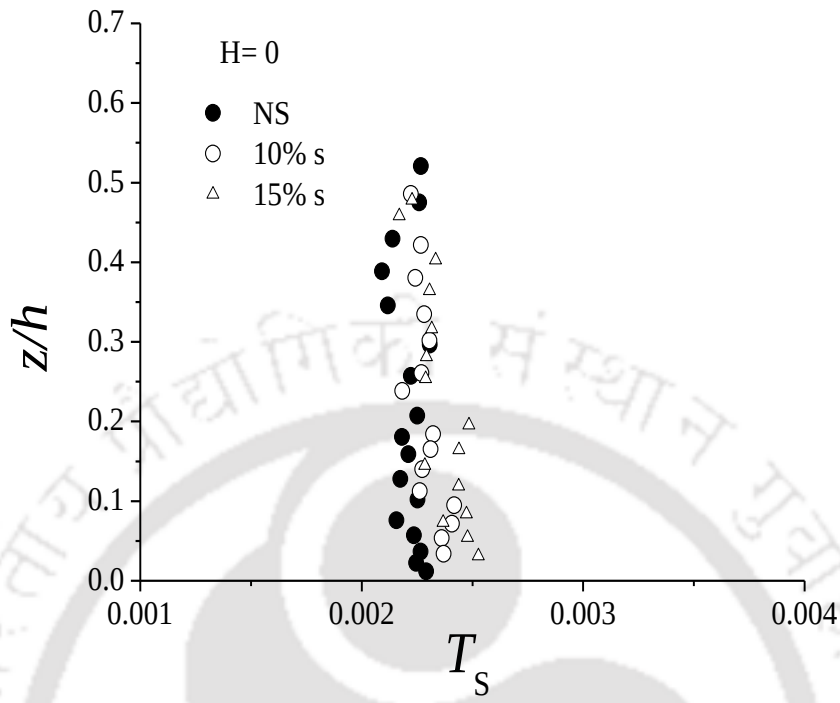


Figure 3.40: T_s as a function of flow depth for $H=0$ subjected to NS, 10%S and 15%S.

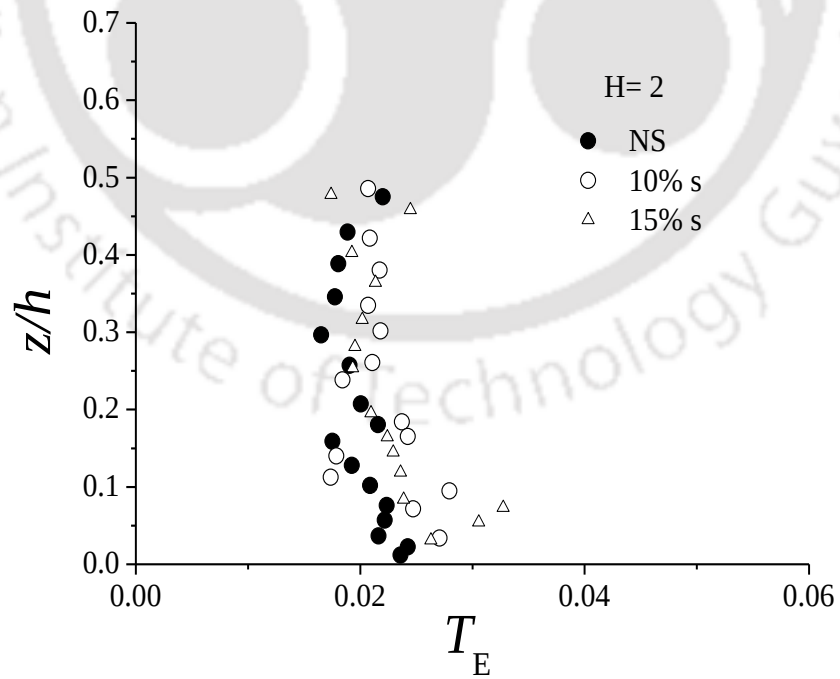


Figure 3.41: T_E as a function of flow depth for $H=2$ subjected to NS, 10%S and 15%S.

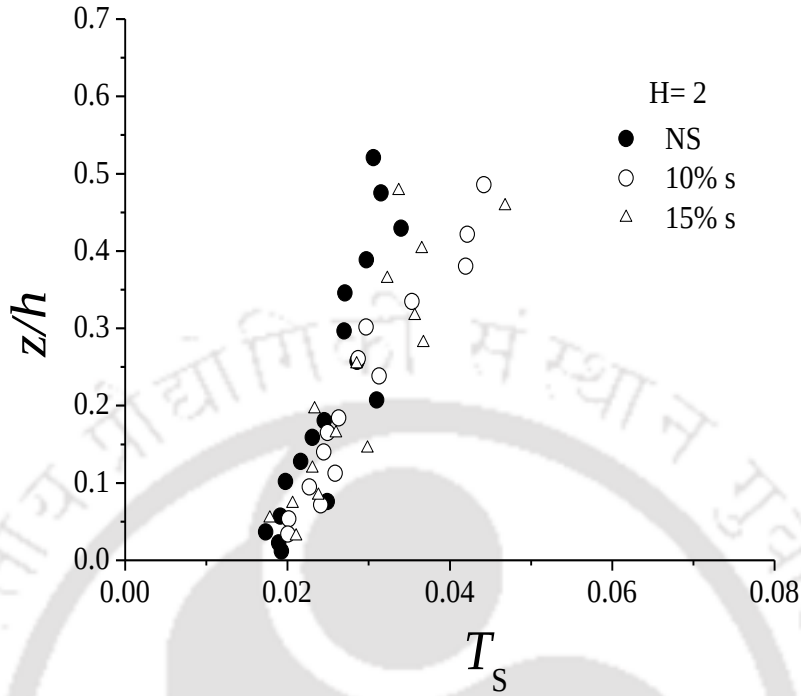


Figure 3.42: T_s as a function of flow depth for $H=2$ subjected to NS, 10%S and 15%S.

3.9 Higher Order Correlations of Velocity Fluctuations

Despite many studies on the turbulent flow characteristics over smooth and rough boundaries, still there is a lack in understanding the third order and fourth order correlations in flows subjected to downward seepage. The third order correlation or skewness indicates non-symmetric distributions which are required to get the information about the flux of stresses caused by the turbulence. Symmetric distribution about the mean or the Gaussian distribution indicates zero skewness while negative and positive skewness represents the distribution is skewed towards left and right respectively of the mean. On account of conservation of signs, third order correlation is directly linked with the turbulent coherent convey necessary information on the temporal features of the velocity fluctuations (**Gad-el-Hak and Bandyopadhyay, 1994**). The time averaged third order moments of velocity fluctuations can be expressed as (**Raupach, 1981**)

$$\left. \begin{aligned} M_{jk} &= \overline{\hat{u}^j \hat{w}^k} \text{ where } j+k=3 \\ \hat{u} &= \frac{u'}{(\overline{u'u'})^{0.5}}, \hat{w} = \frac{w'}{(\overline{w'w'})^{0.5}} \end{aligned} \right\} \quad (3.16)$$

where $M_{30} = \overline{\hat{u}^3}$ represents the stream wise flux of Reynolds normal stress (RNS) in stream wise direction, $M_{03} = \overline{\hat{w}^3}$ defines the vertical flux of RNS in vertical direction, $M_{12} = \overline{\hat{u}\hat{w}^2}$ represents the transport of vertical RNS in stream wise direction, $M_{21} = \overline{\hat{u}^2\hat{w}}$ characterising the transport of stream wise RNS in vertical direction. Figures 3.43-3.46 shows the individual profiles of the third order correlations M_{jk} for sets of no seepage and seepage run whose distributions are simply changed to some extent by the influence of seepage. The profiles of M_{jk} in the near bed flow zone are directly or indirectly proportional to each other. **Raupach (1981)** carried out the experiments in a wind-tunnel on rough wall boundary layer flows. He developed the relationship between third order correlations by a linear law as $M_{30} = -2.02M_{21} = 1.97M_{12} = -1.7M_{03}$. The results of the present work with no seepage confirm the linear law relationship but with slightly varying coefficients, which is $M_{30} = -2.18M_{21} = 1.94M_{12} = -1.75M_{03}$. With the application of seepage, the law changes to some extent. For 10%S, $M_{30} = -2.7M_{21} = 2.24M_{12} = -1.97M_{03}$ and for 15%S, $M_{30} = -2.57M_{21} = 2.36M_{12} = -1.66M_{03}$. **Raupach (1981)** found that irrespective of the surface roughness, third order moments follow universal patterns except very close to the boundary. In flows with no seepage and seepage, M_{30} starts with small positive value and M_{03} initiates with small negative value close to the boundary. Near the boundary, a strong mixing flow caused by the momentum exchange indicates Gaussian distribution of u' having a skewness tending to zero. Negative values of M_{30} corresponding to the data which is skewed left in the probability density distribution of u' . An examination of M_{jk} profiles reveals that there exists an apparent distinction in the data trends of M_{jk} with seepage from those of no seepage particularly in the near boundary region. Due to exchange of momentum through the boundary, seepage affects the near boundary data of M_{jk} in such a way that M_{30} near the bed is increased with the increase of seepage suggesting the strong influence of increased stream wise flux of stream wise RNS which increased the roughness concentration and particle mobility under the action of seepage. Similar trend was observed in the distribution of M_{12} which reflects the

increase in advection of vertical RNS in stream wise direction in seepage run. On the other hand, M_{03} and M_{21} close to the bed increases with the increase of downward seepage as compared to no seepage run, indicates the increase in vertical flux of vertical RNS and also increase in advection of stream wise RNS in downward direction. The effective response of the bursting events at a point can also probably identified by the third order correlation (**Nezu and Nakagawa, 1993**). It is cleared from Figures 3.43-3.44 that M_{30} and M_{03} are positive and negative respectively within the near bed surface imply Q4 events for both no seepage and seepage run. Away from the bed, the dominance of Q2 events causes M_{30} to be negative and M_{03} to be positive for the rest of the flow depth. This feature was also reported by **Grass (1971)** and **Krogstadt and Antonia (1999)** for smooth boundary. In addition, the transport of the Reynolds normal stresses was examined from M_{12} and M_{21} profiles (Figures 3.45-3.46). Within the near flow zone, the flows with no seepage and seepage correspond to a transport of vertical RNS in the stream wise direction and stream wise RNS in the vertical direction. Therefore, the present results provide clear information that for no seepage and seepage runs, a stream wise acceleration is associated with a downward flux causing Q4 events while a stream wise acceleration is associated with a transport in the upstream direction, representing Q2 events.

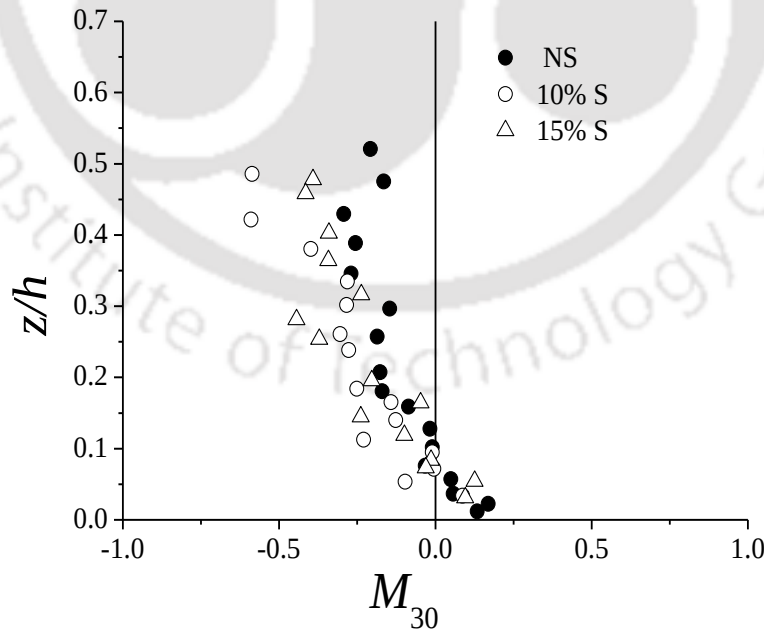


Figure 3.43: Vertical profiles of third order correlations M_{30} for no seepage and seepage flow

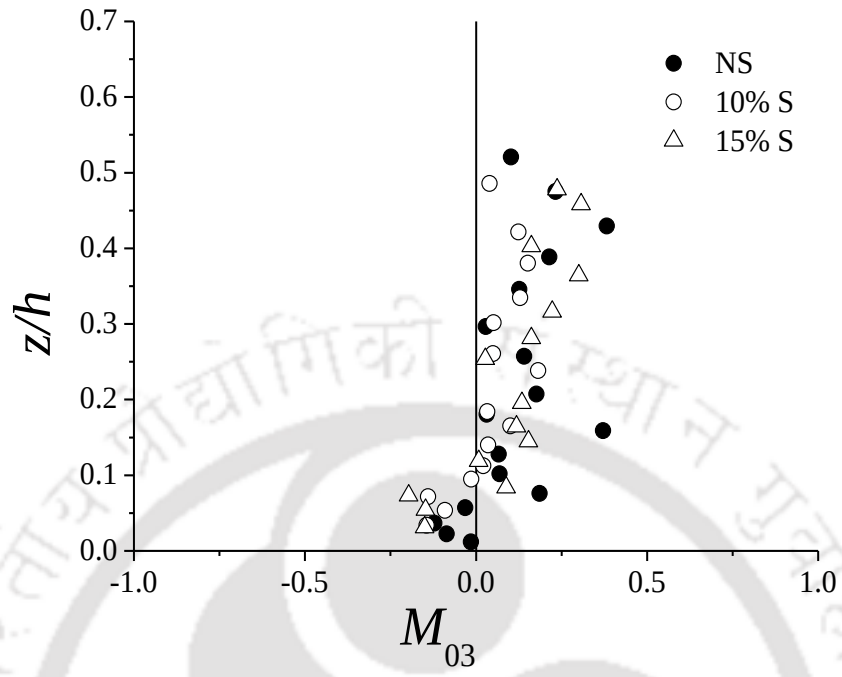


Figure 3.44: Vertical profiles of third order correlations M_{03} for no seepage and seepage flow

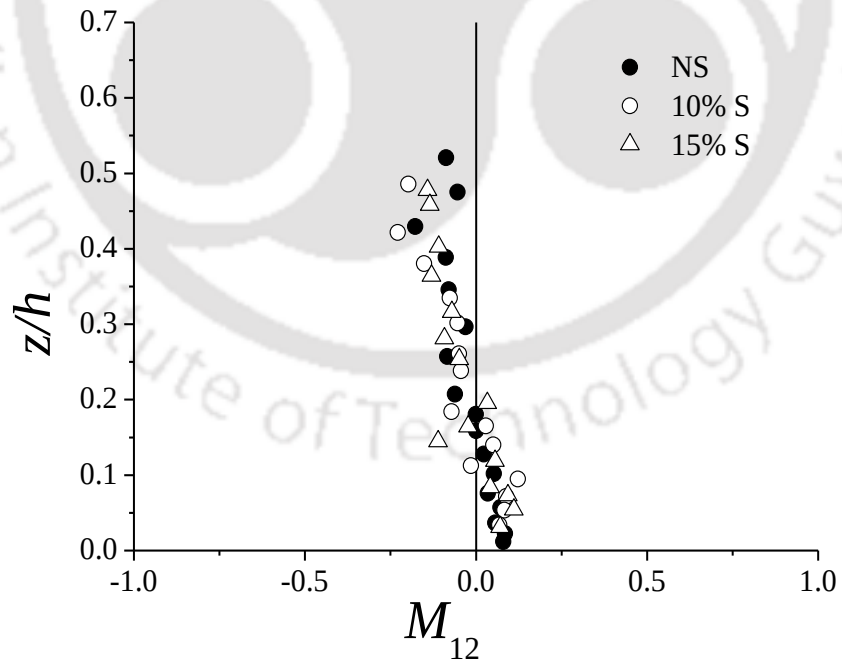


Figure 3.45: Vertical profiles of third order correlations M_{12} for no seepage and seepage flow

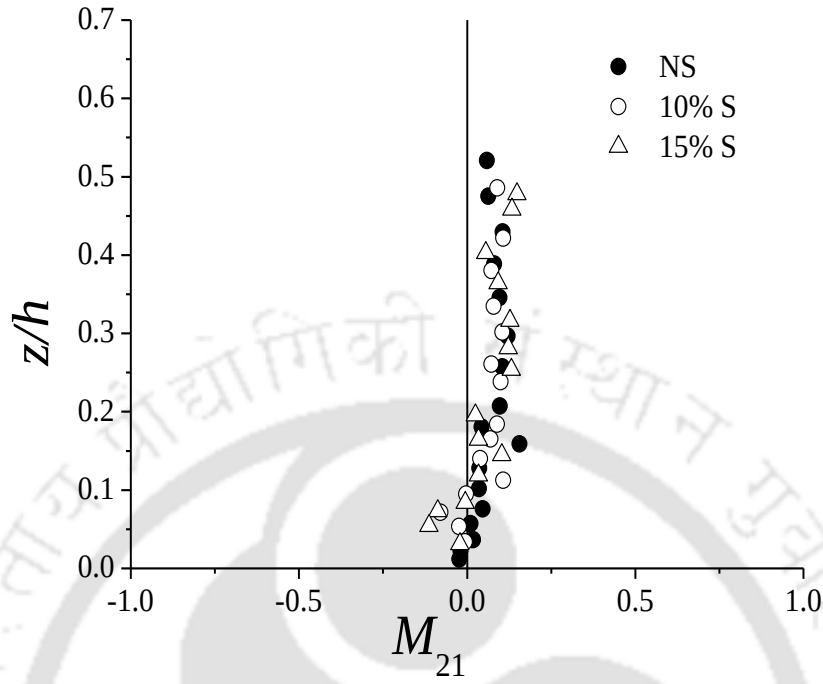


Figure 3.46: Vertical profiles of third order correlations M_{21} for no seepage and seepage flow

Vertical distribution of the difference in the fractional contributions towards Reynolds shear stress production from Q4 and Q2 events ($\Delta S_{i,0} = S_{4,0} - S_{2,0}$) are presented in Figure 3.47. It is observed from Figure 3.47 that, $\Delta S_{i,0}$ increases with seepage due to increase in roughness concentration indicating that dominance of the sweep events in contribution towards the production of Reynolds shear stresses increases with the application of seepage. **Raupach (1981)** found that the difference of the Reynolds shear stress $S_{i,H}$ between Q4 and Q2 events is proportional to M_{30} as $M_{30} = x\Delta S_{i,H} = x(S_{4,H} - S_{2,H})$ where x is the proportionality constant. Variation of M_{30} with $\Delta S_{i,0}$ has been plotted in Figure 3.48 and 3.49 for no seepage and seepage runs subjected to $H=0$ and $H=2$ respectively. The values of proportionality constant x are less than 2.70, which were obtained by **Raupach (1981)**. However the effect of seepage is prominent, as the values of x increases in the presence of downward seepage. It is also noticed that the value of x is decreased with the elimination of weaker events ($H=2$) for both no seepage and seepage flows.

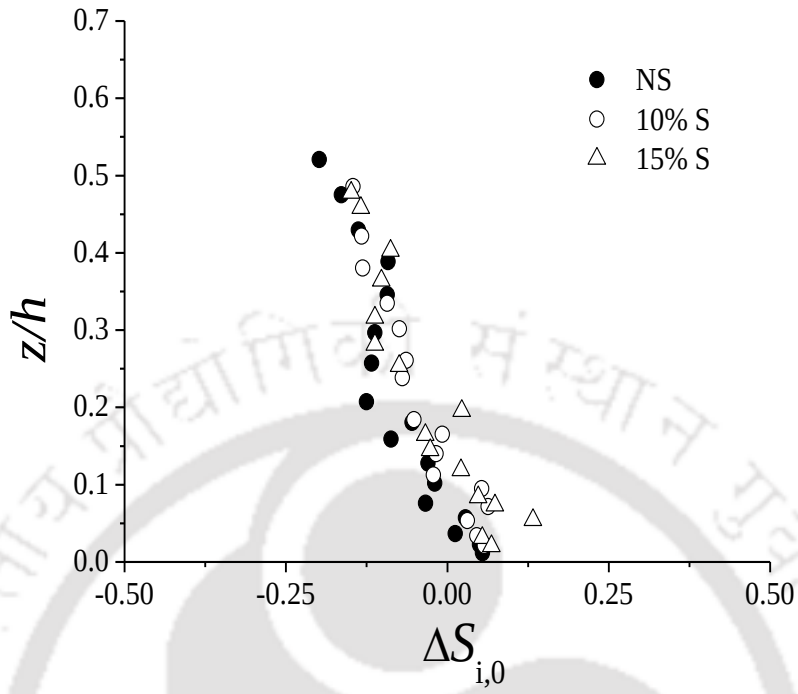


Figure 3.47: Vertical distributions of $\Delta S_{i,0}$ subjected to NS, 10%S and 15%S.

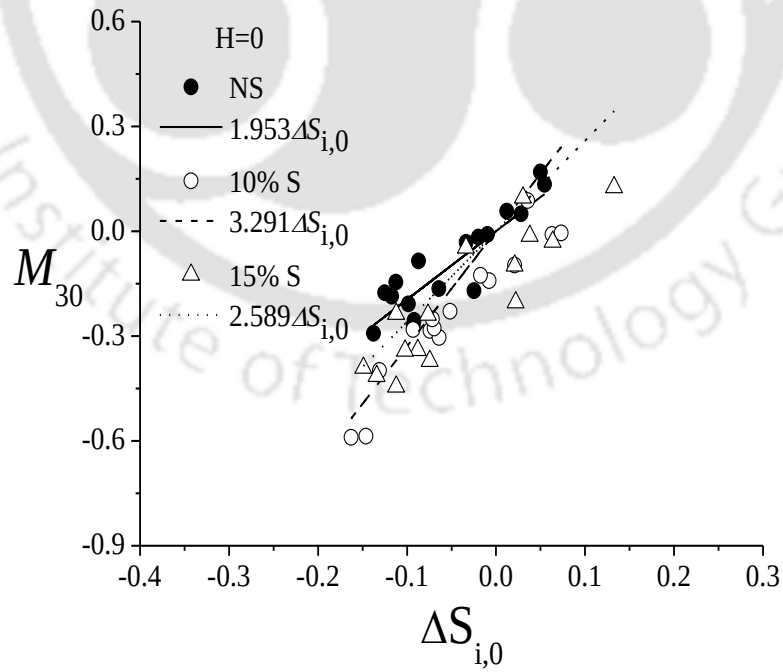


Figure 3.48: Relation between M_{30} and $\Delta S_{i,H}$ for $H=0$

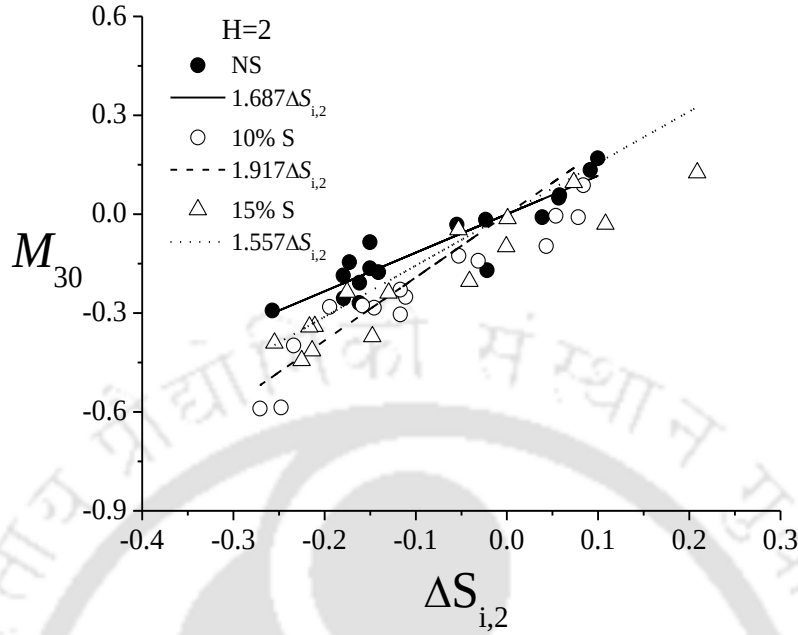


Figure 3.49: Relation between M_{30} and $\Delta S_{i,H}$ for $H=2$

Fourth order correlation known as kurtosis represents the intermittency of turbulence. Figure 3.50 shows the vertical profile of fourth order moment $K_u = \overline{u'^4} / \sigma_u^4$ in the stream wise direction for no seepage and seepage run. It is observed that kurtosis becomes minimum at $z/h = 0.13$ after which the value is increased in both inner and outer layer of turbulent flow which was also observed by **Andreopoulos et al. (1984)**. It suggests that turbulence is highly intermittent in both layers which are confirmed with **Grass (1971)** that sweep and ejection provides strong intermittent contribution to the turbulence production. Noticeably, $K_u > 3$ in flow layer refers to a distribution with a peaky signal characteristic of intermittent turbulent events while $K_u < 3$ refers to a flat characteristic. However the position of occurrence of minimum kurtosis is increased with the percentage of seepage which is $z/h = 0.14$ for 10%S and $z/h = 0.16$ for 15%S. Further kurtosis is increased in the near bed surface with seepage which represents higher level of turbulence intermittency is observed with seepage as compared to that with no seepage.

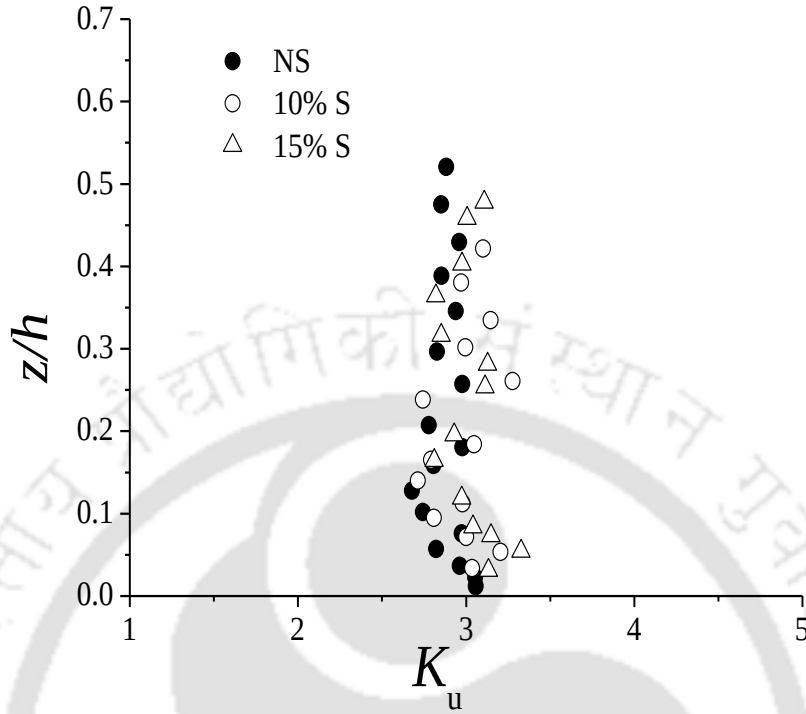


Figure 3.50: Vertical profiles of kurtoses for no seepage and seepage flow

3.10 Turbulent Kinetic Energy and Energy Budget

In the two dimensional analysis, stream wise and vertical flux components of turbulent kinetic energy (TKE) are expressed as (**Raupach, 1981; Krogstad and Antonia, 1999**).

$$\left. \begin{aligned} f_{ku} &= 0.75 \left(\overline{u'^3} + \overline{u'w'^2} \right) \\ f_{kw} &= 0.75 \left(\overline{w'^3} + \overline{w'u'^2} \right) \end{aligned} \right\} \quad (3.17)$$

Flux of the TKE in the streamwise (f_{ku}) and vertical (f_{kw}) directions are non dimensionalized by the shear velocity (u_*)

$$\left. \begin{aligned} F_{ku} &= f_{ku} / u_*^3 \\ F_{kw} &= f_{kw} / u_*^3 \end{aligned} \right\} \quad (3.18)$$

The profiles of stream wise (F_{ku}) and vertical (F_{kw}) flux of the normalized TKE for no seepage and seepage runs are presented in Figure 3.51 and 3.52 respectively. The TKE transport takes place from the zone of energy source to the energy sink through a transitional zone in order to maintain a kinematic equilibrium. In NS runs, F_{ku} is positive near the boundary and changing to negative value for $z/h > 0.057$ while F_{kw} is negative near the bed and changing to positive value for $z/h > 0.036$. This implies that at near bed flow zone, positive F_{ku} represent the stream wise TKE flux is transported in downstream direction while, negative F_{kw} denotes the vertical TKE flux is in the downward transport. Moving away from the bed, negative values of F_{ku} indicate the stream wise TKE flux is transported in upstream direction and positive values of F_{kw} suggest the transport of vertical flux of TKE are in the upward direction. Thus within the near bed region, the positive values of F_{ku} and negative values of F_{kw} imply that the TKE flux is transported downstream with downward direction while away from the bed region, the negative values of F_{ku} and positive values of F_{kw} imply that the TKE flux is transported upstream with upward direction. However, F_{ku} and F_{kw} for seepage runs have almost similar nature of trend than those of no seepage flow. Therefore, greater positive value of F_{ku} is achieved in near bed region and greater negative value for the rest of the depth with seepage than that of no seepage runs.

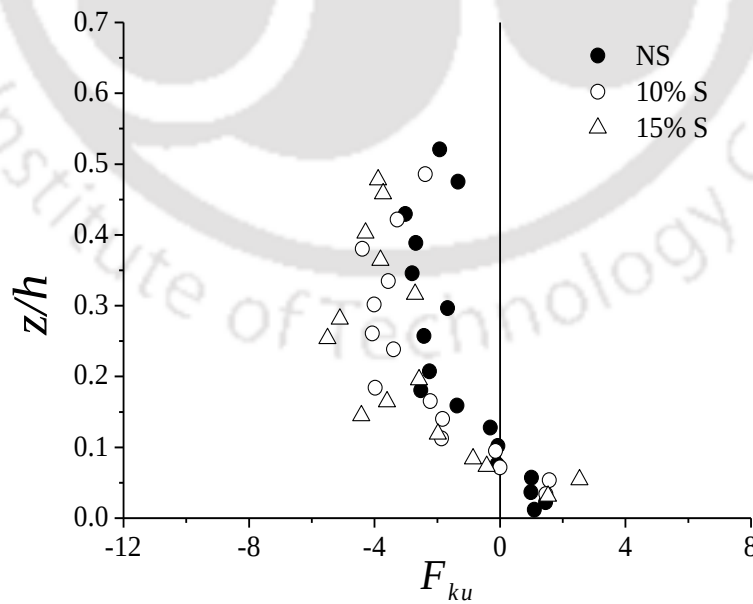


Figure 3.51: Vertical distributions of TKE Flux components (F_{ku})

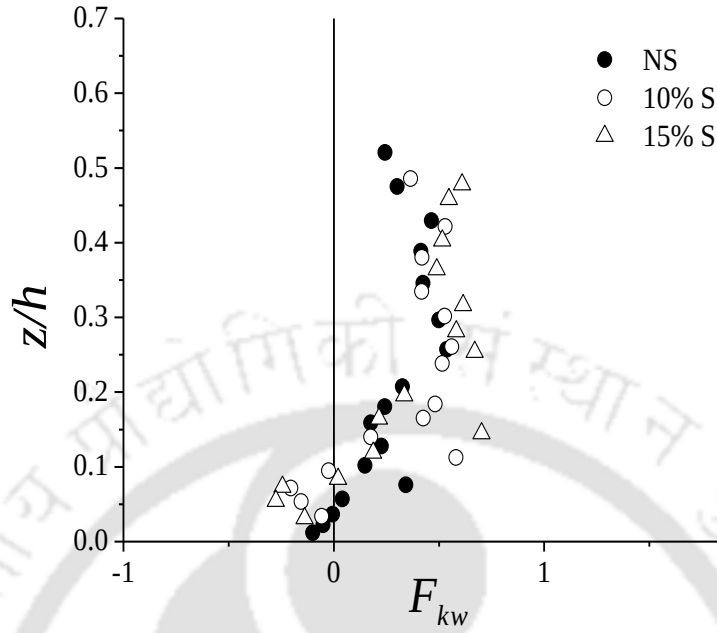


Figure 3.52: Vertical distributions of TKE Flux components (F_{kw})

In two dimensional flow field, the turbulent energy budget comprises the turbulent production ($t_p = -\overline{u'w'} \partial u / \partial z$) which is equal to the summation of turbulent dissipation (ϵ), turbulent energy diffusion ($t_D = \partial f_{kw} / \partial z$), Pressure energy diffusion ($p_D = \partial \overline{p'w'} / \rho_w \partial z$) and viscous diffusion ($v_D = \nu \partial^2 K / \partial z^2$); Where ν is kinematic viscosity, p' is the pressure fluctuations and K is the TKE (Nezu and Nakagawa, 1993). The viscous diffusion v_D contributes insignificantly in turbulent flow over the rough boundary. The turbulent dissipation ϵ is evaluated by two methods as follows (Irwin, 1973; Krogstadt and Antonia, 1999):

$$\epsilon = \frac{2\pi}{u} \left[\frac{\overline{u'^2} J^{5/3} \theta(J)}{C_K} \right] \quad (3.19)$$

$$\epsilon = \frac{15\nu}{u^2} \left(\frac{\partial u'}{\partial t} \right)^2 \quad (3.20)$$

Where $\theta(J)$ is non-dimensional spectral density of u' such that, $\int_0^{\infty} \theta(J) dJ = 1$, J is wave number

in the stream wise direction; and C_K is non-dimensional Kolmogorov constant =0.52. Sampling tests predict that Equation (3.19) yielded consistently higher values of ε that was incomparable to the turbulent production. Thus, the value of ε measured from Equation (3.20) was used due to their consistency, although the coefficient 15 in the right hand side of Equation (3.20) denotes the isotropic turbulence. In the present work, the unchanged value of the coefficient is used because of non-availability of the revised value of coefficient in open channel flows although it may give slightly higher value for ε . However, **Irwin (1973) and Krogstadt and Antonia (1999)** observed Equation (3.20) a good predictor for ε . The pressure energy diffusion rate p_D is calculated from unknown of energy budget equation as $p_D = t_p - \varepsilon - t_D$. The parameters of TKE budget are normalised in the form as $T_p, E_D, T_D, P_D = (t_p, \varepsilon, t_D, p_D) \times (h/u_*^3)$. Figures 3.53-3.56 presented the TKE budget parameters in flows subjected to no seepage and seepage run. It is observed that T_p and E_D decrease rapidly with an increase in z/h becoming nearly constant (with small magnitude) for near flow surface region. Positive T_p and negative T_p specifies the conversion of TKE from the time mean flow to turbulence and vice versa. T_p is increased mainly in the near bed region with seepage run than those of no seepage, suggesting the increased level of turbulence production with seepage. This increased level of turbulence is related with the increased turbulence intensities and increased momentum transfer through the non-uniform sand bed in the form of seepage flow. Dissipation of turbulent kinetic energy E_D is reduced in magnitude with seepage because more energy is transferred to turbulent fluctuations; the gain in turbulent kinetic energy becomes higher. In seepage run, dissipation of TKE is less as compared to that with no seepage. The vertical profiles of E_D are similar to those of T_p with a lag. The lag gets smaller in downward seepage with respect to no seepage case. The turbulence diffusion T_D is reduced in the near bed region with respect to seepage run due to gain in turbulence production and subsequent reduction in the TKE dissipation and pressure energy diffusion rate with the application of downward seepage. Importantly, **Nezu and Nakagawa (1993)** reported that T_p nearly equals E_D and T_D is almost balanced by P_D in flows over smooth and rough beds. The

most important nature lies on the near bed distributions of P_D is that it is increased with seepage which suggest that rate of sediment transport is increased due to gain in turbulence production.

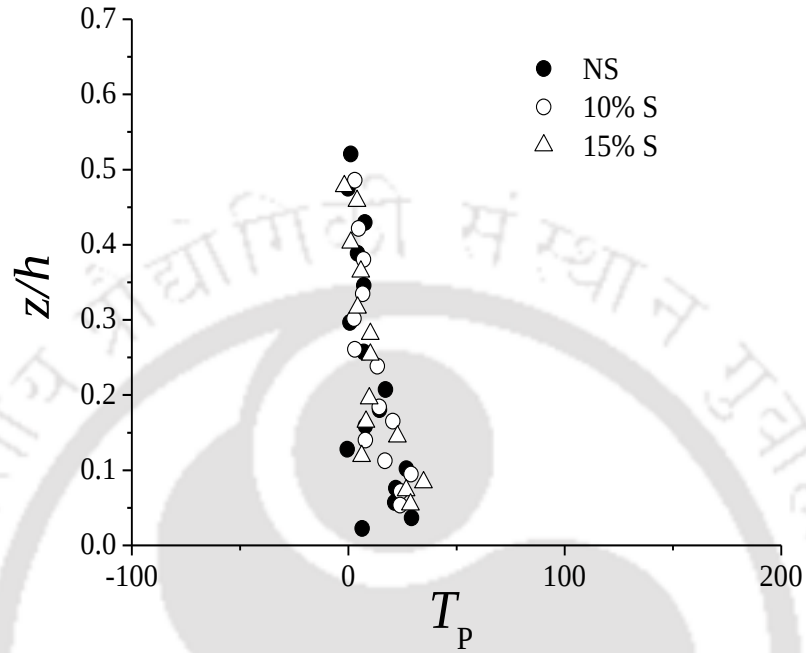


Figure 3.53: Vertical profiles of turbulent production for no seepage and seepage flow

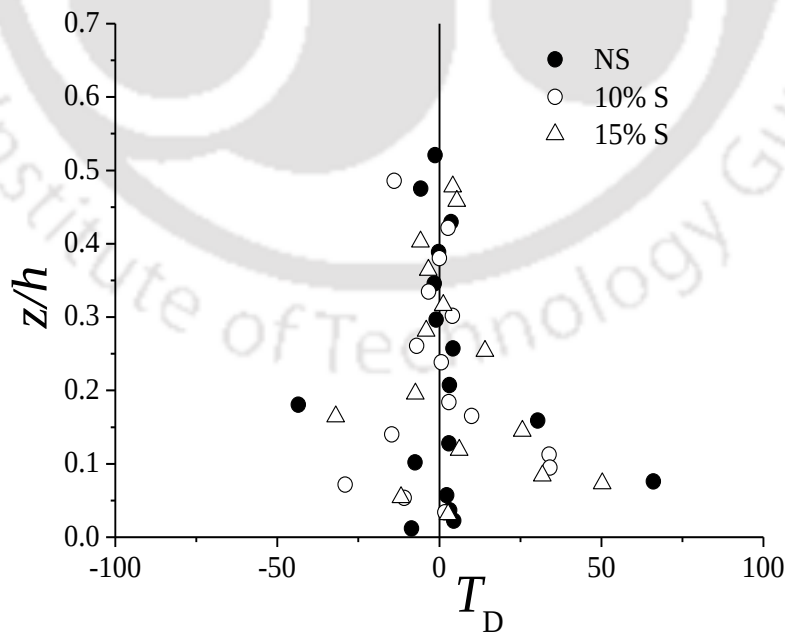


Figure 3.54: Vertical profiles of turbulent diffusion for no seepage and seepage flow

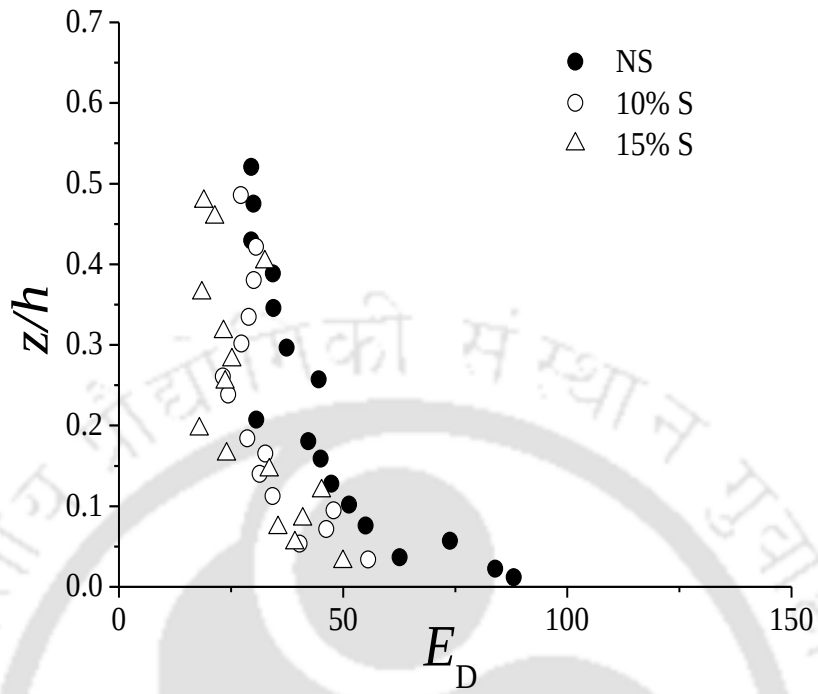


Figure 3.55: Vertical profiles of turbulent energy dissipation for no seepage and seepage flow

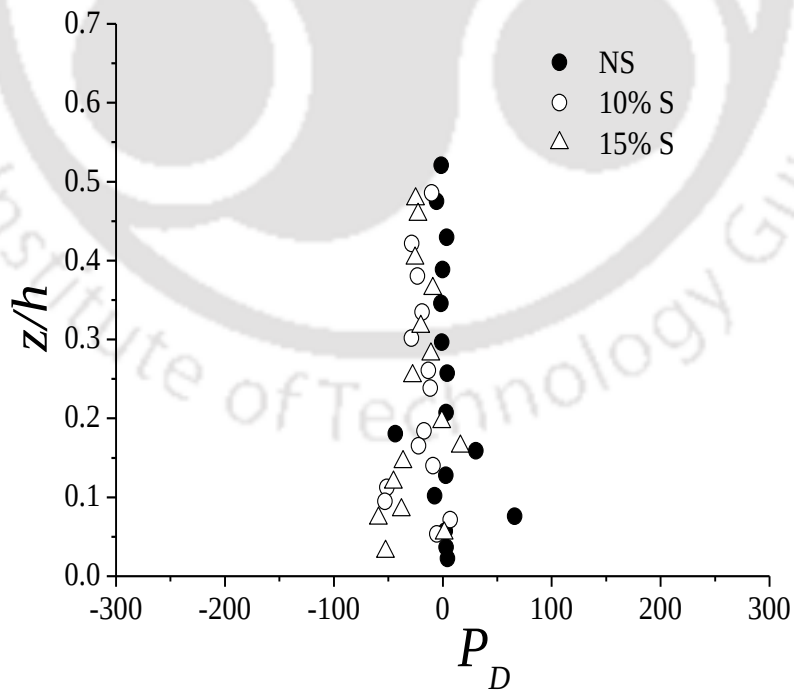


Figure 3.56: Vertical profiles of pressure energy diffusion for no seepage and seepage flow

3.11 Turbulence Scale

Kolmogorov's second similarity hypothesis (**Pope, 2000**) states that the statistics of motion of scale in turbulent flow at sufficiently high Reynolds number have a universal form that is uniquely determined by turbulent dissipation rate (ε) independent of kinematic viscosity (ν). Taylor micro scale λ_t defines eddy size in the inertial sub range and can be defined as

$$\lambda_T = \left(15\nu\sigma_u^2/\varepsilon\right)^{0.5} \quad (3.21)$$

Where, ε is the TKE dissipation rate calculated from the Equation (3.20). Figure 3.57 displays the normalised Taylor scale $\lambda_d = d_{50}/\lambda_T$ which is the variations of the ratio of sediment size to Taylor micro scale with respect to depth subjected to no seepage and seepage run. Throughout the flow depth, the value of λ_d is increased that is associated with the decrease in the turbulent kinetic energy dissipation rate (ε) and increase in turbulent intensity in stream wise direction with seepage flow as compared to that with no seepage. Also it is observed that λ_d decreases away from the bed for both no seepage and seepage case. At near the bed ($\frac{z}{h} = 0.036$), the values of λ_d is about 6.6% smaller with 10%S and about 11.40% smaller with 15%S, as compared to that with no seepage. This reduction of λ_d corresponds to the increase in λ_t with seepage indicating that eddy size is increased in the vicinity of the bed causing subsequent increase in bed particle mobility in seepage flow as compared to that with no seepage run.

On account of governance of turbulent sweep events in the near bed region directly related with coherent structures of the turbulent flow, initiation of bed features was formed (**Gyr and Schmid, 1989**). This bed features were developed by coherent structure which keep the bed in mobile condition (**Best, 1992**). Some of the researchers (**Venditti et al., 2005; Patel et al., 2015**) reported that the origin of bed features is correlated with integral scale of flow in near bed region. In the present experiments, rate of sediment transport is observed at the downstream end of the channel for both no seepage and seepage run. The time scale and integral length scale were calculated for the near bed velocity of the no seepage, 10% seepage and 15% seepage runs in order to describe the changes in rate of sediment transport after the application of seepage.

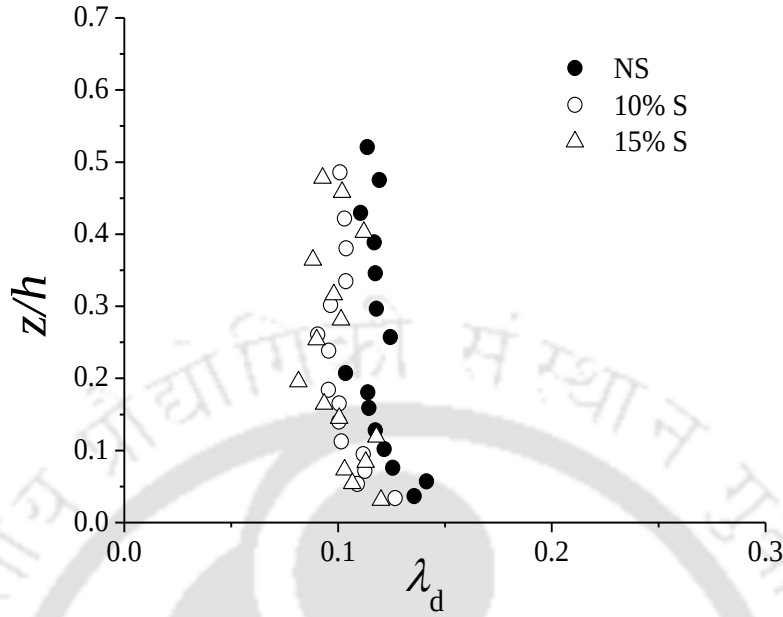


Figure 3.57: Ratio of particle size to Taylor micro scale (λ_d) as a function of flow depth

Integral time scale and length scale are measured at $\sim 4.0\text{mm}$ above the bed surface using the 5 minutes time series data collected at the same depth. An integral length scale represents the large eddy size in the flow and corresponding time scale indicates the eddy turnover time at a given point. The transfer of momentum and turbulent kinetic energy in the flow are linked with these eddy. The Eulerian integral time scale is expressed as

$$E_T = \int_0^T R(t) dt \quad (3.22)$$

Where, $R(t)$ is the autocorrelation function, dt is the lag distance, and T is the time at which $R(t)$ nearly reaches to zero (**Tennekes and Lumley, 1972**). Autocorrelation was measured using linear interpolation in order to convert the time series into regularly spaced events. To minimize the error in the estimation of turbulence integral scales, instantaneous velocities of around 30000 samples were collected in 5 minutes. Value of T was estimated based on autocorrelation at which $R(t) \approx 0.01$ and lag time is 0.01 s. The Eulerian integral length scale (**Taylor, 1935**) is expressed as

$$E_L = E_T u. \quad (3.23)$$

Value of E_T and E_L are shown in Table 3.3 which shows that eddy length and eddy turnover time close to the bed surface are increased significantly with downward seepage. Increased eddy size in the near-bed surface tends to higher energy and momentum transfer and lower destruction of turbulent motions. Thus higher levels of turbulence is achieved near the bed with an increased eddy size, which results in higher rate of sediment transport with downward seepage.

Table 3.3: Integral length scale and time scale at 4.0 mm above the bed surface

Case	Eulerian time scale E_T (s)	Eulerian length scale E_L (m)
NS	0.343	0.0768
10%	0.354	0.0888
15%S	0.367	0.0924

3.12 Probability Distribution Function of Turbulence Parameters

3.12.1 Overview

Probability density function of a random variable is a function that defines the relative likelihood for this random variable to deal with a given value. Probability distribution function can be represented by Gaussian distribution, Log normal distribution and many other etc. These distributions are based on the interval over which the convergence is finite. The advantage of Gram Charlier (GC) series to other series is that it is applicable to series containing interval over which the application of convergence is infinite (**Milne, 1929**). Also, PDFs of GC series is developed considering the higher order moments up to four (**Cramer, 1999**).

The knowledge of Probability density function (PDF) of turbulent statistics such as velocity fluctuation and Reynolds shear stress helps in understanding the related turbulent bursting events (**Nakagawa and Nezu, 1977; Afzal et al., 2009**). Considering correlations at two points separated in time, **Van Atta and Chen (1968)** predicted that correlations of even- order can be achieved by assuming a Gaussian based joint PDF while the Gaussian based Gram Charlier (GC)

PDF is followed by odd-order correlations. Considering non-Gaussian nature of velocity fluctuation in stream wise and vertical direction, **Antonia and Atkinson (1973)** followed GC series expansion for the study of joint PDFs of velocity fluctuation achieved by inverting a Gaussian based characteristics function. **Nakagawa and Nezu (1977)** used GC based joint PDF in the form given by **Feriet and Kampe (1966)**. The PDFs can be used to compute the change of turbulent flows extending from shear layer of incompressible turbulent flows (**Pope, 1985; Haworth and Pope, 1987**) to turbulence reacting flows with high density fluctuations and also in case of finite rate chemistry (**Pope and Correa, 1986; Chen et al., 1989**). The statistical characteristics of any signal observed in turbulence for experimental conditions are highly reproducible despite being of the stochastic nature in open channel flow (**Andreotti and Douady, 1999**). These basic remarks have induced researchers to search for a probabilistic explanation of turbulence. Thus, PDFs of various turbulent quantities such as velocity fluctuations, Reynolds shear stress and bursting events should be measured by experimentally as well as theoretically. **Bose and Dey (2010)** gave the theoretical expression for the PDFs of velocity fluctuation and Reynolds shear stress based on GC series exponential distribution and also reflecting skewness and kurtosis in the distribution. Recently, **Sharma and Kumar (2016)** experimentally investigated the PDFs of turbulence parameters in curvilinear cross section mobile bed channel and observed that PDF obtained with the experimental data are in good agreement with the theoretical curves based on GC series exponential distribution.

The study discussed in the previous research concerns analysing probability distribution function in mobile or immobile bed condition. Previous works neglected the factor of downward seepage which contributes a significant role in influencing the main flow characteristic in an alluvial channel. Considering the importance of seepage, the present study experimentally investigates PDFs of turbulent parameters such as velocity, Reynolds shear stress, bursting events for both no seepage and seepage runs and compares with the theoretical equation given by **Bose and Dey (2010)**. Also the present study develops the theoretical expression for the PDFs of turbulent intensity which is not available in the existing literature.

3.12.2 Probability Functions for Velocities Fluctuations

Velocity fluctuation (u' , w') follows the Gram Charlier (GC) series based on exponential distribution at which u' and w' is normalized as $\hat{u} = u'/\sigma_u$ and $\hat{w} = w'/\sigma_w$ where, σ_u and σ_w are the stream wise and vertical turbulence intensity respectively (**Sharma and Kumar, 2016**). The probability density function (PDF) for stream wise and vertical velocity fluctuation is expressed in the following form of equations (**Bose and Dey, 2010; Nezu, 1977**).

$$P_{\hat{u}}(\hat{u}) = \left[\frac{1}{2} + \frac{1}{4} C_{10} \hat{u} - \frac{1}{16} C_{20} (1 + |\hat{u}| - \hat{u}^2) - \frac{1}{96} C_{30} \hat{u} (3 + 3|\hat{u}| - \hat{u}^2) + \frac{1}{768} C_{40} (9 + 9|\hat{u}| - 3\hat{u}^2 - 6|\hat{u}|^3 + \hat{u}^4) + \dots \right] \exp(-|\hat{u}|) \quad (3.24)$$

$$P_{\hat{w}}(\hat{w}) = \left[\frac{1}{2} + \frac{1}{4} C_{01} \hat{w} - \frac{1}{16} C_{02} (1 + |\hat{w}| - \hat{w}^2) - \frac{1}{96} C_{03} \hat{w} (3 + 3|\hat{w}| - \hat{w}^2) + \frac{1}{768} C_{04} (9 + 9|\hat{w}| - 3\hat{w}^2 - 6|\hat{w}|^3 + \hat{w}^4) + \dots \right] \exp(-|\hat{w}|) \quad (3.25)$$

$$\text{Where, } C_{10} = m_{10}, C_{20} = \frac{1}{2} m_{20} - 1, C_{30} = \frac{1}{6} m_{30} - 2m_{10}, C_{40} = \frac{1}{24} m_{40} - \frac{3}{2} m_{20} + 2 \quad (3.26)$$

$$C_{01} = m_{01}, C_{02} = \frac{1}{2} m_{02} - 1, C_{03} = \frac{1}{6} m_{03} - 2m_{01}, C_{04} = \frac{1}{24} m_{04} - \frac{3}{2} m_{02} + 2 \quad (3.27)$$

$$\text{Where, } m_{j0} = \int_{-\alpha}^{\alpha} \hat{u}^j P_{\hat{u}}(\hat{u}) d\hat{u}, m_{0k} = \int_{-\alpha}^{\alpha} \hat{w}^k P_{\hat{w}}(\hat{w}) d\hat{w}$$

The experimental data are used to determine the coefficient C_{j0} and C_{0k} mentioned in Equations (3.26) and (3.27). The relative frequencies $f_{\hat{u}}(\hat{u})$ and $f_{\hat{w}}(\hat{w})$ of the random variables $\hat{u}$ and $\hat{w}$ are calculated from the experimental data at a purpose flow depth z to estimate the theoretical curves for PDFs of $P_{\hat{u}}(\hat{u})$ and $P_{\hat{w}}(\hat{w})$. In order to calculate the moment m_{j0} and m_{0k} , the PDF $P_{\hat{u}}(\hat{u})$ and $P_{\hat{w}}(\hat{w})$ is approximated by $f_{\hat{u}}(\hat{u})$ and $f_{\hat{w}}(\hat{w})$ respectively and the composite Simpson's rule are used to evaluate the integrals. In this manner, Equation (3.25) and (3.26) are used to estimate the C_{j0} and C_{0k} and the PDFs are computed to produce the theoretical curves for $P_{\hat{u}}(\hat{u})$ and $P_{\hat{w}}(\hat{w})$.

The coefficient C_{j0} and C_{0k} are tabulated in Table 3.4. Because of inhomogeneity and anisotropic nature of turbulent flows, it is difficult to model for such flows in the near bed region. But the effect of seepage is prevalent in the vicinity of bed as mentioned in the literature, a measurement location near the bed ($z=0.03h$) and away from the bed ($z=0.4h$) is selected for the study of PDFs $P_u(\hat{u})$ and $P_w(\hat{w})$. Figures 3.58 and 3.59 show $P_u(\hat{u})$ and $P_w(\hat{w})$ distributions at the selected depths of flow for both no seepage and seepage run respectively. The experimental data trends are used to calculate $P_u(\hat{u})$ and $P_w(\hat{w})$ for both no seepage and seepage cases. In the Figures, the value of Jensen-Shannon divergence (JSD) measures the similarity between theoretical and experimental probability distributions (**Lin, 1991**). As, $JSD \rightarrow 0$, divergence of the observed value from the computed value is small. It is found that experimentally calculated $P_u(\hat{u})$ and $P_w(\hat{w})$ collapses reasonably ($JSD=0.0005-0.003$) on the theoretical curve which suggests that $P_u(\hat{u})$ and $P_w(\hat{w})$ can be signified by GC series expansion depending on exponential distribution. The value of JSD for similarity measure obtained in the present work is of same order as obtained in the previous study (**Grosse et al., 2002**). Previously **Bose and Dey (2010)** used Kullback–Leibler (KL) divergence measure and provided the measure in the order of 0.01. JSD is the extension of KL (**Lin, 1991**) and generally the value of JSD is of lower magnitude than KL. Thus, the present magnitude of similarity measures is satisfactory. The experimentally calculated $P_u(\hat{u})$ and $P_w(\hat{w})$ distributions do not achieve a sharp peak at zero velocity fluctuations because of the absence of relative frequencies of such narrow ranges in the histogram which is observed in case of theoretically estimated PDFs distributions. The results conclude that PDFs distributions for velocity fluctuations based on GC series exponential distributions follows the universality that it is applicable for no seepage as well as seepage cases. Close examination of Figures 3.58 and 3.59 indicates that probability distributions of velocity fluctuation are symmetrical about the origin and triangular in shape. The degree of discrepancy is achieved near the bed ($z=0.03h$) and outer flow zone ($z=0.4h$) for both no seepage and seepage flows. At near the bed, discrepancy occurs may be due to the presence of roughness sub layer while at away from the bed, discrepancy occurs may be due to the presence of large eddies. Figure 3.58 and 3.59 also indicates that degree of discrepancy is increased in both inner layer and outer layer of flow under the application of downward seepage which may be due to increase of roughness sub layer in near bed and increase of turbulent eddies in outer layer.

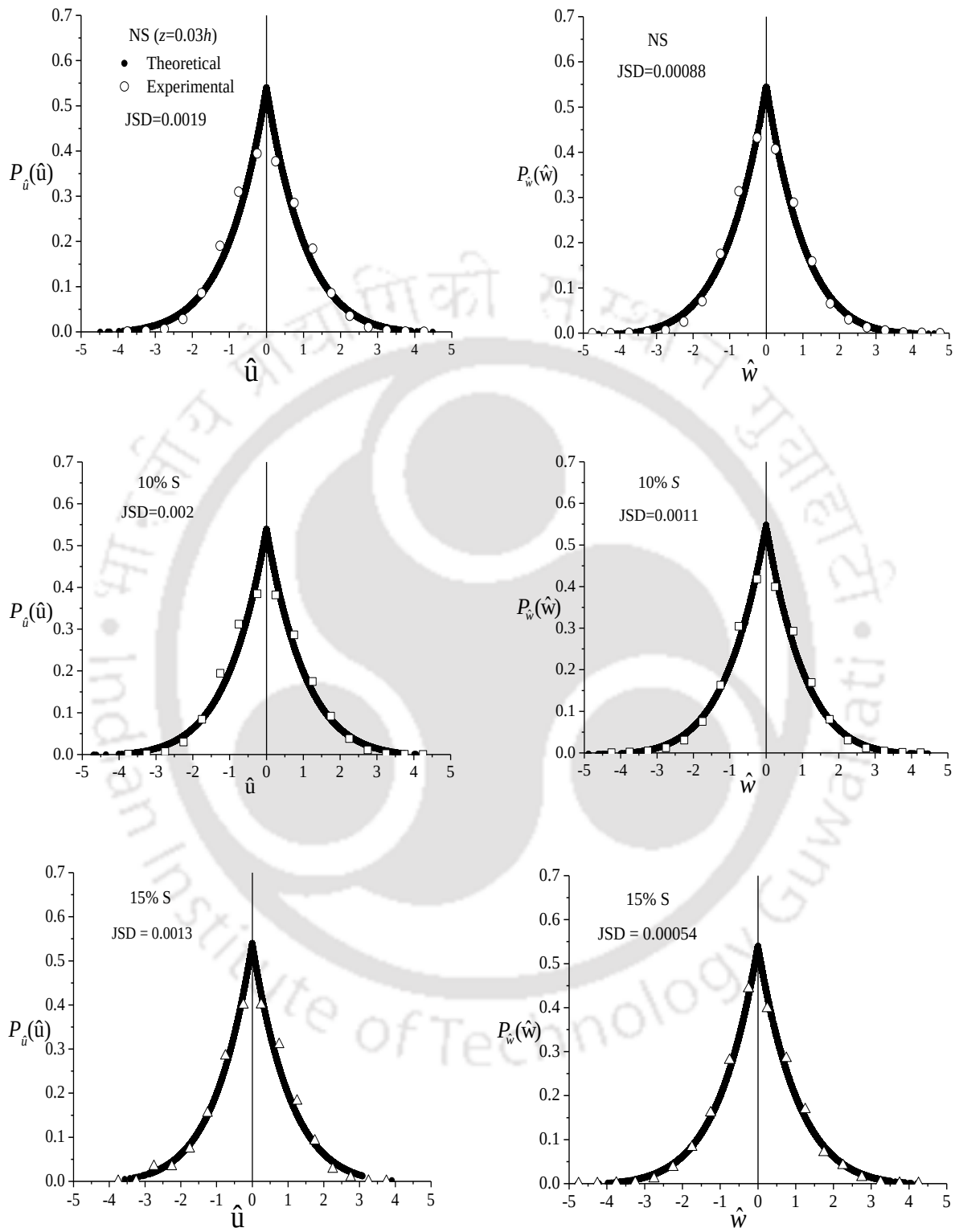


Figure 3.58: Comparisons of computed $P_{\hat{u}}(\hat{u})$ and $P_{\hat{w}}(\hat{w})$ with experimental data at $z=0.03h$

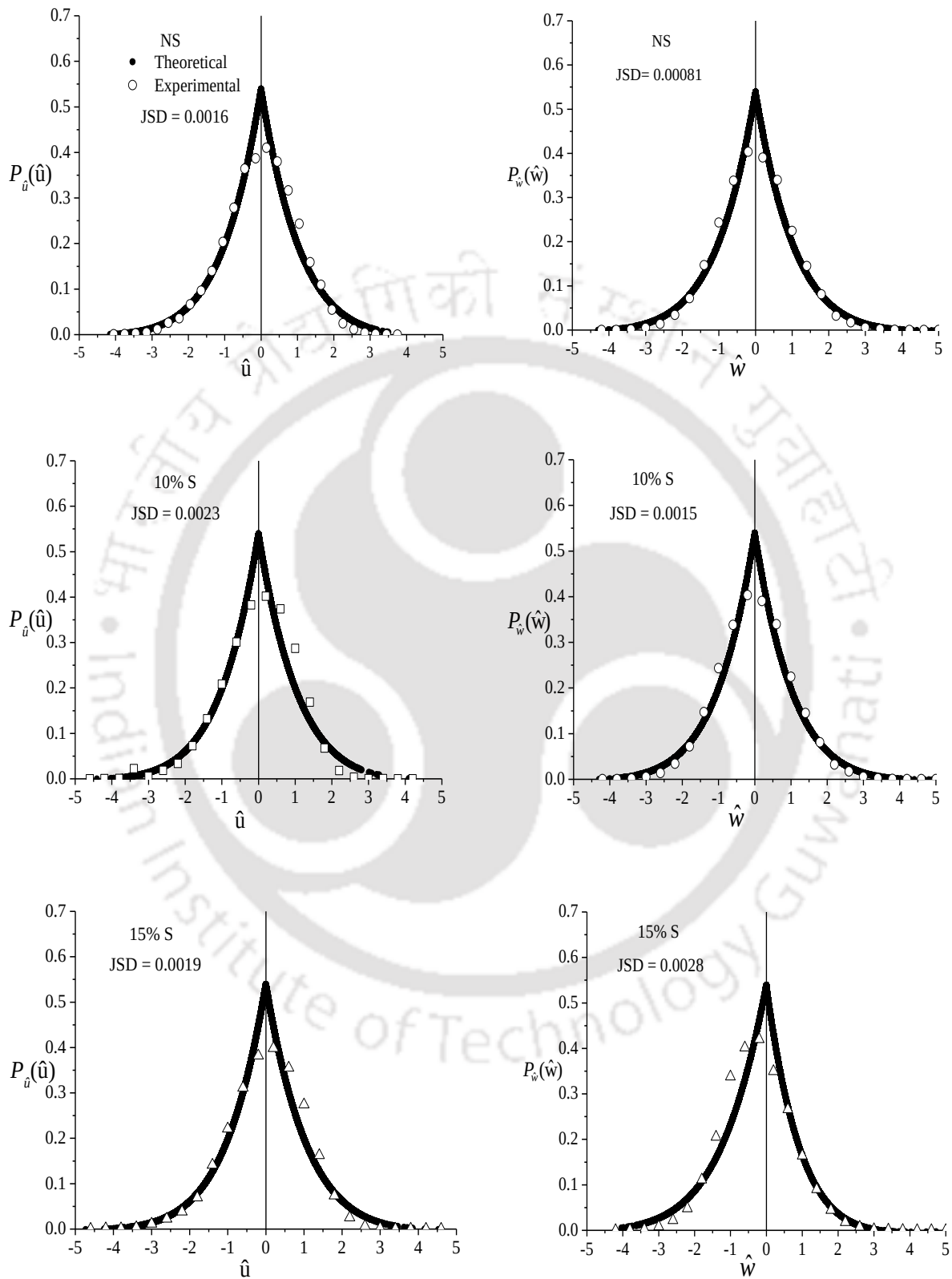


Figure 3.59: Comparisons of computed $P_u(\hat{u})$ and $P_w(\hat{w})$ with experimental data at $z=0.4h$

Table 3.4: Coefficient for the computation of $P_{\hat{u}}(\hat{u})$ and $P_{\hat{w}}(\hat{w})$

Case study	Flow condition	C_{10}	C_{01}	C_{20}	C_{02}	C_{30}	C_{03}	C_{40}	C_{04}
$z=0.03h$	NS	-0.00024	-0.00013	-0.5	-0.5	-0.0017	0.2	0.663	1.349
	10%S	0.000121	-0.0021	-0.5	-0.5	0.0124	-0.005	0.625	0.655
	15%S	6.91E-05	0.00121	-0.5	-0.5	-0.0380	0.003	0.627	0.637
$z=0.4h$	NS	-0.00033	-0.00142	-0.5	-0.5	-0.0370	0.016	0.631	0.638
	10%S	-0.00065	-0.00209	-0.5	-0.5	-0.0844	0.029	0.646	0.642
	15%S	0.000186	-0.2737	-0.5	-0.5	-0.0546	0.436	0.636	0.629

3.12.3 Probability Functions for Reynolds Shear Stress

The fluctuation of stream wise and vertical velocity u' and w' are the random quantities in open channel flow whose mean value of the product $u'w'$ yields the Reynolds shear stress $\tau = -\rho \overline{u'w'}$ due to which the PDFs of τ depends upon the joint PDFs of u' and w' . Dimensionless random variable $\hat{\tau} = \hat{u}\hat{w}$ is considered in order to study the PDFs of τ . **Nezu (1977)** derived the PDFs of $\hat{\tau}$ as

$$\begin{aligned}
 P_{\hat{\tau}}(\hat{\tau}) = & K_0(2\tau_1) - \frac{1}{8}(C_{20} + C_{02})(1 - \tau_1^2)K_0(2\tau_1) \\
 & + \frac{1}{4}C_{11}\hat{\tau}K_0(2\tau_1) + \frac{1}{64}C_{22}\left[(1 - \tau_1^2 + \tau_1^4)K_0(2\tau_1) - 2\tau_1^3K_1(2\tau_1)\right] \\
 & - \frac{1}{96}(C_{31} + C_{13})\hat{\tau}(3 - \tau_1^3)\left[K_0(2\tau_1) + \tau_1K_1(2\tau_1)\right] \\
 & + \frac{1}{384}(C_{40} + C_{04})\left[(9 - 9\tau_1^2 + \tau_1^4)K_0(2\tau_1) - 2\tau_1^3K_1(2\tau_1)\right] + \dots
 \end{aligned} \tag{3.28}$$

Where $\tau_1 = (|\hat{\tau}|)^{0.5}$, K_0 and K_1 are the modified Bessel function of the second kind and orders 0 and 1 respectively. The coefficient in the above expression can be found out by the following moments

$$C_{11} = m_{11}, \quad C_{22} = \frac{1}{4}m_{22} - \frac{1}{2}(m_{20} + m_{02}) + 1, \quad C_{31} = \frac{1}{6}m_{31} - 2m_{11}, \quad C_{13} = \frac{1}{6}m_{13} - 2m_{11} \tag{3.29}$$

The theoretical curve for PDFs of $P_{\hat{\tau}}(\hat{\tau})$ is estimated by approximating the relative frequency $f_{\hat{\tau}}(\hat{\tau})$ of the random variables $\hat{u} \hat{w}$ which is estimated from the experimental data at a flow depth z . The required coefficient is calculated with the Equations (3.26), (3.27) and (3.29) to yield the theoretical expression for $P_{\hat{\tau}}(\hat{\tau})$ given by Equation (3.28). Figure 3.60 and 3.61 depicts computed and observed $P_{\hat{\tau}}(\hat{\tau})$ distributions at near the bed ($z=0.03h$) and away from the bed ($z=0.4h$) for both no seepage and seepage cases respectively. The values of coefficient are tabulated in Table 3.5. It is clear from the Figure 3.60 and 3.61 that experimentally calculated $P_{\hat{\tau}}(\hat{\tau})$ are in good agreement ($JSD=0.001-0.006$) with the theoretical curves which suggests that $P_{\hat{\tau}}(\hat{\tau})$ can be characterized by GC series expansion based on exponential distribution for both no seepage and seepage flows. The result agrees with **Bose and Dey (2010)** and **Sharma and Kumar (2016)**. Figure 3.60 and 3.61 shows that the PDFs of Reynolds shear stress are almost independent of flow depth and seepage discharge. It may be focused that $\hat{\tau} = 0$ has singularity at the origin for both no seepage and seepage flows.

Table 3.5: Coefficient for the computation of $P_{\hat{\tau}}(\hat{\tau})$

Case study	Flow condition	C_{11}	C_{13}	C_{31}	C_{22}
$z=0.03h$	NS	-0.238	0.432	0.235	0.495
	10%S	-0.260	0.386	0.400	0.295
	15%S	-0.339	0.500	0.510	0.329
$z=0.4h$	NS	-0.194	0.274	0.284	0.270
	10%S	-0.290	0.415	0.414	0.326
	15%S	-0.256	-0.391	0.399	0.263

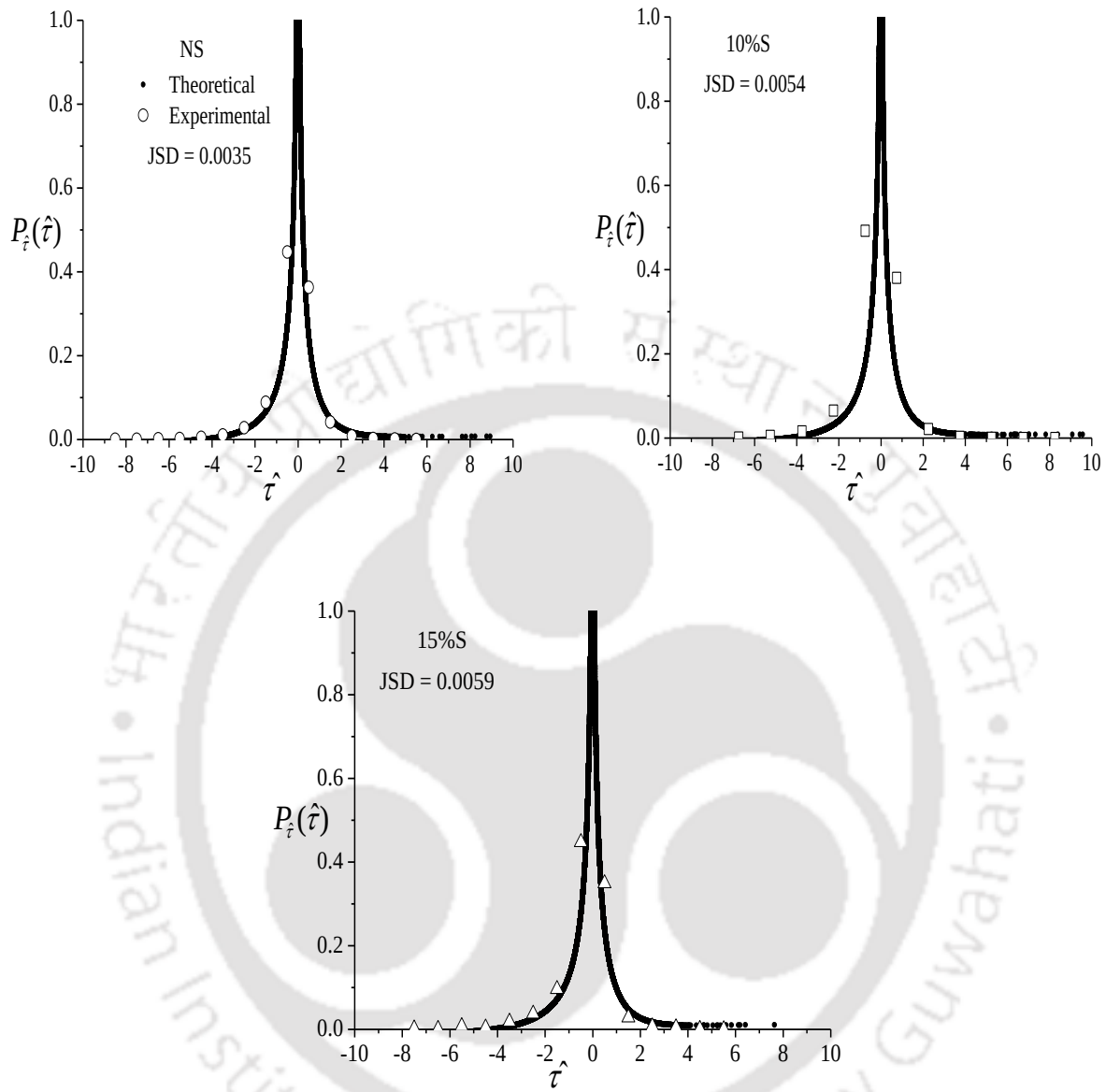


Figure 3.60: Comparisons of computed $P_i(\hat{\tau})$ with experimental data at $z=0.03h$

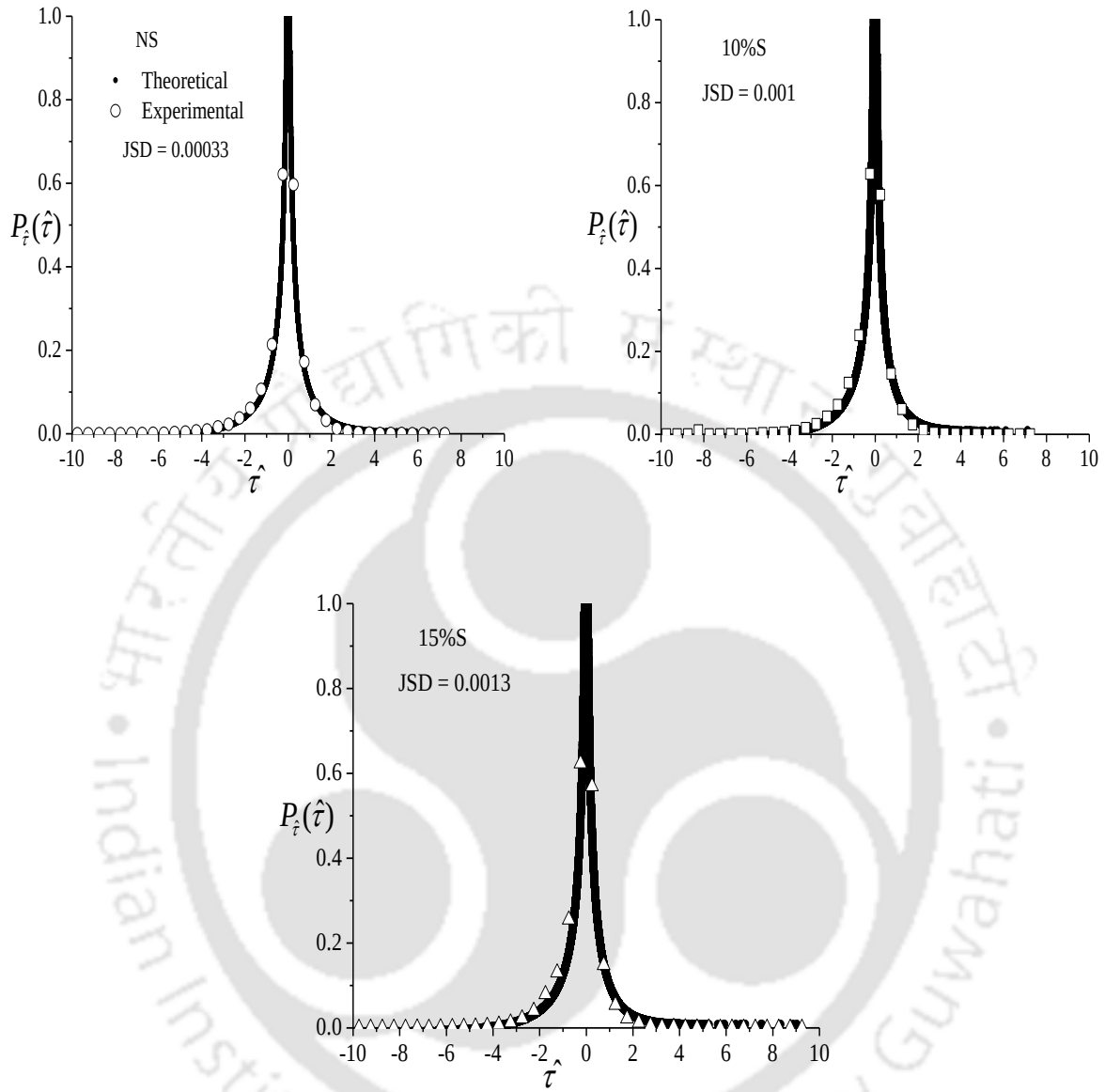


Figure 3.61: Comparisons of computed $P_i(\hat{\tau})$ with experimental data $z=0.4h$

3.12.4 Probability Functions for Bursting Events

The conditional nature of the velocity fluctuations u' and w' is useful to quantify the bursting events. To quantify the bursting events, it is thus necessary to plot u' and w' pairs on $u' w'$ plane (**Lu and Willmarth, 1973**). Following **Nezu and Nakagawa (1993)**, the bursting events are characterized by four quadrants such as outward interactions Q1 ($u' > 0, w' > 0$), ejections Q2 ($u' < 0, w' > 0$), inward interactions Q3 ($u' < 0, w' < 0$) and sweeps Q4 ($u' > 0, w' < 0$). The fractional

contribution from each bursting events to the production of total Reynolds shear stress is represented by the arbitrary variable $\hat{\tau} = \langle \hat{u}\hat{w} \rangle_{Q_i}$ that represents to the appropriate quadrant. The PDFs of events Q1, Q2, Q3 and Q4 are denoted by $P_1(\hat{\tau})$, $P_2(\hat{\tau})$, $P_3(\hat{\tau})$ and $P_4(\hat{\tau})$ respectively which follows that

$$P_1(\hat{\tau}) + P_2(\hat{\tau}) + P_3(\hat{\tau}) + P_4(\hat{\tau}) = P_{\hat{\tau}}(\hat{\tau}) \quad (3.30)$$

$$\begin{aligned} P_1(\hat{\tau} > 0) = P_4(\hat{\tau} < 0) = & \frac{1}{2} P_{\hat{\tau}}(\hat{\tau}) + \frac{1}{4} (C_{10} - C_{01}) \tau_1 K_1(2\tau_1) \\ & - \frac{1}{96} (C_{30} - C_{03}) \tau_1 \left[\tau_1 K_0(2\tau_1) + (4 - \tau_1) K_1(2\tau_1) \right] \\ & + \frac{1}{32} (C_{21} - C_{12}) \tau_1 \left[\tau_1 K_0(2\tau_1) + (1 - \tau_1^2) K_1(2\tau_1) \right] + \dots \end{aligned} \quad (3.31)$$

$$\begin{aligned} P_2(\hat{\tau} < 0) = P_3(\hat{\tau} > 0) = & \frac{1}{2} P_{\hat{\tau}}(\hat{\tau}) - \frac{1}{4} (C_{10} - C_{01}) \tau_1 K_1(2\tau_1) \\ & + \frac{1}{96} (C_{30} - C_{03}) \tau_1 \left[\tau_1 K_0(2\tau_1) + (4 - \tau_1) K_1(2\tau_1) \right] \\ & - \frac{1}{32} (C_{21} - C_{12}) \tau_1 \left[\tau_1 K_0(2\tau_1) + (1 - \tau_1^2) K_1(2\tau_1) \right] + \dots \end{aligned} \quad (3.32)$$

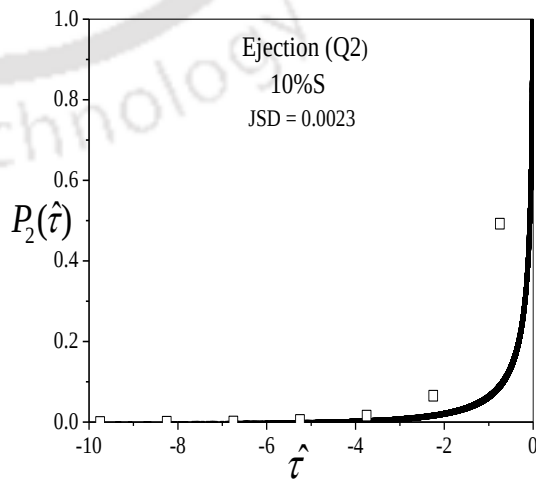
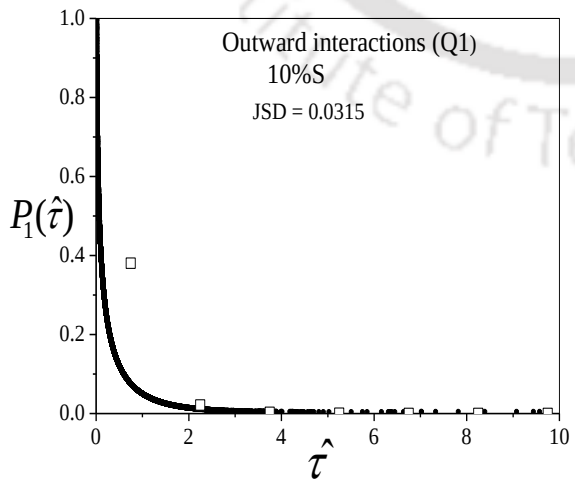
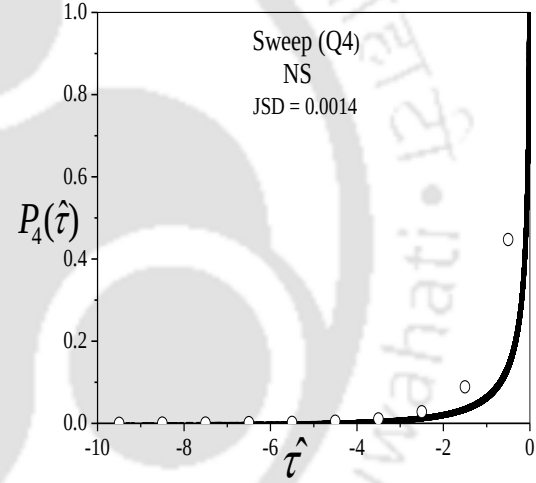
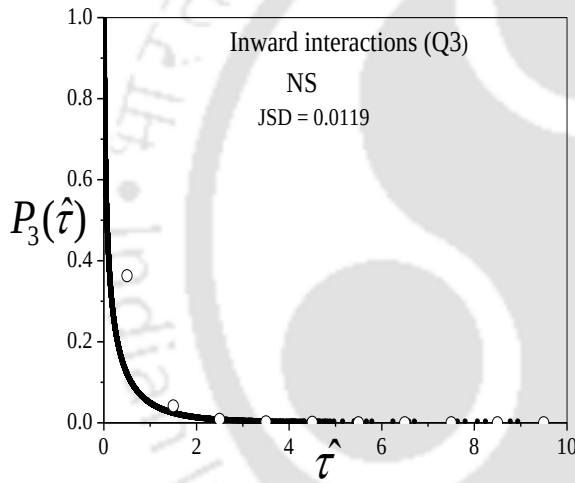
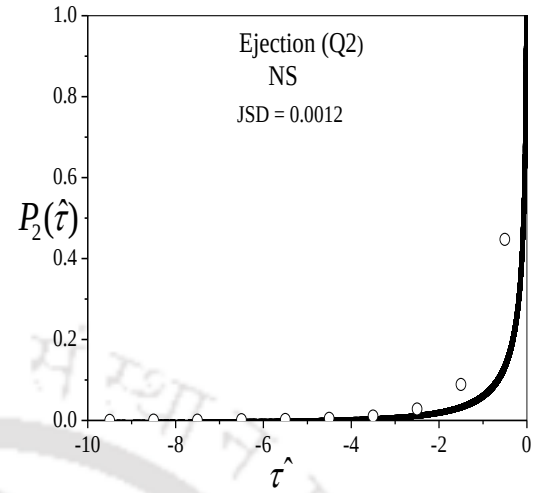
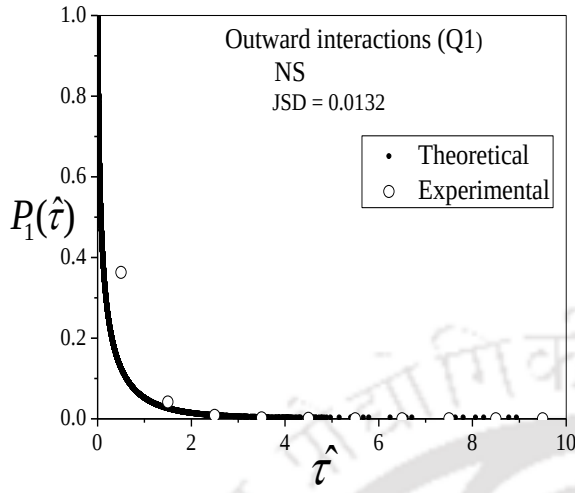
Nezu (1977) showed that the expression for $P_1(\hat{\tau})$ and $P_4(\hat{\tau})$ is the same as that given by Equation (3.31). For $\hat{\tau} < 0$, $P_1(\hat{\tau}) = 0$, the distribution of outward interaction Q1 is one sided and resting on the positive side while for $\hat{\tau} > 0$, $P_4(\hat{\tau}) = 0$, the distribution of sweeps Q4 is one sided and resting on the negative side. In the similar way, the expression for $P_2(\hat{\tau})$ and $P_3(\hat{\tau})$ is the same as that given by Equation (3.32). For $\hat{\tau} > 0$, $P_2(\hat{\tau}) = 0$, the distribution of Q2 is one sided and remaining on the negative side while for $\hat{\tau} < 0$, $P_3(\hat{\tau}) = 0$, the distribution of Q3 is one sided and resting on the positive side. The summation of expression $P_1(\hat{\tau})$, $P_2(\hat{\tau})$, $P_3(\hat{\tau})$ and $P_4(\hat{\tau})$ given by the above Equations (3.31) and (3.32) satisfy the Equation (3.30). The coefficient in the above expressions (3.31) and (3.32) are given by

$$C_{21} = \frac{1}{2} m_{21} - m_{01}, \quad C_{12} = \frac{1}{2} m_{12} - m_{10} \quad (3.33)$$

The experimental data are used to calculate the coefficient mentioned in Equation (3.33). The values of coefficient are presented in Table 3.6. Figure 3.62 and 3.63 compares the computed $P_i(\hat{\tau})$ with those measured for no seepage and seepage runs. The result states that even with the application of seepage, the bursting events corresponding to ejections (Q2) and sweeps (Q4) are well signified (JSD=0.0001-0.0025) by the exponential based Gram Charlier distribution while outward interaction (Q1) and inward interactions (Q3) are slightly deviated (JSD=0.0013-0.032) from computed distribution. This may be because of ejections and sweeps are the stronger events while inward and outward interactions are the feebler events so that both the weaker events may not closely follow the exponential based Gram Charlier distribution. Because of inhomogeneity and anisotropic nature of turbulent flows in the near bed region ($z=0.03h$), instantaneous Reynolds shear stress might have marked highly intermittency in that region due to which PDFs seem to under predict for low values of $\hat{\tau}$ in the near bed region while the case is slightly different in the outer flow region ($z=0.4h$) which is conformity with various researchers for no seepage runs (**Bose and Dey, 2010; Sharma and Kumar, 2016**). The PDFs of bursting events is used to predict the role of each event to the production of Reynolds shear stress. It was observed that the contribution of sweeps event is more important as compared to ejection events to the production of Reynolds stress near to the rough surface (**Nakagawa and Nezu, 1977**).

Table 3.6: Coefficient for the computation of $P_i(\hat{\tau})$

Case study	Flow condition	C_{21}	C_{12}
$z=0.03h$	NS	0.07580	0.000527
	10%	-0.01080	0.06000
	15%S	-0.00345	0.05800
$z=0.4h$	NS	0.01313	-0.03686
	10%	0.06433	-0.08782
	15%S	0.17410	0.056991



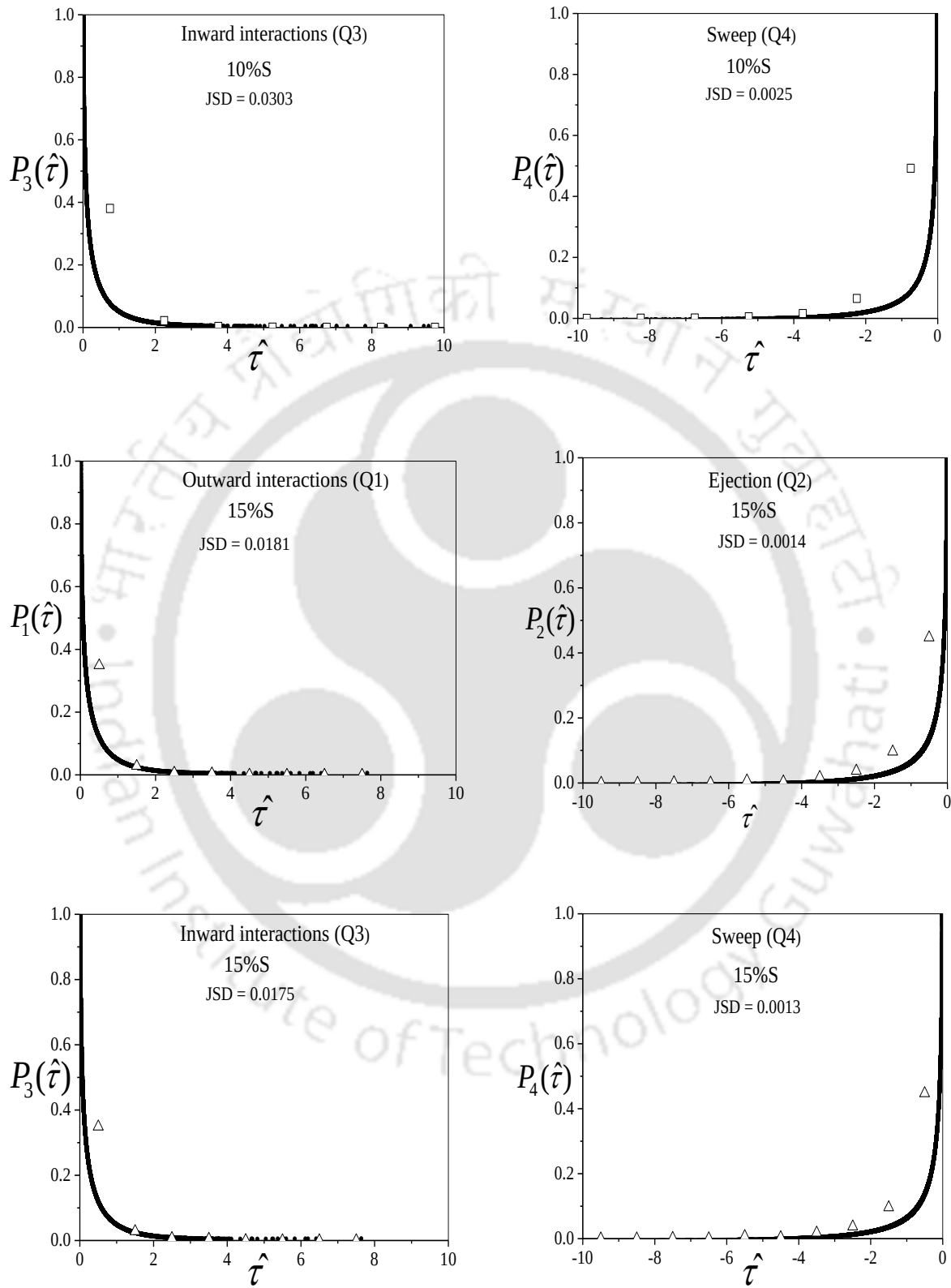
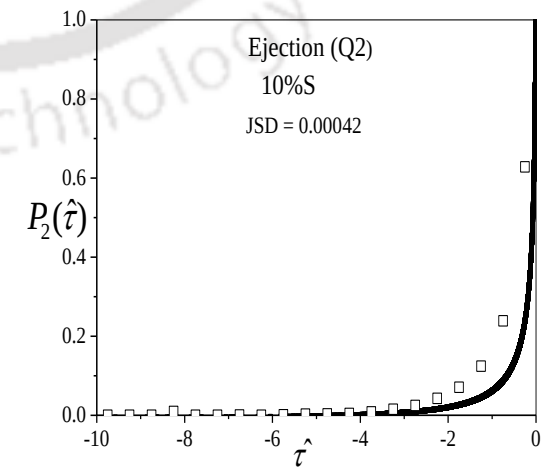
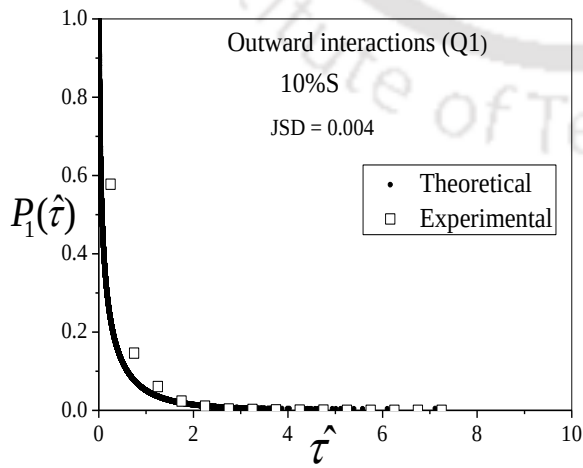
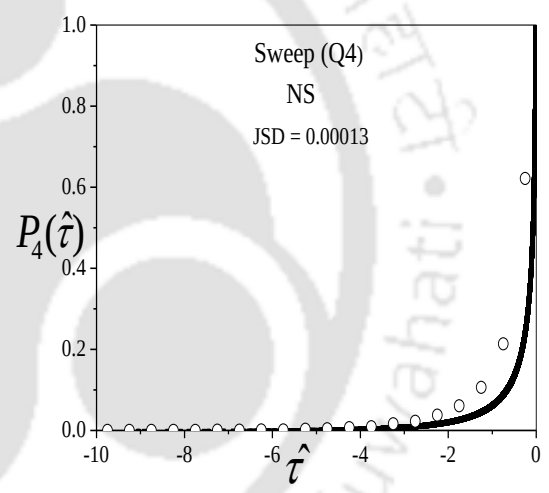
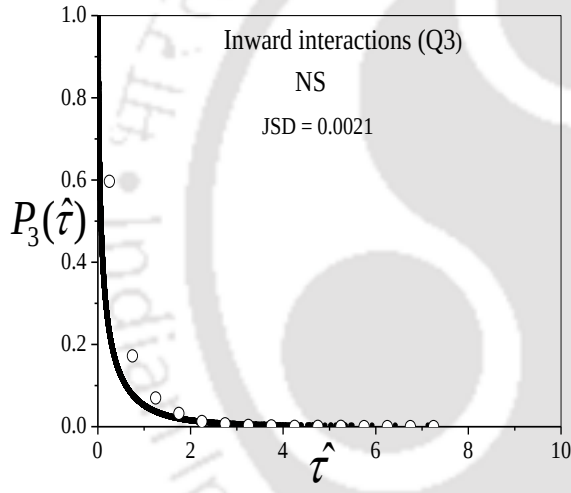
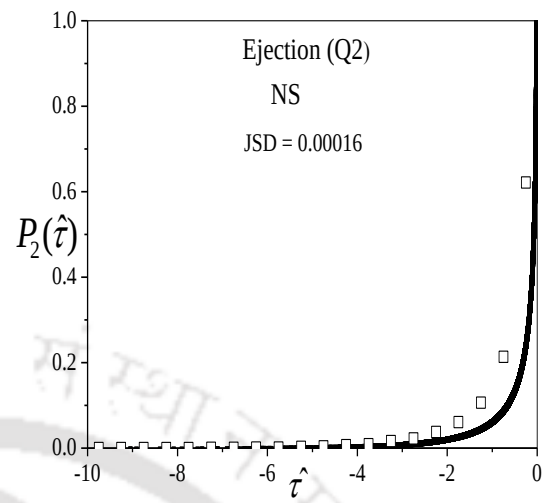
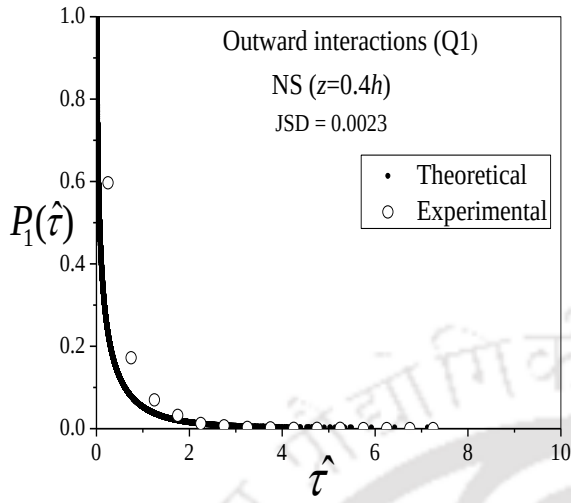


Figure 3.62: Comparisons of computed $P_i(\hat{\tau})$ with experimental data at $z=0.03h$.



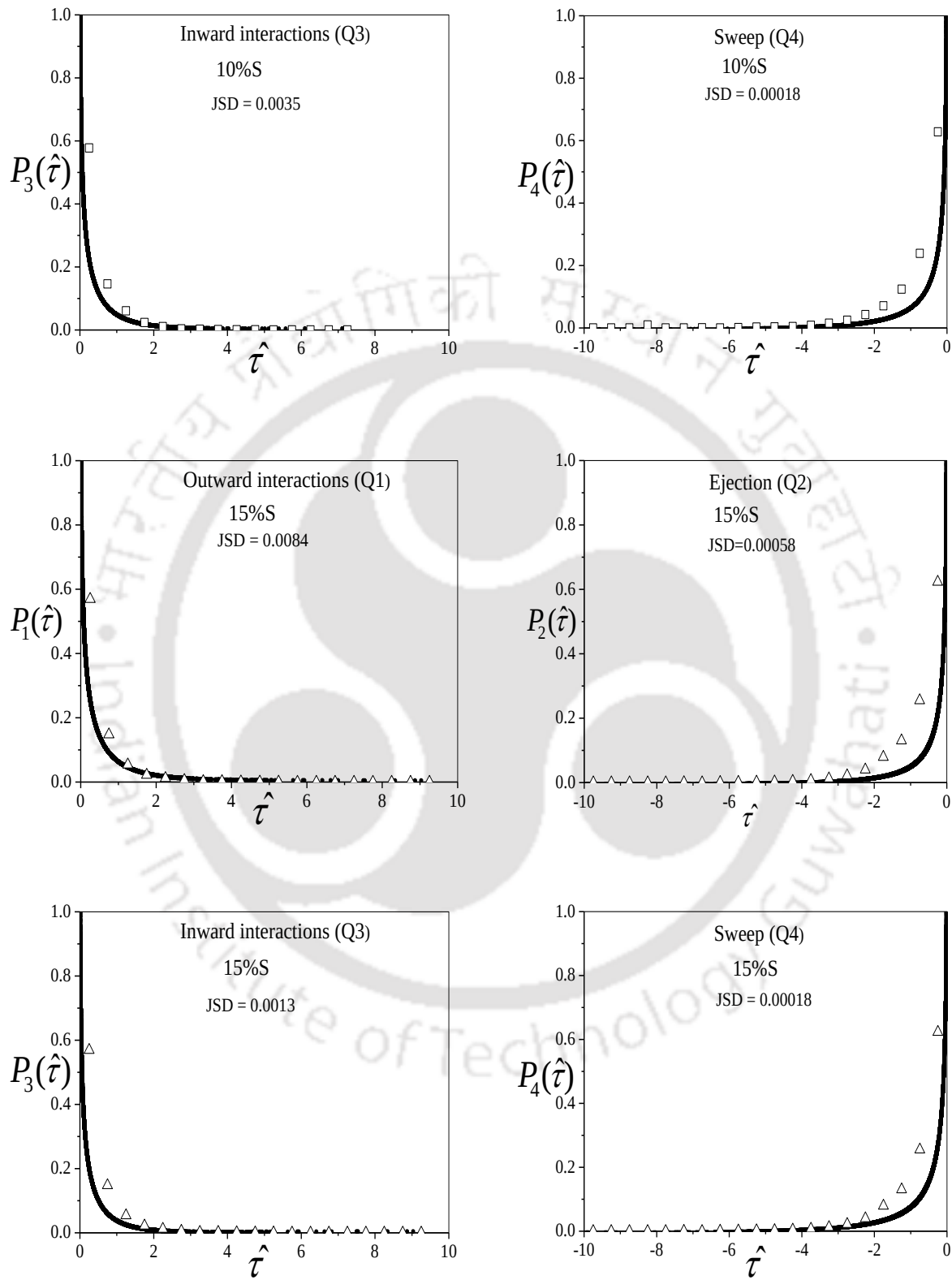


Figure 3.63: Comparisons of computed $P_i(\hat{\tau})$ with experimental data $z=0.4h$

3.12.5 Probability Functions for Turbulent Intensity

The turbulent intensity indicates the level of turbulence and it is defined by turbulence intensity in stream wise direction $\sigma_u = (\overline{u'u'})^{0.5}$ and vertical direction $\sigma_w = (\overline{w'w'})^{0.5}$. Thus, σ_u^2 is the average value of the product of arbitrary variables $u'u'$ whose PDF depends upon the joint PDF of u' and u' . Considering the dimensionless random variable $\hat{\sigma}_u = \hat{u}\hat{u}$, the marginal PDF of $\hat{\sigma}_u$ is given in term of joint PDF $P_{\hat{u},\hat{u}}(\hat{u},\hat{u})$ as

$$P_{\hat{\sigma}_u}(\hat{\sigma}_u) = \int_{-\infty}^{\infty} P_{\hat{u},\hat{u}}\left(\hat{u}, \frac{\hat{\sigma}_u}{\hat{u}}\right) \frac{d\hat{u}}{|\hat{u}|} \quad (3.34)$$

Inserting $P_{\hat{u},\hat{v}}(\hat{u},\hat{v}) = \frac{1}{4\pi^2} \sum_{j=0}^{\infty} \sum_{k=0}^{\infty} i^{j+k} C_{jk} I_j(\hat{u}) I_k(\hat{v})$ in Equation (3.34), where

$$I_j(\hat{u}) = \int_{-\infty}^{\infty} \frac{\xi^j \exp(-i\hat{u}\xi)}{(1+\xi^2)^{j+1}} d\xi, \quad I_k(\hat{v}) = \int_{-\infty}^{\infty} \frac{\xi^k \exp(-i\hat{v}\xi)}{(1+\xi^2)^{k+1}} d\xi \quad (3.35)$$

Integrals of the resulting form is obtained whose value is given by **(Gradshteyn and Ryzhik, 1980)**

$$\int_0^{\infty} x^{n-1} \exp\left(-\gamma x - \frac{\beta}{x}\right) dx = 2 \left(\frac{\beta}{\gamma}\right)^{\frac{n}{2}} K_n\left(2\sqrt{\beta\gamma}\right) \text{ For } \beta > 0, \gamma > 0 \quad (3.36)$$

In this way, Equation (3.34) yields

$$\begin{aligned} P_{\hat{\sigma}_u}(\hat{\sigma}_u) &= K_0(2\sigma_{u1}) - \frac{1}{4} C_{20} \left[K_0(2\sigma_{u1}) + \sigma_{u1} K_1(2\sigma_{u1}) - \sigma_{u1}^2 K_2(2\sigma_{u1}) - \hat{\sigma}_u K_0(2\sigma_{u1}) \right] \\ &+ \frac{1}{64} C_{40} \left[(1 + \sigma_{u1}^2 + \sigma_{u1}^4) K_0(2\sigma_{u1}) + 2\sigma_{u1} (1 - \sigma_{u1}^2) K_1(2\sigma_{u1}) - 2\sigma_{u1}^2 K_2(2\sigma_{u1}) \right] \\ &- \frac{1}{48} C_{40} \hat{\sigma}_u \left[3K_0(2\sigma_{u1}) + 3\sigma_{u1} K_1(2\sigma_{u1}) - \sigma_{u1}^2 K_2(2\sigma_{u1}) \right] \\ &+ \frac{1}{192} C_{40} \left[9K_0(2\sigma_{u1}) + 9\sigma_{u1} K_1(2\sigma_{u1}) - 3\sigma_{u1}^2 K_2(2\sigma_{u1}) - 6\sigma_{u1}^3 K_3(2\sigma_{u1}) + \sigma_{u1}^4 K_4(2\sigma_{u1}) \right] \\ &+ \dots \end{aligned} \quad (3.37)$$

Where, $\sigma_{u1} = (|\hat{\sigma}_u|)^{0.5}$. Providing the recurrence relationship $K_{n+1}(z) - K_{n-1}(z) = \frac{2n}{z} K_n(z)$ of the modified Bessel function $K_n()$, The Equation (3.37) simplifies to

$$P_{\hat{\sigma}_u}(\hat{\sigma}_u) = K_0(2\sigma_{u1}) - \frac{C_{20}}{4} K_0(2\sigma_{u1})(1 - \sigma_{u1}^2 - \hat{\sigma}_u) + \frac{C_{40}}{48} \left[K_0(2\sigma_{u1})(3 - 3\sigma_{u1}^2 + \sigma_{u1}^4 - 3\hat{\sigma}_u + \hat{\sigma}_u \sigma_{u1}^2) + K_1(2\sigma_{u1})(-2\sigma_{u1}^3 + \hat{\sigma}_u \sigma_{u1}^3 - 3\hat{\sigma}_u \sigma_{u1}) \right] \quad (3.38)$$

In the similar way, the expression for $P_{\hat{\sigma}_w}(\hat{\sigma}_w)$ can be derived as

$$P_{\hat{\sigma}_w}(\hat{\sigma}_w) = K_0(2\sigma_{w1}) - \frac{C_{02}}{4} K_0(2\sigma_{w1})(1 - \sigma_{w1}^2 - \hat{\sigma}_w) + \frac{C_{04}}{48} \left[K_0(2\sigma_{w1})(3 - 3\sigma_{w1}^2 + \sigma_{w1}^4 - 3\hat{\sigma}_w + \hat{\sigma}_w \sigma_{w1}^2) + K_1(2\sigma_{w1})(-2\sigma_{w1}^3 + \hat{\sigma}_w \sigma_{w1}^3 - 3\hat{\sigma}_w \sigma_{w1}) \right] \quad (3.39)$$

Where $\hat{\sigma}_w = \hat{w}\hat{w}$ and $\sigma_{w1} = (|\hat{\sigma}_w|)^{0.5}$. The theoretical curves for PDFs of $P_{\hat{\sigma}_u}(\hat{\sigma}_u)$ and $P_{\hat{\sigma}_w}(\hat{\sigma}_w)$ are evaluated by approximating the relative frequency $f_{\hat{\sigma}_u}(\hat{\sigma}_u)$ and $f_{\hat{\sigma}_w}(\hat{\sigma}_w)$ of the random variables $\hat{u}\hat{u}$ and $\hat{w}\hat{w}$ respectively which is estimated from the experimental data at a flow depth z . The required parameters are calculated with the Equations (3.26) and (3.27) to calculate the theoretical expression for $P_{\hat{\sigma}_u}(\hat{\sigma}_u)$ and $P_{\hat{\sigma}_w}(\hat{\sigma}_w)$ given by derived Equations (3.38) and (3.39). Figure 3.64 and 3.65 displays computed and observed $P_{\hat{\sigma}_u}(\hat{\sigma}_u)$ and $P_{\hat{\sigma}_w}(\hat{\sigma}_w)$ distributions at near the bed ($z=0.03h$) and away from the bed ($z=0.4h$) for both no seepage and seepage cases respectively. Figure 3.64 and 3.65 shows that experimentally calculated $P_{\hat{\sigma}_u}(\hat{\sigma}_u)$ and $P_{\hat{\sigma}_w}(\hat{\sigma}_w)$ collapse satisfactorily (JSD=0.007-0.015) with the computed curve which suggests that developed equations for $P_{\hat{\sigma}_u}(\hat{\sigma}_u)$ and $P_{\hat{\sigma}_w}(\hat{\sigma}_w)$ follows the universality which is applicable for both no seepage and seepage runs.

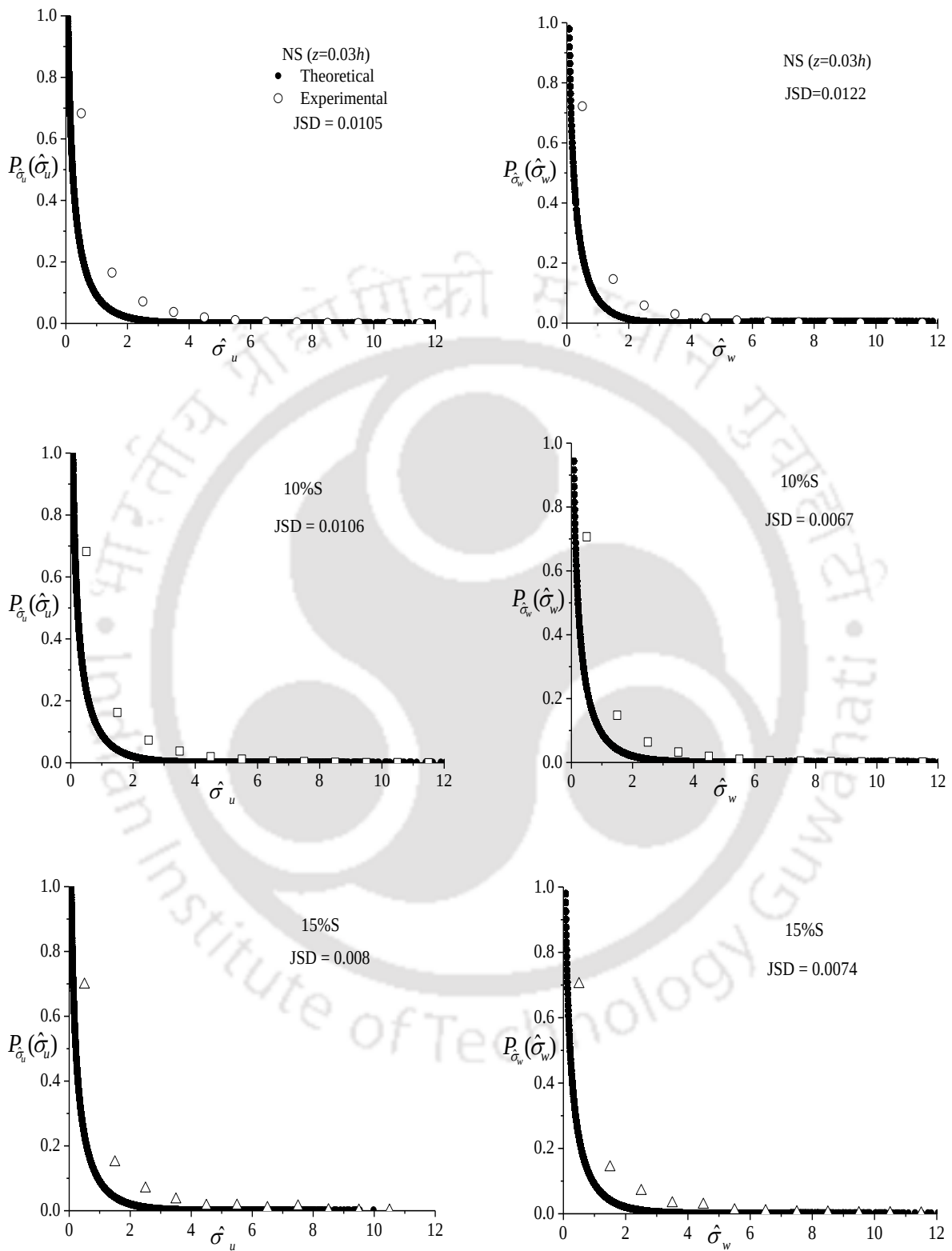


Figure 3.64: Comparisons of computed $P_{\hat{\sigma}_u}(\hat{\sigma}_u)$ and $P_{\hat{\sigma}_w}(\hat{\sigma}_w)$ with experimental data at $z=0.03h$

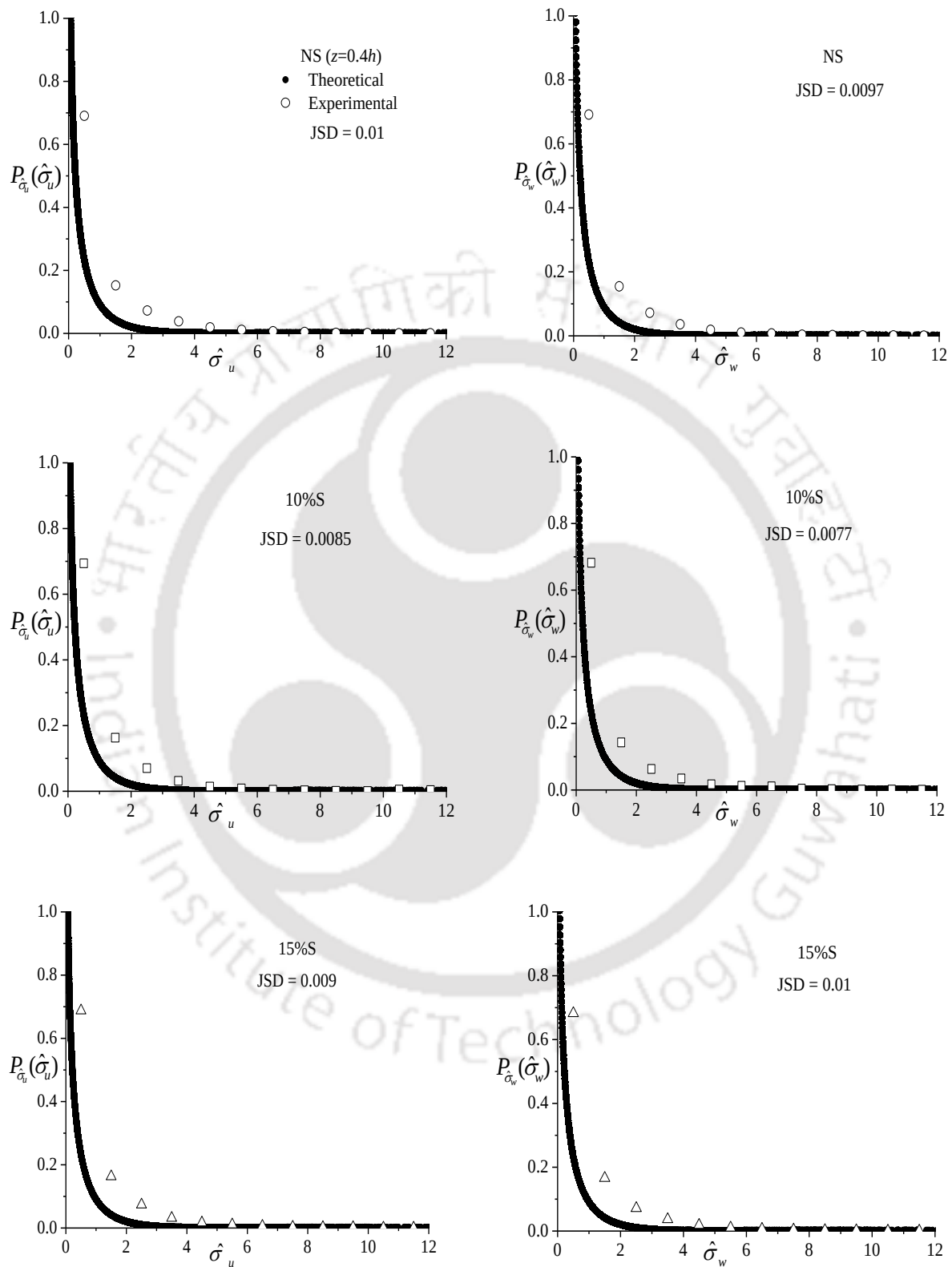


Figure 3.65: Comparisons of computed $P_{\hat{\sigma}_u}(\hat{\sigma}_u)$ and $P_{\hat{\sigma}_w}(\hat{\sigma}_w)$ with experimental data at $z=0.4h$

3.13 Discussions

The basic results corresponding to the application of seepage are coherent with those mentioned in the literature (**Deshpande and Kumar, 2016a**) that (a) time averaged velocity profile is influenced by the seepage (b) The turbulent intensities are enhanced by downward seepage, (c) The Reynolds shear stress near the bed and thus shear velocities increased with seepage. One of the main motivation of this study is the use of non-uniform sediment mixture which is practically observed in the riverbeds and tried to observe every possible parameter of turbulent flow which is directly or indirectly linked with sediment transport of heterogeneous sediment mixture. Though the particle size distribution of sediment in transport is finer than the distribution of bed material because of selective transport, the coarser fraction of sand present in sediment mixture induced more resistance to the flow as compared to the bed composed of uniform sand. Due to this reason, the profiles of turbulent flow structure in non-uniform sediment beds as mentioned in the results section is more or less good agreement with flow characteristics in case of uniform sand bed provided in the existing literature (**Rao and Sitaram, 1999; Lu et al., 2008; Rao et al., 2011; Cao and Chiew, 2014**). Further, in general, the flow profiles in the near-bed flow zone are important for understanding the flow-bed interactions. Primarily, to explain the shear force exchanged by the fluid and solid phases, one can study the near bed flow features as the cause of motivating force for the entrainment of bed particles. Therefore, instantaneous velocities data were taken by ADV immediately after the application of seepage to understand the change in turbulent flow characteristics. Close observation of turbulence parameters with no seepage and seepage runs, leads to a number of insights. Analysis of the time-averaged velocity profile reveals that velocity increases on an average value of 2.36% with 10% seepage and the velocity for 15% seepage is slightly higher with an average increasing value of 0.40% than the velocity for 10% seepage. This increase in velocities near the bed with seepage caused the bed materials to move rapidly. Increase in the depth of the virtual bed level and the zero velocity level reflect that the particles on the channel bed were exposed to a higher component of stream wise velocity. Increase in the near-bed stream wise velocities caused by the presence of downward seepage is in conformity with the findings of previous researchers (**Oldenziel and Brink, 1974; Maclean, 1991b; Deshpande and Kumar, 2016a**). Effect of sediment transport on von Karman's constant is visible in several studies (**Bennett and**

Bridge, 1995; Nikora and Goring, 1998; Cooper, 2006; Koll, 2006; Gaudio *et al.*, 2011).

These researchers have reported that for flows with bed load transport, the value of von Karman's constant decreased from its universal value. In the present study, decrease in the value of von Karman's constant suggests the prevalence of sediment transport during the seepage experiment. Increase in maximum Reynolds shear stress by 13.8% with 10%S and 14.3% with 15%S corresponds to increase in shear velocities (5.83% with 10%S and 8.85% with 15%S) suggesting greater momentum transfer towards the boundary in seepage runs, where higher turbulence intensities have also been observed as compared to no seepage runs. Near the bed in seepage runs, greater positive values of F_{ku} and greater negative values of F_{kw} than those of no seepage runs correspond to an increased flux of stream wise turbulent kinetic energy in the stream wise direction and increased flux of vertical turbulent kinetic energy in the downward direction, which in turn correlates with the increased bed mobility in seepage runs. In the upper flow layer, negative F_{ku} suggests the flux of stream wise turbulent kinetic energy is transported in the opposite direction, which can be related to the retardation caused by inertia of the fluid layer. In seepage runs, less negative F_{ku} in upper flow layers and greater positive F_{ku} near the bed mean that retardation was reduced; the effect of which can be thought of as a relative increase in velocities when the downward seepage was applied. Further, the investigation of bursting events results increased contribution of ejections and sweeps towards the production of Reynolds shear stresses in seepage runs as compared to no seepage runs. Analysis of the contribution by stronger events also shows the dominance of sweep events in Reynolds shear stress production. This increase in high speed fluid parcel towards the boundary in seepage runs is responsible for increased sediment transport with seepage. Vertical distributions of third moments of velocity fluctuations also give insights into phenomena taking place after the application of downward seepage. In the region near to the bed, greater stream wise flux of stream wise RNS in the stream wise direction and greater vertical flux of RNS in the downward direction indicate increased roughness concentration and increased particle mobility in the seepage run. With the application of downward seepage, higher turbulence production associated with decreased turbulent diffusion and decrease in turbulent kinetic energy dissipation has been observed, which indicate that there was an increased level of turbulence in the main channel flow that may have caused increase in sediment transport with seepage. The analysis of turbulent mixing length suggests that the traversing length of turbulent eddy decreases, which further signifies the increase in

sediment transport in the channel under the action of downward seepage. Increased eddy size in the near bed surface tends to higher energy and momentum transfer and lower destruction of turbulent motions as reflected from turbulence scale distributions. Thus higher levels of turbulence is achieved near the bed with an increased eddy size, which results in higher rate of sediment transport with downward seepage.

3.14 Conclusions

An experimental study has been carried out to observe changes in the turbulent characteristics of flow when downward seepage was applied to an alluvial channel composing of non-uniform sediment mixture. Close observation of the turbulent structure of flow leads to the conclusion that bed material transport increases after the application of downward seepage to the channel.

The variations of stream wise velocity with vertical distance in flows with downward seepage in the near bed region are greater than no seepage flow. This increase in stream wise velocities was sufficient for increased rate of sediment transport with seepage. The profiles of Reynolds shear stress is slightly scattered in general and increase with seepage which signifies the greater momentum transfer towards the boundary. The distribution of Reynolds shear stress undergoes a damping due to a decreasing level of turbulence fluctuation within the wall shear layer. The thickness of roughness sub layer and shear velocities were increased with the application of downward seepage. The reduced value of von Karman's constant corresponds to increase in the bed load transport with seepage. In presence of seepage, stream wise and vertical turbulence intensities increases as compared to turbulence intensities in no-seepage. The turbulence is strongly anisotropic being seepage independent and varying approximately linear with flow depth. The correlation coefficient r_{uv} decreases almost linearly with vertical distance in no-seepage. The profiles of r_{uv} are smaller in seepage than those in no-seepage. The turbulent diffusivity and the mixing length decrease with seepage. Decrease in the slope of turbulent mixing length corresponds to lower mixing length of turbulent eddies with seepage.

In quadrant analysis, the contribution of sweep events towards Reynolds shear stress production near the boundary is increased with increased zone of sweep dominance. This increase in high speed fluid parcel towards the boundary in seepage runs is responsible for higher rate of sediment transport. The mean time of occurrence of ejections and that of sweeps in seepage are

longer than those in no-seepage. In near bed flow, the third order correlations M_{jk} show positive distributions of M_{30} and M_{12} and negative distributions of M_{03} and M_{21} . The distributions of M_{jk} suggest that the sweeps are prevalent over the near bed flow depth associated with the arrival of high speed fluid parcel. Increased fourth order correlation in stream wise direction within near bed surface represents higher level of turbulence intermittency with seepage.

In the near-bed flow, the stream wise TKE flux is observed to travel in downstream direction and the vertical TKE flux is transported in the downward direction. Both stream wise and vertical TKE flux with seepage are greater than those of no seepage. Importantly, the effect of seepage on the TKE budget is pronounced. Seepage influenced higher turbulence production and higher pressure energy diffusion rate, lower turbulent diffusion and lower TKE dissipation rate due to which the level of turbulence in the flow has been increased with the application of downward seepage. A reduction in near bed TKE dissipation rate in seepage flows corresponds to an increase in near bed eddy size reflected from the Taylor micro scale. The integral time scale and length scales were increased significantly with seepage, which results the increased energy and momentum transfer caused by the larger eddy size in the near bed region.

The PDF distributions of Reynolds shear stress, velocity fluctuations, and bursting events are calculated theoretically using a GC series expansion depending on the exponential distribution considering the moment up to fourth order. The computed and experimental PDF distributions for both no seepage and seepage case are in good agreement except that a slight departure of inward and outward interactions is observed which may be due to weaker events. Also the present study develops the theoretical expression for the turbulence intensity which is good agreement with the experimental results and follows the universality which is applicable for no seepage and seepage runs. The presented work is theoretically applicable to probabilities of mobile bed sediments in seepage affected alluvial channels typically influenced by the turbulent parameters. Further, accurate predictions of PDFs of various turbulent parameters are a crucial element of many stochastic bed load models and therefore can extend the applicability to more realistic and complex conditions. Furthermore, PDFs provide a broad statistical picture of the turbulent parameters either at single or finite number of points in the turbulent flow field.

4 Sheet Flow Hydrodynamics over Non Uniform Sand Bed Channel

4.1 Overview

The hydrodynamics of sediment transport in alluvial channel is of crucial significance in predicting the elevation of the bed profile. The mode of sediment transport is generally in the form of saltation and changes to sheet flow, when main flow is sufficient to fully transport sediment (**Bagnold, 1966**). Because of the difference between tractive force and resistive force in a sand bed channel, movement of a large quantity of sediment forms a sheet layer (**Gotoh & Sakai, 1997**). Based on laboratory and field data, various researchers (**Dingler & Inman, 1976; Conley & Inman, 1992; Ribberink & Al-Salem, 1994**) observed that the sheet flow layer is a thin layer having few centimeter thickness comprised of moving sediment particles and suggested that the bed remains plane under sheet flow conditions and becomes independent of other bed topography features. Based on field observations, **Inman et al. (1986)** found a substantial increase in the sheet flow layer thickness corresponds to the occurrence of turbulent bursting close to the point of maximum velocity. **Wilson (1966)** pointed that the roughness height may be much larger under the condition of sheet flow. The sediment particles interaction or the interaction between the sediment particle and water in the layer of sheet flow caused high energy dissipation which may be responsible for the increase in the roughness height (**Camenen et al., 2006**). Based on the experimental observations, **Wilson (1987)** predicted that the roughness thickness is approximately equal to the sheet layer thickness. **Asano (1992)** applied a new technique based on video observation to determine the sheet layer thickness and observed that it is dependent on Shields parameter. **Li and Sawamoto (1995)** observed that the sheet layer thickness is directly related with the unsteadiness of the flow. The thickness of the sheet layer is directly linked to the turbulence intensity over Shields parameter (**Dohman-Janssen, 1999, Wilson, 1989**). Many researchers (**Sumer et al., 1996; Wilson, 1987**) have proposed a linear

relation in between the thickness of the sheet flow layer and the maximum Shields parameter in order to evaluate the maximum sheet layer thickness. Field measurements of sand concentration in the sheet flow layer have been done in the swash zone on a beach on the island of Norderney by **Bakker *et al.* (1988)** and **Yu *et al.* (1990)**. They measured the maximal intrusion depth of sheet flow of about 2 to 5 mm and the maximal moving layer of about 8 mm. Based on experimental study, **Flores and Sleath (1998)** developed relations for sheet flow layer thickness with median particle diameter and Shields parameter. **Jenkins and Hanes (1998)** observed that the sheet flow layer affects the hydraulic flow properties in an alluvial channel. The occurrence of a sheet layer increased the roughness height and turbulence damping of flow (**Dohmen-Janssen *et al.*, 2001**).

An image analysis technique proposed by **Ahmed and Sato (2003)** and conductivity concentration meter (CCM) probes used by researchers (**Dohmen-Janssen *et al.*, 2002**) were used to measure the thickness of the sheet flow layer. **Ahmed and Sato (2003)** suggested a model to forecast the movement of the sheet layer using a characteristic particle size of mixed sediments. **Hsu *et al.* (2004)** found that the roughness height is considerably influenced by the sheet flow layer of high sediment concentration. **O'Donoghue and Wright (2004)** proposed the relation between temporal variation of sheet layer thickness and temporal variation of Shields parameter. Considering a Gaussian-based, narrow band arbitrary process, **Myrhaug and Holmedal (2007)** proposed logical relations for specific statistical values of the thickness of the sheet flow layer. **Sato (2011)** utilised an image analysis technique to forecast the maximum thickness of the sheet layer in an asymmetric oscillatory flow combination with the relatively opposite currents. In collinear waves and currents, considering the wave current interaction effect and the influence of flow acceleration, **Sato and Liu (2013)** utilized the optical illumination technique to estimate the maximum thickness of sheet flow layer. **Revil-Baudard and Chauchat (2013)** developed an analytical model in order to form the relation of non-dimensional sheet flow layer thickness with median particle diameter, mean sheet flow layer concentration, Shields parameter, and the static friction coefficient. Shear stress more than the critical shear stress caused the formation of a sheet layer in the overbank flow condition (**Pitlick *et al.*, 2013**). **Lanckriet *et al.* (2014)** used an instrument named the conductivity concentration profiler to measure the sheet flow layer thickness in the field of swash-zone hydrodynamics and developed a relation among sheet flow layer thickness, median particle diameter and a mobility number.

Patel et al. (2015) developed an empirical equation for the sheet flow rate subjected to downward seepage in a curved cross section sand bed channel. Recently, **Deshpande and Kumar (2016b)** developed an empirical equation for the sheet layer thickness subjected to downward seepage in a curved cross section, sand bed channel.

Considering the importance of seepage (section 1.3) and non-uniform sediment mixture (section 1.2), the present chapter introduces both the parameters in the study of sheet layer movement. Although, there have been earlier research on sheet flow interaction, the flow structure in the occurrence of sheet layer subjected to downward seepage and sheet flow hydrodynamic with seepage in non-uniform sand bed channel are still unexplored. In order to evaluate the effect of downward seepage, present chapter emphasizes the effect of downward seepage on the flow velocity profile, Reynolds shear stress, turbulence intensities and energy dissipation in the occurrence of sheet flow layer through a series of experiments conducted over non-uniform sand bed channel. The change in sheet layer movement along the longitudinal direction is also studied which will deliver a comprehensive knowledge on non-uniform sand bed channel with the effect of downward seepage.

4.2 Changes in Turbulence Parameters in the Presence of a Sheet Layer

Experiments were carried out in no seepage and with seepage conditions. **Sumer et al. (1996)** observed a sheet flow layer when flow velocity is in the range of (0.27-1.21) m/sec due in accordance with these findings a flow discharge (Q_0) of $0.0488 \text{ m}^3/\text{sec}$ was selected with a corresponding flow velocity of 0.417 m/sec in the present study to observe the sheet flow layer. Measurements of flow parameters were taken in both conditions to assess the hydraulic parameters. Here, some of the turbulent parameters such as flow velocity, Reynolds shear stress, turbulence intensity and energy production and dissipation are analyzed with and without the presence of sheet flow layer for understanding the behavior of the channel. Analyses along the depth profile illustrate the effect of downward seepage along the vertical flow depth. Despite knowing that sheet layer affects the flow characteristics, the main objective lies in understanding the flow conditions at the absence and the presence of sheet layer, which will enhance our existing knowledge about the effects of sheet flow layer on turbulent flow characteristics.

4.2.1 Time Averaged Velocity

Figure 4.1 and Figure 4.2 show the vertical distribution of the streamwise (u) and vertical (w) time averaged velocities with and without the presence of a sheet layer. In Figure 4.1, the stream wise velocities are plotted against the normalized depth of flow (z/h) for no seepage and seepage runs, where z is the distance from the bed surface and h is the flow depth. For no seepage, the streamwise and vertical flow velocity at the sheet layer achieves maximum velocity near the free surface and is gradually reduced towards the bed because of the drag imposed by the sheet thickness.

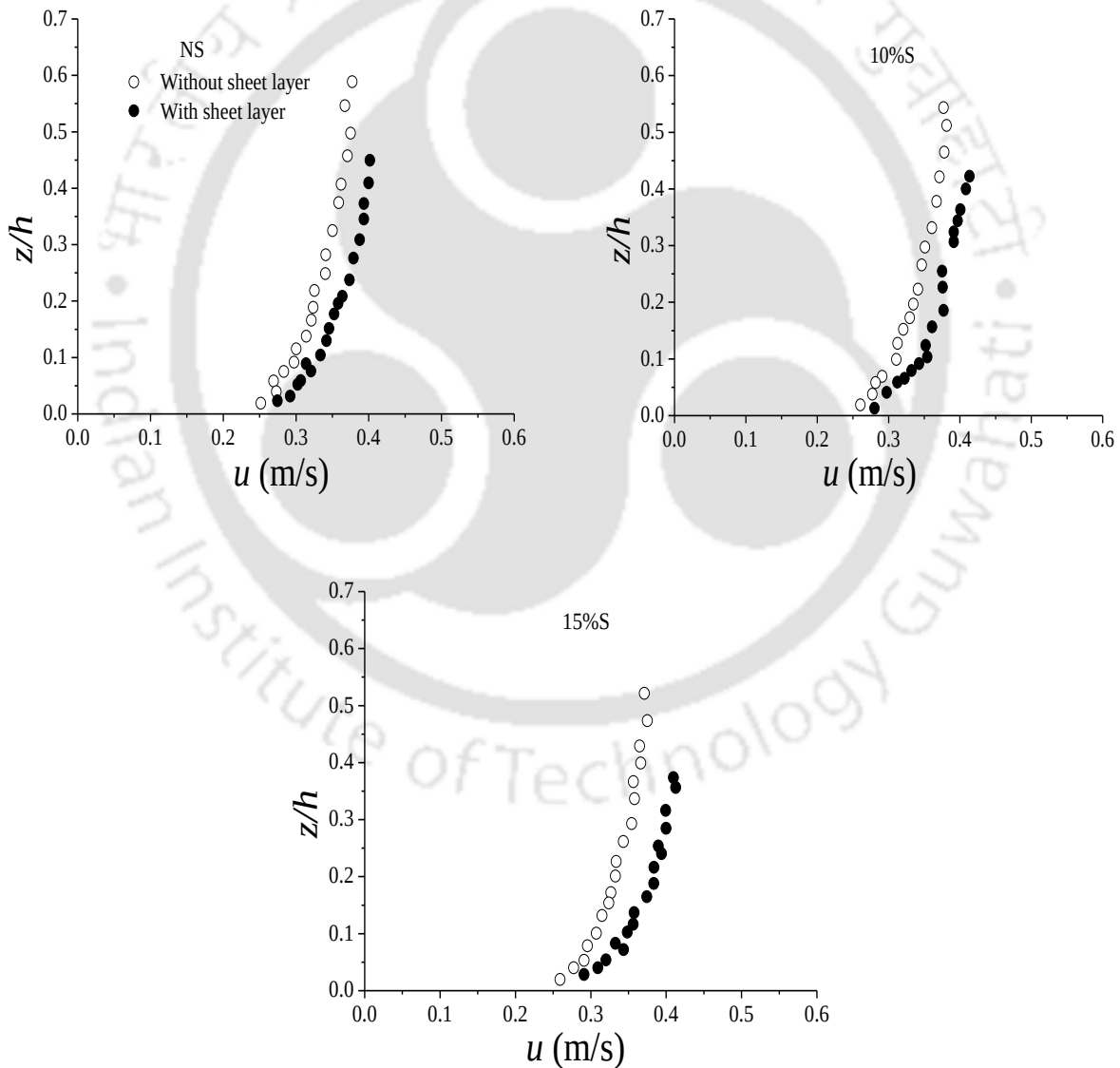


Figure 4.1: Vertical profiles of stream wise velocity profile with and without the sheet layer

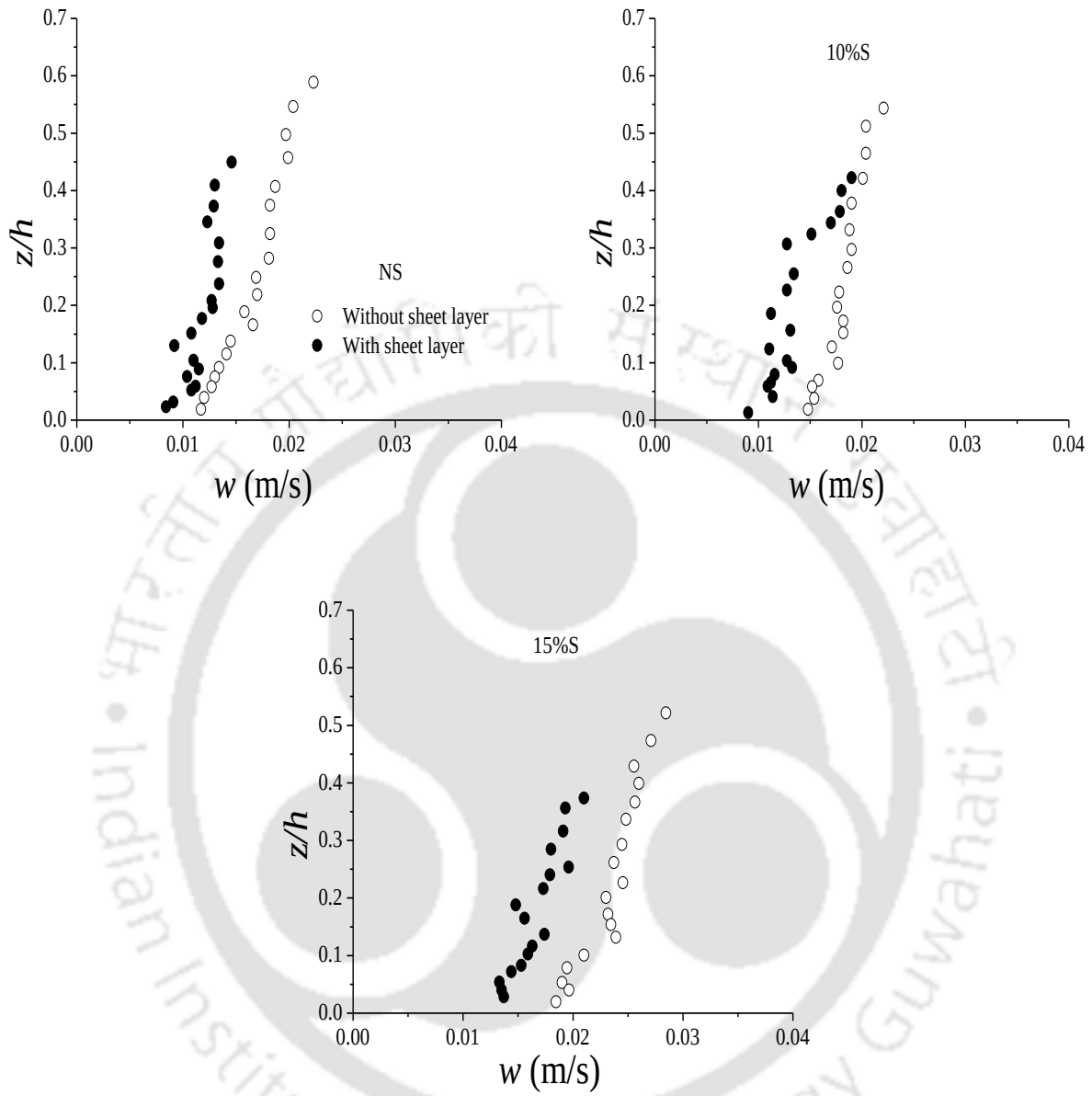


Figure 4.2: Vertical profiles of vertical velocity profiles with and without the sheet layer

From Figure 4.1, it can be seen that the presence of the sheet layer accelerates the velocity profiles with regard to the absence of the sheet layer while the result is the opposite in the case of flow velocity in the vertical direction (Figure 4.2). The presence of the sheet layer reduces the flow depth in the channel which suggests that velocities over the sheet layer increased in the streamwise direction and decreased in the vertical direction. Because of mass conservation, increase in the stream wise velocities tends to decrease in the vertical velocities with a sheet layer. Further, quantitative analysis suggests that at near to the bed, streamwise velocity

increases on an average by 8-12% and vertical velocity decreases by 32-40% respectively in the presence of a sheet layer. Previous investigations into the application of downward seepage have shown that the velocity profile moves downward and, hence, a higher velocity is achieved in the near bed region (**Cao and Chiew, 2014, Devi et al., 2016**). Velocities in the seepage run were measured just after the application of downward seepage. With the application of seepage, the flow velocity in the absence and presence of a sheet layer is higher than in the no-seepage case, because of the shifting of the higher velocity zone near the bed, as found in the previously mentioned research on application of downward seepage. This increase in velocity caused an increase in the rate of sheet layer movement in the channel. Even with the application of seepage, it is also observed that in the lower flow region, the streamwise velocity in the presence of sheet layer is higher with an average increase value of 7-12% and vertical velocity becomes lower by 34-40% than the velocity without a sheet layer.

4.2.2 Reynolds Shear Stress (RSS)

Reynolds shear stress indicates the momentum exchange in between the turbulent flow and the sediment particles. It is, therefore, an important parameter in governing the soil erosion and sediment transportation.

Vertical distributions of RSS with and without a sheet layer for no seepage and seepage runs are shown in Figure 4.3. It can be seen from Figure 4.3 that RSS increases along the channel bed are associated with the momentum provided from the main flow to maintain sediment particle motion overcoming the bed resistance, which attain a maximum value in between $0 < z/h < 0.2$ and then again decreases towards the boundary because of the presence of a roughness sub-layer in the near bed region. In the present experiments, the profiles of RSS distribution were found to be similar in the presence and absence of a sheet layer but the magnitudes are higher in the presence of a sheet layer subjected to no seepage and seepage runs. This leads to the occurrence of more momentum exchange in the presence of a sheet layer. Additionally, the reducing nature of RSS is recognized as the result of the fact that the farther the location from the sheet layer, the smaller is the turbulence generated by the flow due to a sheet layer. Thus, the vertical distribution of RSS is determined in order to define the momentum diffusion mechanism. An important observation

is that the maximum RSS in the presence of a sheet layer is more than the RSS without a sheet layer by a value of 16.7% and 25-43% for no seepage and seepage respectively.

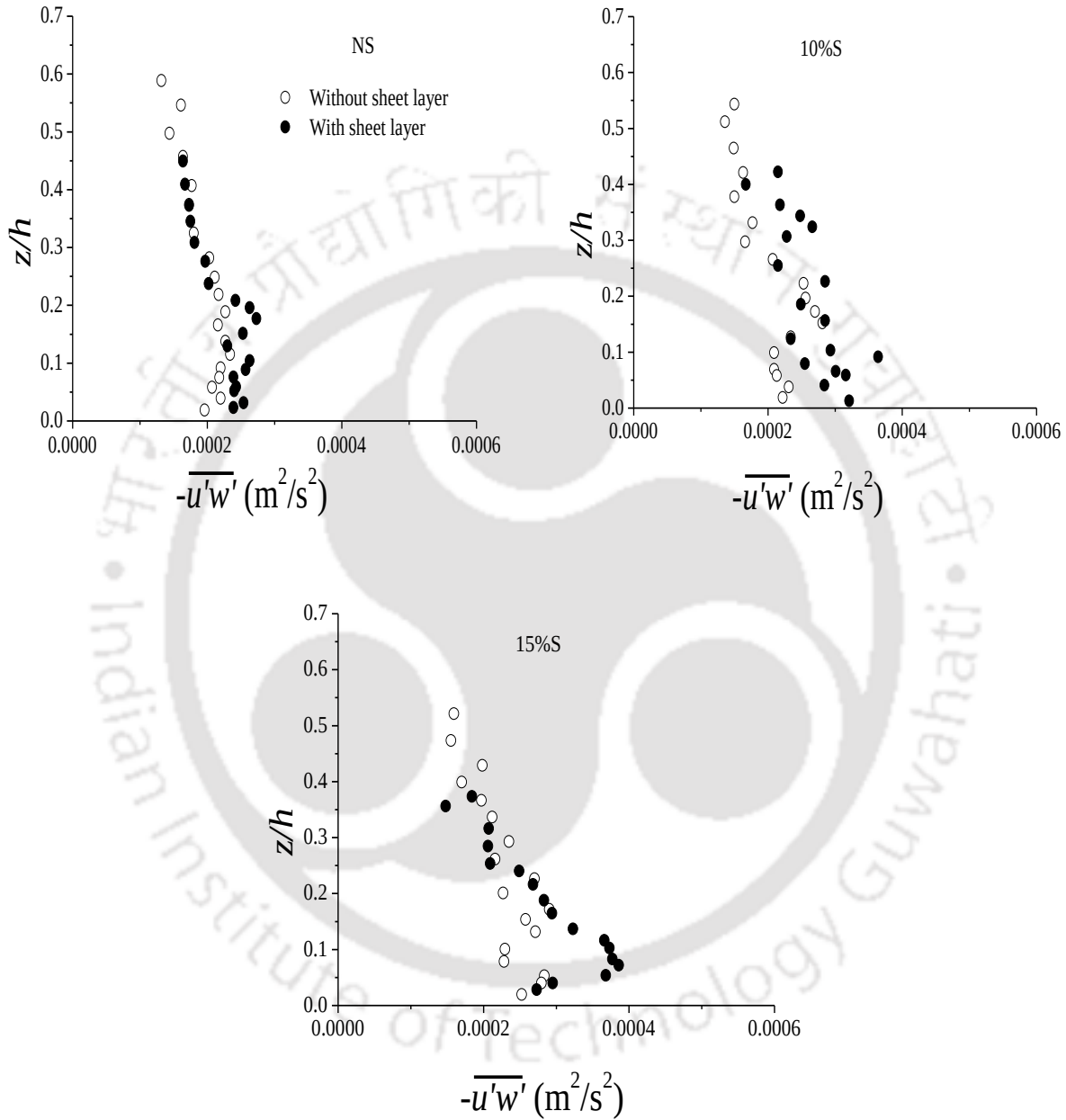


Figure 4.3: Vertical profiles of Reynolds shear stress profiles with and without the sheet layer.

It is known that the downward seepage increases the RSS which leads to higher rate of sediment transport than those of no-seepage (**Devi et al., 2016**). In all the profiles, with the increase in seepage, an increase in the RSS occurs. This increase in the RSS suggests the higher momentum

transfer arising near the bed in the presence of downward seepage. This reiterates that the presence of seepage in the downward direction increases the RSS thereby increasing bed material transport. Further, a significant increase in the RSS has been found in the flow over the sheet layer. The maximum RSS in the presence of a sheet layer is increased by 38% from no-seepage to 10% seepage and 47% from no seepage to 15% seepage. Consequently, these results mean that because of the increase in the RSS in the presence of downward seepage, the rate of sheet layer movement is increased with the increase of seepage. It can be concluded that RSS generated large fluctuations in the presence of a sheet layer which means that flow velocities induce strong fluctuations over the sheet layer. Because of the effect of fluctuations in the velocity, RSS accounts for turbulent fluctuations in the flow. RSS is produced by the velocity fluctuations and can lead to considerable turbulence intensities.

4.2.3 Turbulence Intensities

Turbulence intensity is the result of differences in instantaneous velocity at a point, which can be influenced by the magnitude of the velocity, and bed roughness, etc.

Vertical distributions of streamwise and vertical turbulence intensities with and without the sheet layer for no seepage and seepage runs are shown in Figure 4.4 and 4.5 respectively where the vertical ordinate (z) has been made non-dimensional with the flow depth (h). At the location of the sheet layer, turbulence intensities show the maximum value in the range $0 < z/h < 0.2$, where the maximum RSS is achieved and it becomes stable near the water surface. In the presence of a sheet layer, it fluctuates more near the bed as compared to the free surface because of the influence of the boundary effect. The presence of the sheet layer increases the level of turbulence intensities, as compared to the case without a sheet layer. An important observation is that the streamwise turbulence intensity (σ_u) in the near bed flow zone over the sheet layer is more than the turbulence intensity without a sheet layer by a value of 9-14% and 6-13% for no seepage and seepage, respectively. In the case of vertical turbulence intensity (σ_w), the increases becomes 5-10% and 5-12% for no seepage and seepage, respectively. Downward seepage increases the fluctuation component that helps in increasing the turbulence in the flow; In general, the profiles of turbulence intensity with downward seepage are greater than the profiles of turbulence intensity with no seepage. This observation is in conformity with previous researchers (Nezu,

1977; Kim and Sung, 2003). Vertical profiles of turbulence intensities are evident within the near bed flow zone. As compared to no seepage, the degree of turbulence intensity in the presence of a sheet layer increases by an average value of 14.6% for σ_u and 5.1% for σ_w with 10%S and increases with 15%S on average by 22.2% for σ_u and 7.6% for σ_w .

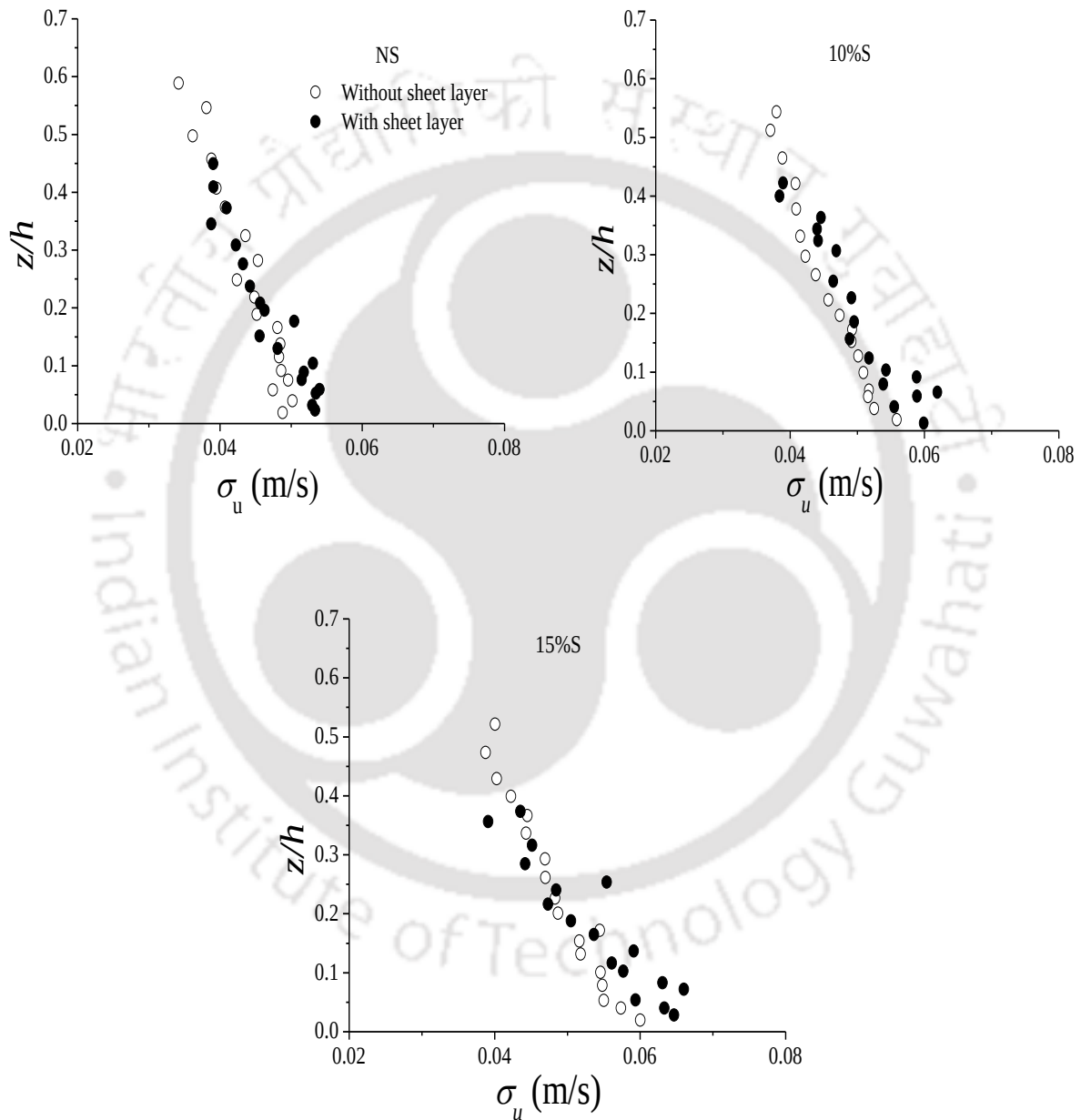


Figure 4.4: Vertical profiles of stream wise turbulence intensity with and without sheet layer

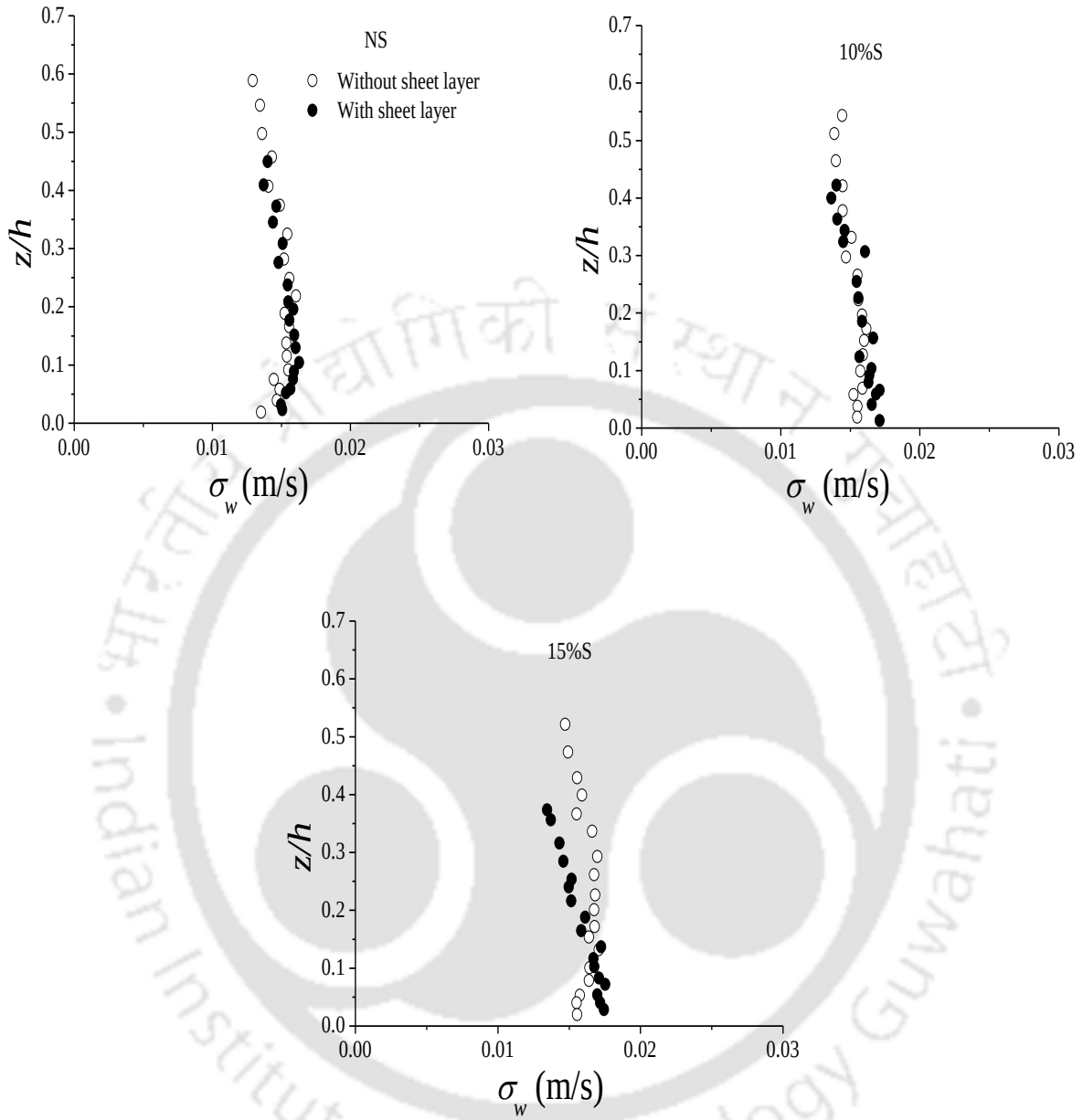


Figure 4.5: Vertical profiles of vertical turbulence intensity with and without sheet layer

4.2.4 Energy Production and Dissipation of Flow

The TKE production for uniform open channel flow is given by **Nezu and Nakagawa (1993)**.

$$t_p = -\overline{u'w'} \frac{\partial u}{\partial z} \quad (4.1)$$

Turbulence dissipation, ε , is evaluated by method as follows (Irwin 1973, Krogstadt and Antonia, 1999):

$$\varepsilon = \frac{15\nu}{u^2} \overline{\left(\frac{\partial u'}{\partial t}\right)^2} \quad (4.2)$$

The turbulent energy production and dissipation are normalized in the form as $(T_p, E_D) = (t_p, \varepsilon) \times (h/u_*^3)$ where u_* is the shear velocity. Figure 4.6 presented the energy production and dissipation in flows subjected to no seepage and seepage run. For all profiles of T_p and E_D , it is observed that maximum value is found near the bed region where there is a high velocity differential. The normalized values of T_p and E_D show that similar pattern has been observed with and without sheet layer for no seepage, 10% seepage, and 15% seepage. It is observed that T_p and E_D decrease rapidly with an increase in z/h becoming nearly constant (with small magnitude) for flow near the surface region. Positive T_p and negative T_p specifies the conversion of TKE from the time mean flow to turbulence and vice versa. The turbulence production is increased with a sheet layer as compared to without a sheet layer because of increased Reynolds shear stress and increased momentum transfer from the flow to the sediment particles in the presence of a sheet layer. The turbulence dissipation is decreased with a sheet layer as compared to without a sheet layer which may correspond to the increase in the turbulence production. This decreased level of turbulence dissipation is related with the increased time-averaged velocity and decreased gradient of time-averaged velocity fluctuation in the presence of a sheet layer. Considering the seepage, T_p is increased mainly in the near bed region with seepage than those of no seepage, suggesting an increased level of turbulence production with seepage. This increased level of turbulence is related to the increased turbulence intensities and increased momentum transfer through the non-uniform sand bed in the form of seepage flow. Dissipation of turbulent kinetic energy E_D is reduced in magnitude with seepage because more energy is transferred to turbulent fluctuations; the gain in turbulent kinetic energy becomes higher. The vertical profiles of E_D are similar to those of T_p with a lag. The lag gets smaller in downward seepage with respect to the no seepage case. The most important nature lies in the near bed distributions of E_D that are decreased with seepage which suggests that the rate of

sediment transport is increased due to the gain in turbulence production as a result of the increased rate of sheet layer movement with the increased seepage.

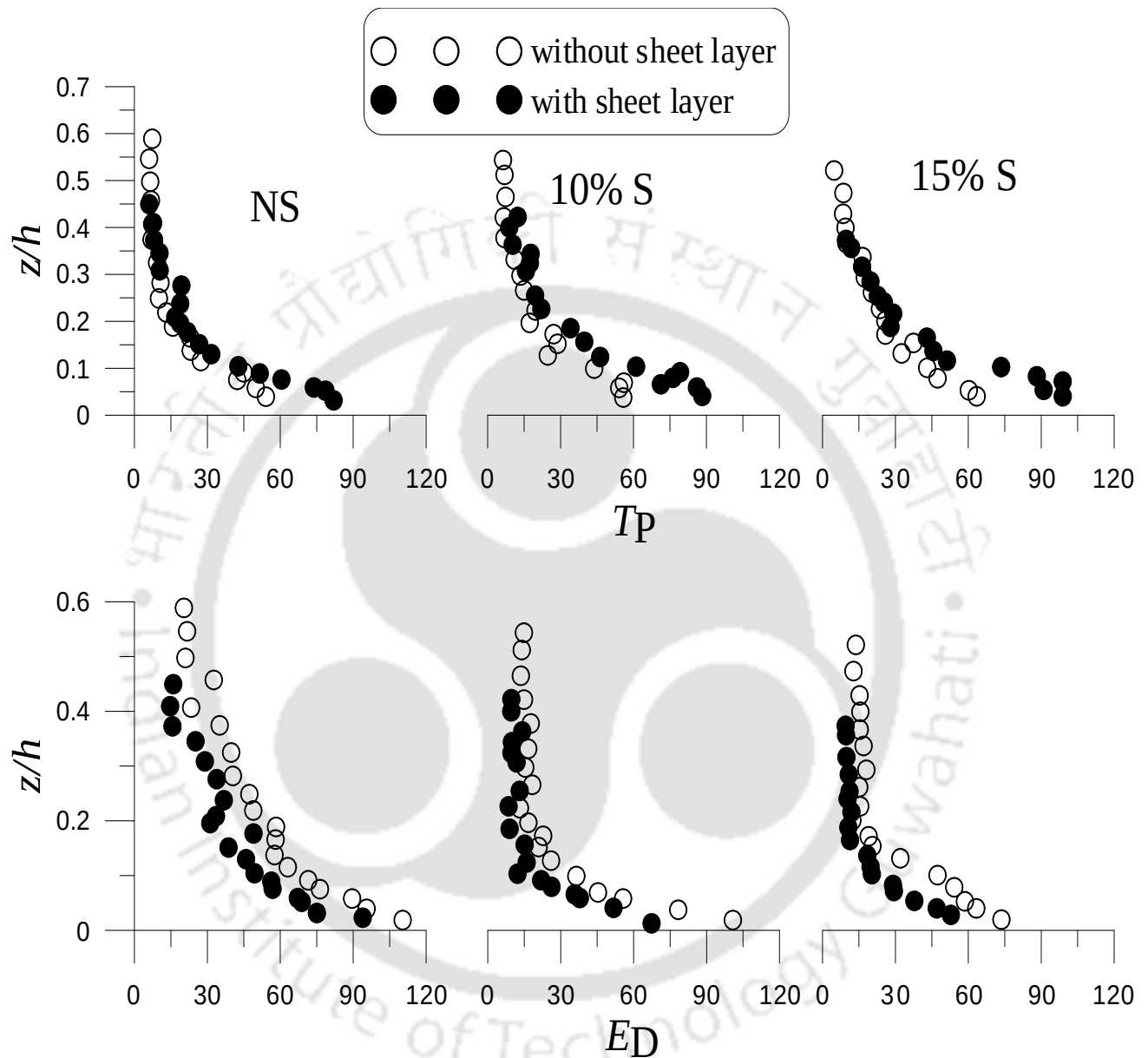


Figure 4.6: Vertical profiles of turbulent production and dissipation with and without sheet layer

4.3 Morphological Dynamics of Sheet Flow

4.3.1 Time Evaluation of Sheet Flow Movement and Sheet Thickness

Sediment particles resting on a wall boundary in an alluvial channel start to move when the degree of hydrodynamic forces is greater than a threshold value. In the present chapter, a

magnitude of force equal to 0.345 N/m^2 which falls in the data range when a sheet layer was developed in the previous studies (**Simons and Richardson, 1965**), is applied as the threshold value. Sediment transport occurred in the form of a sheet flow layer with the absence of any notable bed features. This mode of sediment transport changes to sheet flow when the main flow discharge is high implies that flows transporting sediment particles become suitably powerful. Movement of a large quantity of sediment occurs due to a large difference between the hydrodynamic force and the resistive force which results in the formation of a sheet flow layer (Figure 4.7). As per laboratory and field studies, a sheet flow layer is defined as a thin layer having few centimeter thicknesses comprised of moving sediment particles (**Dingler and Inman, 1976; King, 1991**). The observed sheet layer thickness ($\approx 1.6 \text{ cm}$) for no seepage flow in the present study is near to the approximation of a few centimeter thicknesses of the sheet flow layer (**Dingler and Inman, 1976; King, 1991**). In the present experiments, the channel bed was eroded at the upstream end because of the increased bed shear stress and the eroded material from the bed at a section was deposited on the channel bed at the adjacent section towards downstream and carried forward by the water along the longitudinal length of the main channel in the form of a sheet flow layer. A large amount of sediment eroded from the beds is transported in the form of sheet flow over the bed and the sediment particles move in such a way that the particles roll over one another. In the present study, the sheet flow movement and sheet layer thickness with time increases with the increase of downward seepage as shown in Figure 4.8 and 4.9 respectively. This increase in sheet layer movement and sheet thickness is related to the increase in sediment transport with seepage. These results can be directly link with the role of turbulent flows in sediment transport because the morphodynamical changes are governed by turbulence in the form of entraining and depositing sediment particles. The turbulence parameters such as time-averaged streamwise velocity (Figure 4.1); RSS (Figure 4.3), streamwise turbulence intensity (Figure 4.4) and turbulent energy production and dissipation (Figure 4.6) is increase with seepage which corroborates that the rate of sediment transport is higher in the presence of seepage. This increased rate of sediment transport with seepage results to the increased rate of sheet layer movement as well as the increased rate of sheet layer thickness. In the presence of downward seepage, with the passage of time, the thickness of the sheet layer changes along the length of the channel. Initially, the transport of the sheet flow layer is faster, but after some time interval, the degree of movement is reduced. This change in

movement speed can be observed from the center line profile of sheet layer movement that is shown in Figure 4.10-4.12.



Figure 4.7: Snapshot of the channel showing sheet flow layer

From Figure 4.10, it can be seen that it took around 3, 7.5 and 12 hr for the sheet layer to reach the sections at 12.5, 10 and 8 m, respectively for no seepage flow. This result quantifies that for the initial time interval of 4.5 hr, the sheet layer moves a distance of 2.5m and another 4.5 hr, it moves a distance of 2 m. The rate of sheet layer movement reduced with time, and there was very little difference observed in the sheet layer thickness, and its further movement was reduced after 12 hr. Similarly for the case of seepage flow (Figure 4.11 and 4.12), the rate of sheet layer movement is more reduced with time and further movement was reduced after 12hr. Finally, the results show that for both no seepage and seepage flow, with the reduced increase in sheet layer movement with time, it can be said that the channel was about to attain an equilibrium condition.

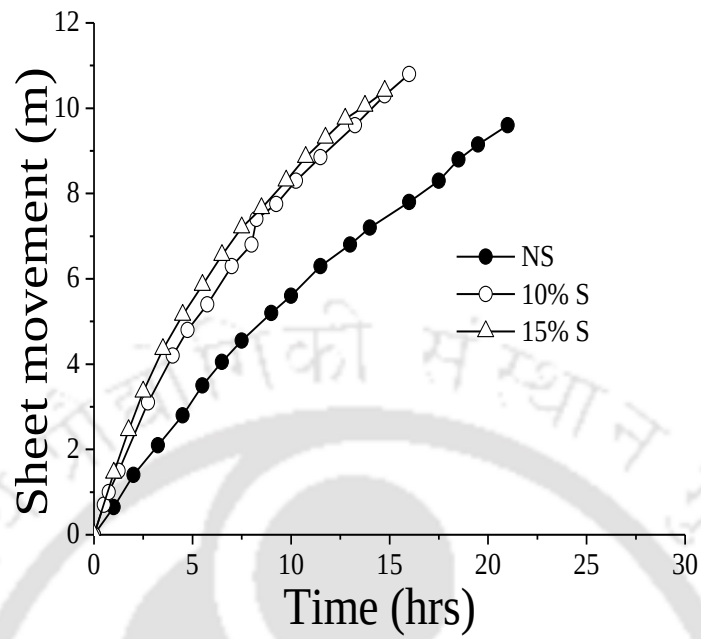


Figure 4.8: Sheet flow layer movement with time

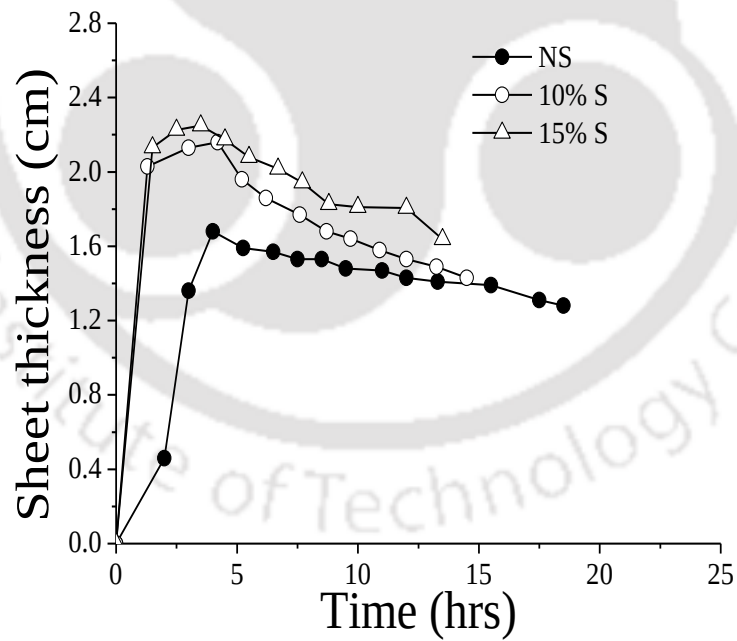


Figure 4.9: Sheet flow layer thickness with time

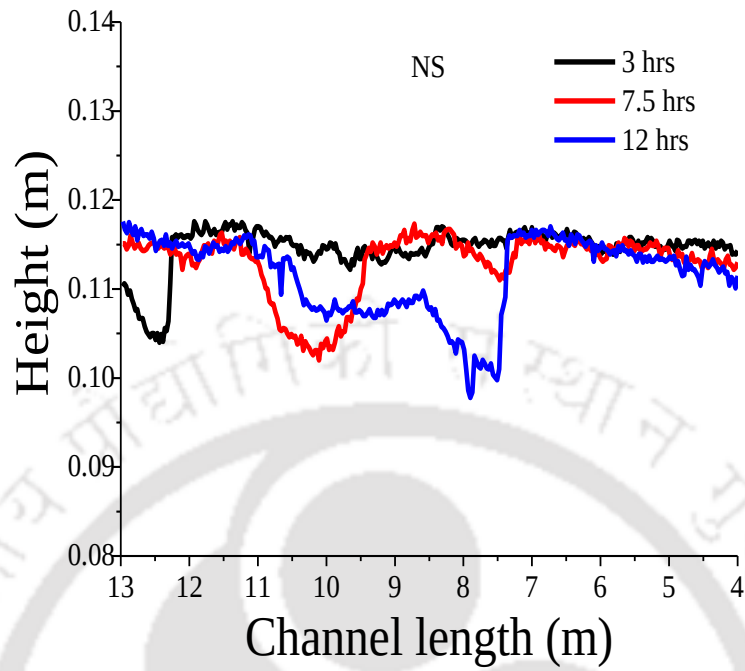


Figure 4.10: Centre line profile of sheet flow movement in the test section (4-13 m from the downstream end) at different time intervals for no seepage flow

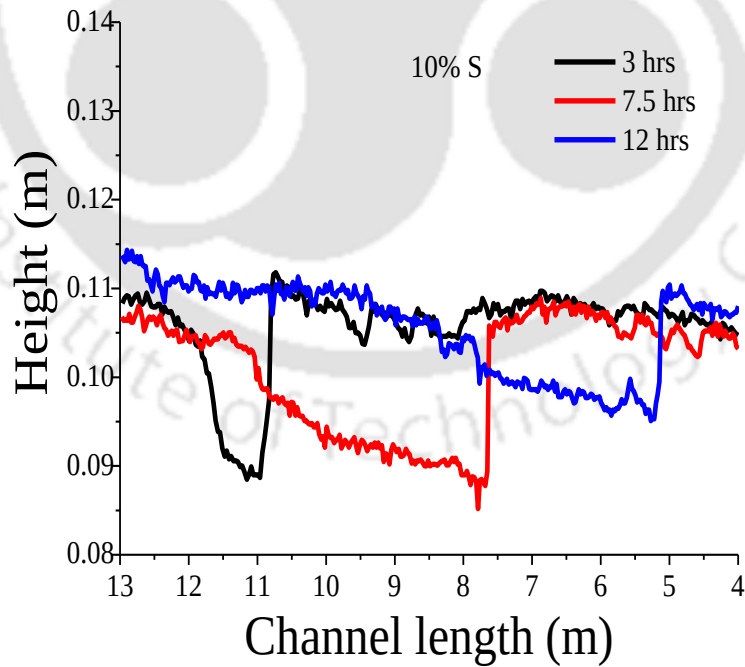


Figure 4.11: Centre line profile of sheet flow movement in the test section (4-13 m from the downstream end) at different time intervals for 10% seepage flow

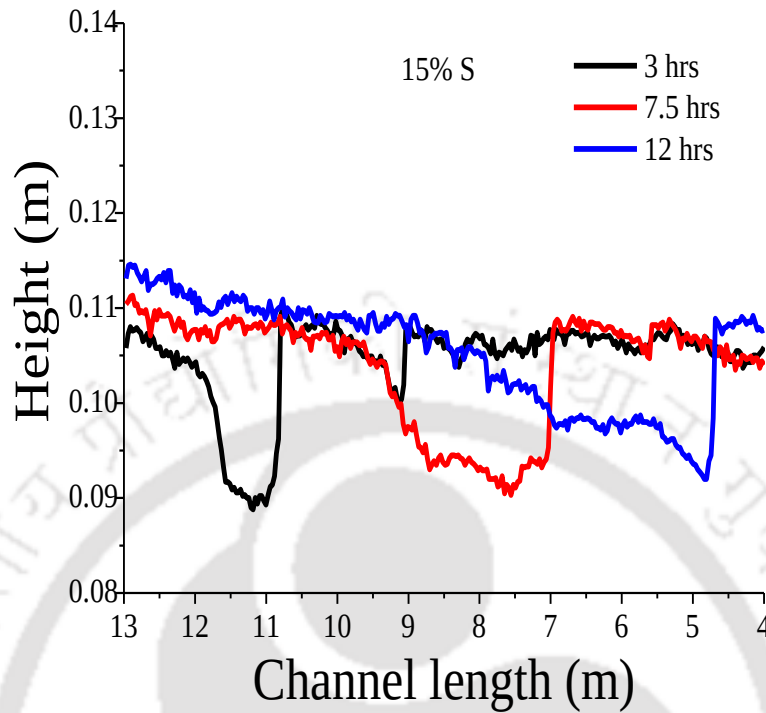


Figure 4.12: Centre line profile of sheet flow movement in the test section (4-13 m from the downstream end) at different time intervals for 15% seepage flow.

4.3.2 Celerity of Sheet Flow Layer

Several studies have investigated the morphodynamic evolution of bedforms from initial conditions to the equilibrium stage to clearly comprehend the multifaceted splitting and merging of bedforms (Defina, 2003; Jerolmack and Mohrig, 2005). Spectral analyses of temporal and spatial bed elevation series at multiple scales (Nordin and Algert, 1966; Nikora *et al.*, 1997; Nikora and Goring, 2001) have been performed to statistically characterize the bedforms. These studies have revealed the presence of multi scale dynamic properties in migrating alluvial bedforms. This was more thoroughly analysed by Singh *et al.* (2011). In their study, time scale dependent celerity of bedforms was computed from two time series data of bed elevation series recorded simultaneously by two echo sounders, placed at a certain distance apart. These echo sounders, in their experiments, were placed at a large distance (compared with bedform dimension) from each other, which stood out as a problem in the efficient computation of celerity. However, in the present study, the data recorded was spatially varied with 7.8 mm

resolution at different time spans in the experimental run. However, to the best of our knowledge, no work has devised the effect of flow on the dynamic properties of migrating sheet flow. To address this lack of knowledge, the main objective of this section is focused on finding the effect of main flow discharge with downward seepage on the celerity of migrating sheet flow in a large scale steady state flume experiment, in which, sheet flow was developed in a non-uniform sand bed channel.

Dynamic characteristics of sheet flow signify the scale dependent celerity of sheet flow. Alluvial sheet flow shows a significant variability of the propagation velocity over the range of different spatial and temporal scales. To capture this phenomenon, wavelet analysis was done over the bed elevation series and scale dependent celerity was obtained by dividing the lag, corresponding to the maximum cross correlation coefficient between two consecutive bed elevation series for a particular scale, by the time interval between those two bed elevation records. In present study, the data recorded were spatial datasets of 0.78 cm resolution recorded on a different time spans of a single experiment. Therefore, the length scale dependent celerity of the sheet flow was computed. By following this approach, the error resulting from the incapability of the cross correlation method to accurately compute the scale dependent celerity from the elevation series recorded by echo sounders placed at a large distance from each other can be eliminated. This circumvents the problem encountered in the experiments done by **Singh *et al.* (2011)**. For analysing quantities that show wide disparity of physical and dynamic characteristics among various ranges of scales, a wavelet transform helps to provide with a more clear and meaningful insight. The continuous wavelet transform of a signal $f(t)$ is defined as the integral transform with a series of functions $\Psi_{a,b}(t)$

$$W_f(a,b) = \int_{-\infty}^{+\infty} \psi_{a,b}(t), f(t) dt \quad (4.3)$$

where $\Psi_{a,b}(t)$ is called the wavelets. Wavelets are obtained from a mother wavelet by translation and scaling

$$\psi_{a,b}(t) = \frac{1}{\sqrt{a}} \psi\left(\frac{t-b}{a}\right) \quad (4.4)$$

That is, the similarity between the signal $f(t)$ and scales of an elementary function is measured. 'a' is known as the scaling parameter and 'b' is known as the location parameter. The factor $1/\sqrt{a}$ is used to conserve the norm (Mallat, 1998; Kumar and Foufoula-Georgiou, 1997). The functions used in the expansion have altering time frequency tiles because of the scaling. $\Psi_{a,b}(t)$ will be short and of high frequency for small a ($a < 1$), while for large a ($a > 1$), $\Psi_{a,b}(t)$ will be long and of low frequency. Thus, a natural discretization will use large time steps for large a , and conversely, fine time steps for small a . For an invertible wavelet transform, the mother wavelet must follow the admissibility condition, i.e. the area under the curve should be zero.

$$\int_{-\infty}^{+\infty} \psi(t) dt = \psi(0) = 0 \quad (4.5)$$

This property makes the wavelet work like a local differencing filter. Wavelets with higher order vanishing moments are also available resulting in higher order differencing filters which are able to remove polynomial trends from a signal. A wavelet with N vanishing moments eliminates a polynomial of order $(N-1)$. A well-known mother wavelet is derived from the family of Gaussian based wavelets. These are defined as the N^{th} order derivative of a Gaussian function, $g_0(t)$, i.e. $g_N(t) = (d^N/dt^N)g_0(t)$, ($N=1,2,\dots$), where N is the order of the derivative. It should be noted that $N=2$ for the Mexican hat wavelet, has been used in the present study. In the present study, the fluctuations of bed elevations at different scales have been computed using the Mexican hat wavelet $g_2(t)$ as the mother wavelet:

$$g_2(t) = (2/\sqrt{3})(\pi)^{-1/4}(1-t^2)e^{-(t^2/2)} \quad (4.6)$$

It should be noted that the present study focuses on the analysis of a spatial dataset. Therefore, instead of time scales, the current signal are discretized into a range of length scales and the wavelet coefficients of bed elevation profiles at a particular scale are considered as bed elevation fluctuations. The wavelet coefficients (WC_f) at a location b and scale a were obtained as:

$$WC_f(a,b) = \frac{1}{\sqrt{a}} \int_R \psi\left(\frac{x-b}{a}\right) f(x) dx \quad (4.7)$$

The bed elevation series shows variability over multiple length, as well as, time scales. Therefore, a multi scale wavelet based cross correlation analysis is expected to offer a valuable

insight regarding the dynamic characteristics of sheet flow. The wavelets cross covariance between two signals, $f_1(x)$ and $f_2(x)$, can be calculated as

$$WCC_{f_1 f_2}(a, \Delta x) = \int_{-\infty}^{+\infty} W_{f_1}(a, b) W_{f_2}(a, b + \Delta x) db \quad (4.8)$$

where $W_{f_1}(a, b)$ and $W_{f_2}(a, b + \Delta x)$ are the wavelet coefficients of two bed elevation series, $f_1(x)$ and $f_2(x)$, respectively, at scale a and two adjacent locations b and $b + \Delta x$ respectively. The wavelet cross correlation can be calculated by normalizing this by variance of the signals. For a particular scale, a , the lag corresponding to the maximum cross correlation coefficient of the two series of wavelet coefficients, obtained from the wavelet transform of two consecutive bed elevation series, is computed at first. As the current datasets were sampled at a resolution of 0.78 cm, one lag will be equal to 0.78 cm. So, the propagation velocity (sheet flow celerity at that scale) was computed by dividing the lag by the time interval between the two consecutive datasets.

The power spectral density function (PSD) reflects the strength of the variations in energy as a function of the frequencies (or scales). The PSDs are the average of the Fourier transform magnitude squared, over a large time interval:

$$S_x(f) = \lim_{T \rightarrow \infty} E \left\{ \frac{1}{2T} \left| \int_{-T}^T x(t) e^{-j2\pi ft} dt \right|^2 \right\} \quad (4.9)$$

Similarly, the PSD can be defined as the Fourier transform of the auto-correlation function.

$$S_x(f) = \int_{-T}^T R_x(\tau) e^{-j2\pi f\tau} d\tau \quad (4.10)$$

where, τ is the time lag and the auto-correlation function, $R_x(\tau)$ is defined as :

$$R_x(\tau) = E \{ x(t) x^*(t + \tau) \} \quad (4.11)$$

where E is the error function. As the data recorded in present experiments is spatially varied, the frequency response will be in wave number instead of hertz. Wave number ω is defined as the inverse of the spatial resolution ($=2\pi/dx$, where $dx=0.78$ cm). The celerity of the sheet flow ranging from a length scale of 50 mm to 250 mm was calculated using wavelet cross correlation. The reason behind choosing 50 mm as the smallest characteristic length scale was the limitation

of our sampled dataset to capture sheet flow layer below a characteristic length scale of 7.8 mm. This length scale was ensured to maintain reasonable accuracy for calculating the sheet flow celerity. The celerity of the sheet flow for various length scales is shown in Figure 4.13. For no seepage and seepage runs, the celerity of the sheet flow has increased in the time duration of (7-15) hr as compared to (0-7) hr.

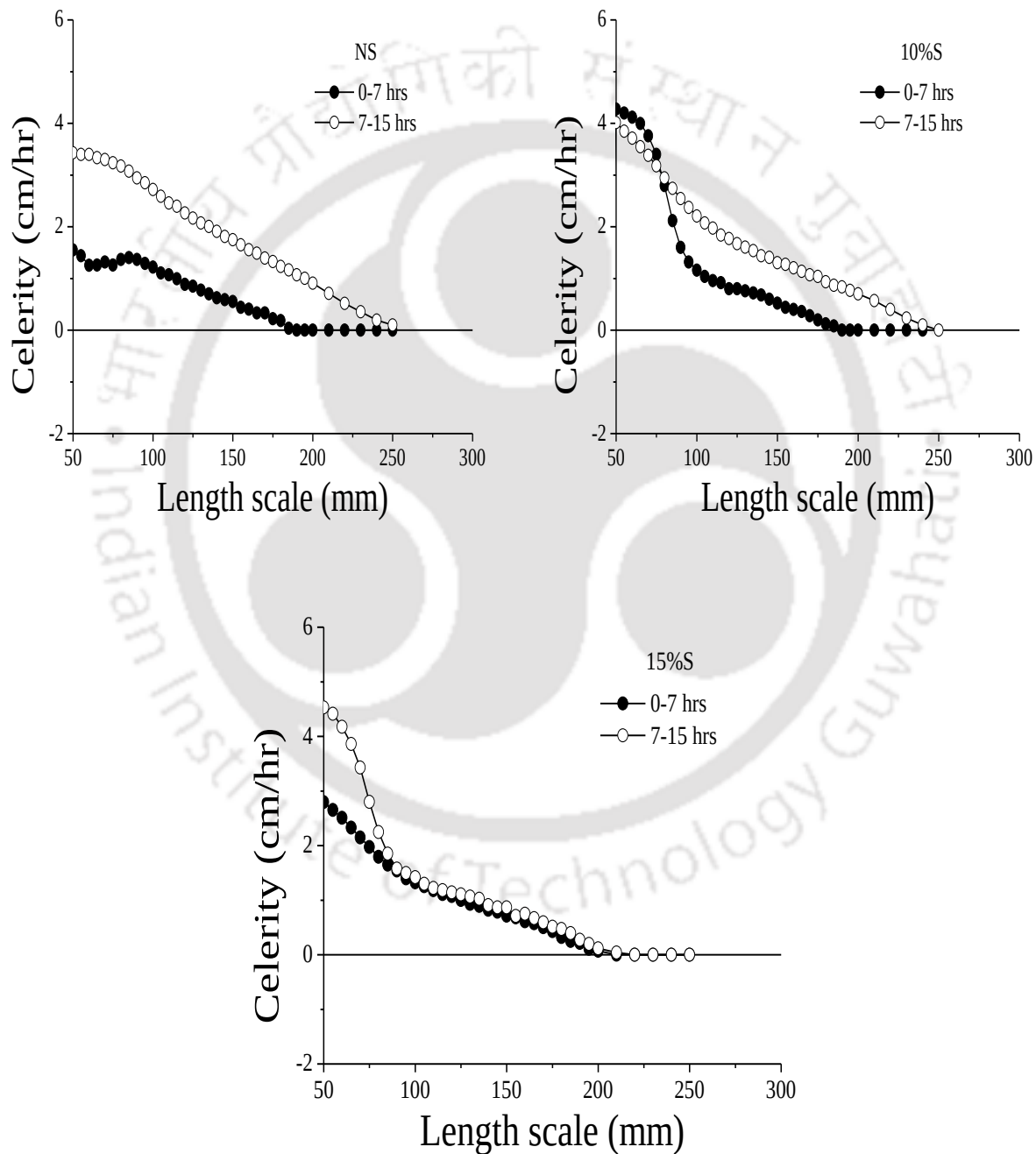


Figure 4.13: Celerity of sheet flow at different time interval

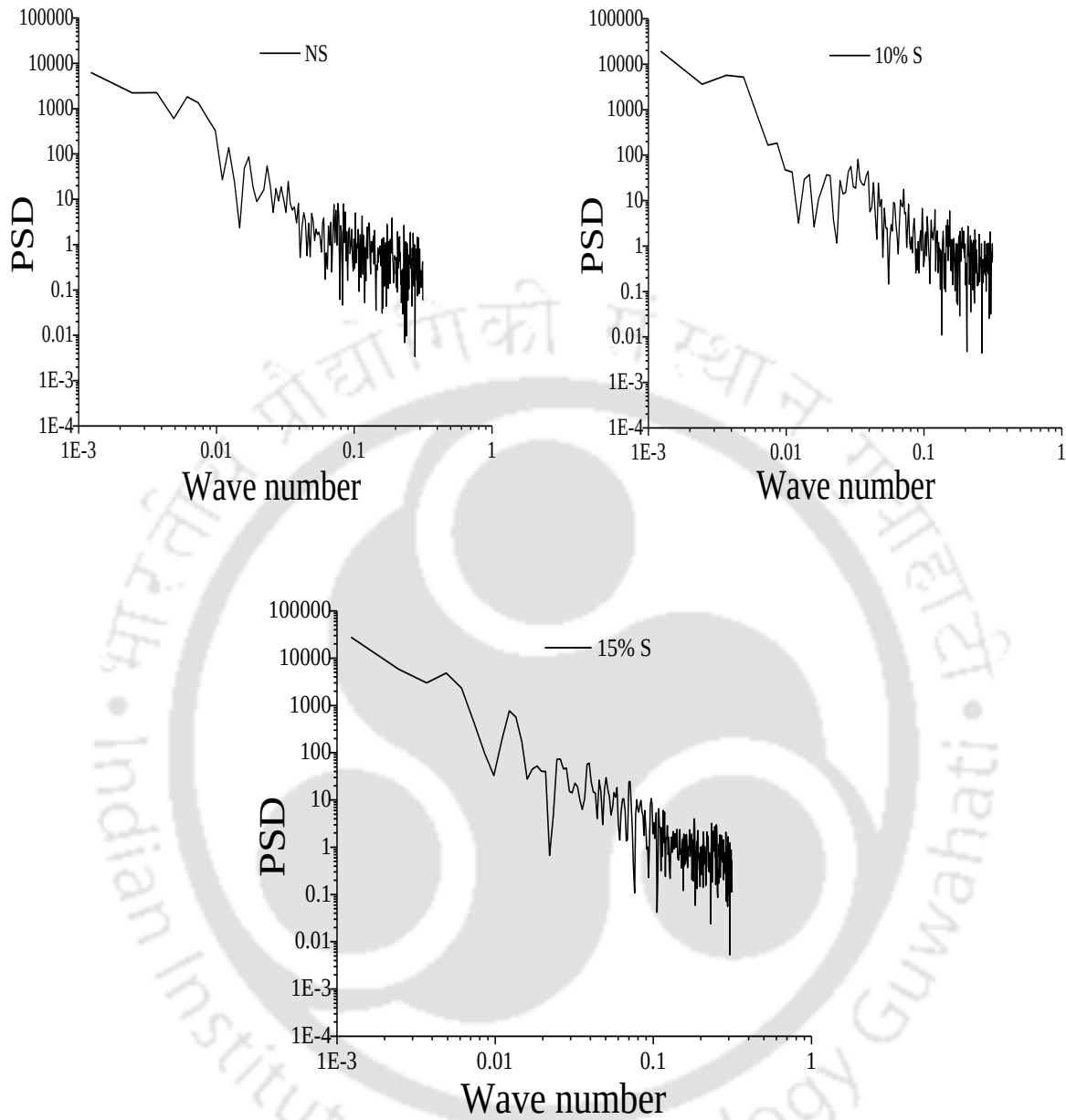


Figure 4.14: Power spectra of bed elevation series after 7 hours

The results show an interesting observation regarding the sheet flow celerity for experiments with the same length scale of same size. The celerity of sheet flow having the same length scale was observed to be lower in the case of no seepage than seepage. The statistics of the sheet flow celerity is tabulated in the Table 4.1, 4.2 and 4.3. The saturation level of the power spectral density with wave number (mm^{-1}) is larger in the case of seepage (Figure 4.14) which justifies an increase in the sheet flow size and consequently an increase in average bed celerity.

Table 4.1: Statistics of the sheet flow celerity (no seepage flow)

Length Scale (mm)	Celerity (cm/hr)					
	0-7 hrs	Mean	Standard Deviation	7-15 hrs	Mean	Standard Deviation
50	1.54839	0.689	0.535	3.43042	1.97	0.996
55	1.43779			3.39806		
60	1.25346			3.39806		
65	1.25853			3.33333		
70	1.31613			3.30097		
75	1.25806			3.23625		
80	1.36406			3.17152		
85	1.40092			3.07443		
90	1.36406			2.94498		
95	1.29032			2.8479		
100	1.21659			2.71845		
105	1.10599			2.589		
110	1.06912			2.45955		
115	0.99539			2.39482		
120	0.88479			2.26537		
125	0.84793			2.16828		
130	0.77419			2.0712		
135	0.70046			2.00647		
140	0.62673			1.90939		
145	0.58986			1.8123		
150	0.553			1.74757		
155	0.4424			1.65049		
160	0.40553			1.5534		
165	0.3318			1.48867		
170	0.3318			1.39159		
175	0.2212			1.32686		
180	0.18433			1.22977		
185	0.03687			1.16505		
190	0			1.06796		
195	0			1.00324		
200	0			0.90615		
210	0			0.71197		
220	0			0.5178		
230	0			0.35599		
240	0			0.19417		
250	0			0.09709		

Table 4.2: Statistics of the sheet flow celerity (10% seepage flow)

Length Scale (mm)	Celerity (cm/hr)					
	0-7 hrs	Mean	Standard Deviation	7-15 hrs	Mean	Standard Deviation
50	4.27572	1.18	1.39	4.01338	1.729	1.093
55	4.1958			3.84615		
60	4.11588			3.71237		
65	3.996			3.54515		
70	3.75624			3.37793		
75	3.3966			3.17726		
80	2.7972			2.94314		
85	2.11788			2.74247		
90	1.5984			2.54181		
95	1.31868			2.37458		
100	1.15884			2.20736		
105	1.03896			2.07358		
110	0.95904			1.97324		
115	0.91908			1.83946		
120	0.7992			1.77258		
125	0.7992			1.67224		
130	0.75924			1.60535		
135	0.71928			1.53846		
140	0.67932			1.43813		
145	0.5994			1.40468		
150	0.51948			1.30435		
155	0.43956			1.2709		
160	0.3996			1.20401		
165	0.35964			1.13712		
170	0.27972			1.07023		
175	0.1998			1.03679		
180	0.11988			0.93645		
185	0.07992			0.86957		
190	0			0.83612		
195	0			0.76923		
200	0			0.70234		
210	0			0.56856		
220	0			0.40134		
230	0			0.23411		
240	0			0.10033		
250	0			0		

Table 4.3: Statistics of the sheet flow celerity (15% seepage flow)

Length Scale (mm)	Celerity (cm/hr)					
	0-7 hrs	Mean	Standard Deviation	7-15 hrs	Mean	Standard Deviation
50	2.7957	0.990	0.807	4.53202	1.311	1.286
55	2.65233			4.41379		
60	2.50896			4.17734		
65	2.32975			3.86207		
70	2.15054			3.42857		
75	1.97133			2.79803		
80	1.79211			2.24631		
85	1.64875			1.85222		
90	1.54122			1.57635		
95	1.39785			1.49754		
100	1.32616			1.41872		
105	1.25448			1.30049		
110	1.1828			1.22167		
115	1.11111			1.18227		
120	1.07527			1.14286		
125	1.00358			1.10345		
130	0.9319			1.06404		
135	0.89606			1.02463		
140	0.82437			0.9064		
145	0.78853			0.867		
150	0.71685			0.867		
155	0.681			0.70936		
160	0.60932			0.74877		
165	0.57348			0.66995		
170	0.50179			0.59113		
175	0.43011			0.51232		
180	0.32258			0.47291		
185	0.2509			0.39409		
190	0.21505			0.27586		
195	0.10753			0.19704		
200	0.07168			0.11823		
210	0			0.03941		
220	0			0		
230	0			0		
240	0			0		
250	0			0		

4.4. Non-Uniformity of Sheet Layer Concentration

The transport of sediment mixtures in open channel flows is complicated because the condition for initiation of motion of a given sediment size as well as its transport rate is affected by the presence of the other sizes in the mixture. The finer fractions moving as bed load are sheltered by the coarse fractions, and the coarser particles are exposed more to the action of flow past them. As such, the actual force experienced by a given fraction of the sediment mixture at a given average shear stress is different from what it would be at the same shear stress if the material were uniform. A bedload sampler was placed at the downstream end of the flume to collect the resultant sheet layer concentration.

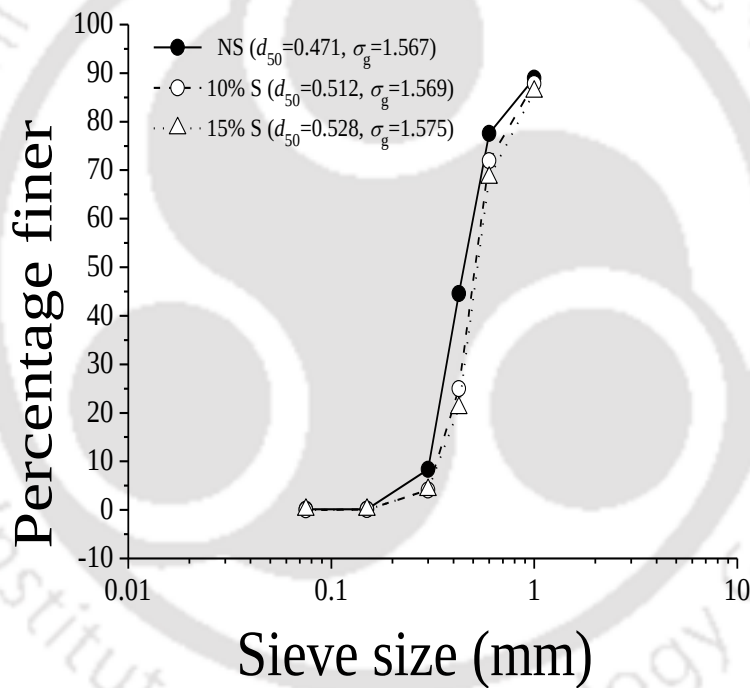


Figure 4.15: Particle size distribution curve of final sheet layer concentration

It has been observed that the sheet layer concentration is increased in the presence of seepage. Also, the particle size distribution curve of the final sheet layer concentration (Figure 4.15) has been modified with and without seepage as compared to the original particle size distribution curve (Figure 2.1). The influence of seepage increased median particle size (d_{50}) and the geometric standard deviation (σ_g) as compared to no seepage runs which suggest that non uniformity of sheet material will be increased with the increase of seepage percentages.

4.5 Conclusions

An experimental study has been done to observe changes in the turbulence characteristics of flow in the presence of a sheet layer in an alluvial channel composed of a non-uniform sediment mixture. The present chapter addresses the vertical distribution of turbulent flow for no seepage and seepage runs. Close observation of the turbulence structure of the flow leads to the conclusion which resulted in increased sheet material transport in the presence of downward seepage in the channel. The variations of streamwise velocity with a sheet layer are greater than those without a sheet layer but the result is the opposite in the case of vertical velocity. Similar trends are observed in no seepage and seepage flows except streamwise and vertical velocities are increased in the presence of seepage. This increase in streamwise velocity was sufficient for an increased rate of sediment transport which corresponds to an increased rate of sheet layer transport with seepage. The profiles of Reynolds shear stress are slightly scattered, in general, and increased with a sheet layer, which signifies a greater momentum transfer towards the boundary. Similar trends are observed in no seepage and seepage flow and Reynolds shear stress increased in the presence of seepage. The distribution of the Reynolds shear stress undergoes a damping due to a decreasing level of turbulence fluctuation within the wall shear layer. In presence of a sheet layer, increases in streamwise and vertical turbulence intensities are observed compared to turbulence intensities without a sheet layer. Similar trends are observed for both no seepage and seepage flow and the trends are increased in the presence of seepage. Importantly, the effect of the sheet layer on the TKE production and dissipation is pronounced. Sheet flow influenced higher turbulence production and a lower TKE dissipation rate due to which the level of turbulence in the flow is increased with the presence of a sheet layer. The centerline profile of the sheet flow movement suggests that initially, the sheet layer moves faster, but with the period of time its movement slows down. The net effect of sheet layer development is to increase the cross-sectional area until the developing shape is about to attain equilibrium where the thickness of sheet flows has very minimal variation. The time taken to reach this stage is around 21 hr for no seepage and 16 hr for seepage runs. The rate of sheet flow movement has been increased in the presence of downward seepage. The reasons behind the increases of sheet layer movement in the presence of downward seepage have been linked to turbulent flow measurements. The present results reveal that the average celerity of sheet flow increases with an increasing

percentage of seepage. This may happen due to the dominance of a high rate of sheet flow movement with seepage in downward direction. Interestingly, sheet flow of same length scale also shows an increase in the celerity with downward seepage. At present, this may be hypothesized as the result of increasing instantaneous flow measurements under the action of downward seepage which destabilizes the bed particles hindering them to follow any continuous and unidirectional movement pattern. Reasons behind formation of the sheet layer of sediment particles for no seepage and downward seepage has been linked to instantaneous flow measurements. Non-uniformity in the resultant collected sheet material (sheet material after the experiment) is increased with the increase in the seepage percentage. These new results provide insight to explain the distinctive morphology of a non-uniform grained sheet layer and turbulent flow structure with a sheet layer subjected to no seepage and seepage runs in a non-uniform sand bed channel.

5 Bedload Transport of Non-uniform Sediment in Seepage Affected Alluvial Channel

5.1 Overview

Bedload transport is one of the crucial parameter for river engineering, channel hydrodynamics and fluvial hydraulics (**Recking, 2009**). Bedload transport occurs owing to the contact of turbulent flow structure with the bed materials (**Einstein, 1950**). The important approach regarding the bedload transport mechanics is using critical parameter such as shear velocity, flow depth and flow velocity. The assumption behind this approach is that there is no bedload transport until the bed shear stress exceeded by critical shear stress and that the bedload transport rate increases in proportion to the increase in the bed shear stress beyond the critical value.

Although previous studies explored bedload transport in the case of non- uniform sand (section 1.2), the bed load transport of alluvial channel composed of non-uniform sediment with downward seepage remains unexplored. Considering the importance of downward seepage as discussed (section 1.3), the current work takes the attentions of influence of seepage flow on bed load transport over a mobile rough boundary. Flume experimental data are generally chosen for the experimental observations because of capability of controlling over flow parameters and bed materials (**Chen and Stone, 2008**). Field investigations normally contain several confusing issues such as dimension of bed material distributions, parameter channel geometry and flow environments that influence the data quality. A wisely planned flume experiment can avoid most of the said problems and offer a more quality data set. Thus, experiments have been done for free-surface flows exposed to downward seepage from the boundary. The present chapter introduces the empirical formula for time rate of sediment transport. Also the present chapter investigates the influence of seepage on the non-uniformity of bed material.

5.2 Non-Uniformity of Resultant Bedload Material

We conducted 90 runs with different median grain size, bed slope, main flow discharges (no seepage flow) and downward seepage rates (Table 2.4). The movement of sediment mixture in open channel flow is complex as the condition for initiation of movement of given particle size as well as its movement rate is influenced by the occurrence of the other sizes in the mixture. The finer materials acting as bed load are sheltered by the coarse materials, and the coarser materials are exposed to the finer materials and flow behaviour. In order to understand the phenomena of sediment transport of heterogeneous sand, we have placed bed load sampler at the downstream end of the flume to collect the transported bed material. It was observed that bed load transport is increased with the application of seepage. We have investigated the particle size distribution curve of resultant bed material for flow discharge $Q=0.04025 \text{ m}^3/\text{s}$, bed slope $S_0=0.001$ and median grain diameter $d_{50}=0.5\text{mm}$.

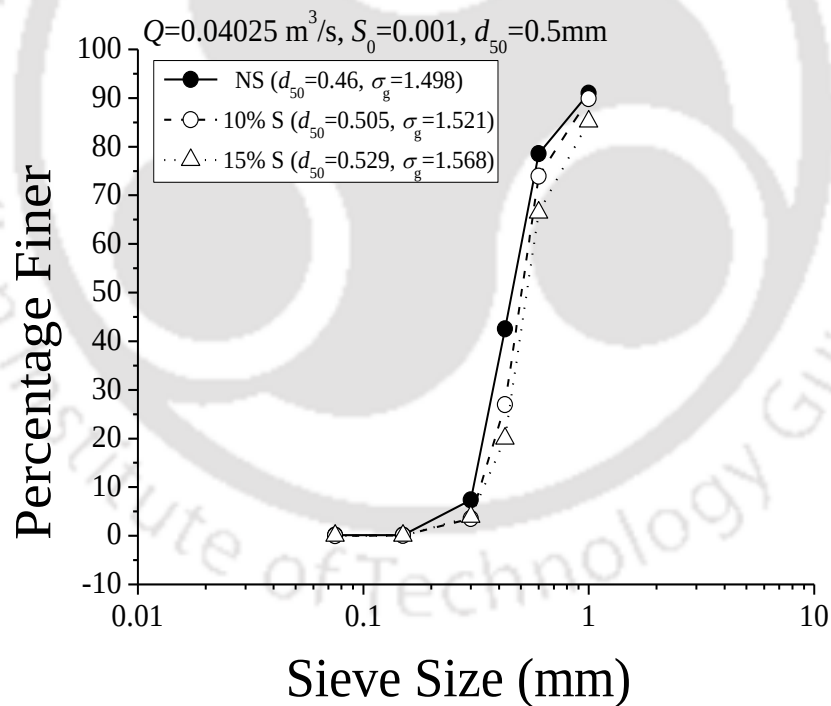


Figure 5.1: Particle size distribution curve of resultant bedload for no seepage and seepage flow

It was observed that the particle size distribution curve of collected sediment mixture (Figure 5.1) has been changed with and without seepage as compared to original particle size distribution

curve (Figure 2.1). The median particle size (d_{50}) and geometric standard deviation (σ_g) of bed load transport is increased in the presence of seepage as compared to no seepage runs, which suggest that non uniformity of collected material is increased with the increase of seepage percentage. The observed median particle size (d_{50}) and geometric standard deviation (σ_g) of bed load transport is further used for empirical formulation of bedload transport in the next section of the present chapter.

5.3 Empirical Formulation of Bedload Transport

Sediment particles resting on wall boundary in alluvial channel starts to move when the degree of hydrodynamic forces is greater than the threshold value. This mode of sediment transport change to bedload when main flow discharge is high implies that flows transporting sediment particles become suitably powerful. Movement of large quantity of sediment is occurring due to large difference between hydrodynamic force and resistive force which result in the formation of bedload. During experiments, the bed shear stress is higher at the upstream owing to degradation of the bed material at that section and this degraded material was deposited on the bed surface at the next section towards downstream which further moves towards downstream along the channel length in the form of bedload transport. A large quantity of degraded soil eroded from the beds is transported as bedload over the bed and the bed materials transport in such a way that the bed materials roll over one another. According to various researchers (**Myer Peter and Muller, 1948; Engeland and Henson, 1967; Ashida and Michiew, 1972; Englend and Fredsoe, 1976; Madsen, 1991; Nielsen, 1992; Fredsoe et al., 1992; Van Rijn, 1993; Nino and Garcia, 1994; Julien, 2002; Abrahams and Gao, 2006; Camenon et al., 2006**), bedload transport occurs when the Shields parameter is greater than its critical value. In the present work, the bedload transport was found during no seepage and seepage runs which pointed out that Shields parameter is greater than its threshold value during no seepage and seepage experiments. The bedload transport increase with seepage which eventually validates that the bed shear stress is higher with the downward seepage. In order to find the effect of seepage on the bedload transport, a proper dimensional study was made to produce an experiential relationship for the bedload transport at which seepage parameter is an explicit variable. The bedload transport depends on regression equations that are used to calculate the dependent parameters. The functional form of the bedload transport (q_b) for seepage affected channel is as:

$$q_b = f(P, L, h, Q_0, S_f, d, \rho_s, \rho, g, \nu, q_s) \quad (5.1)$$

Where P = wetted perimeter of the channel, L = seepage zone length, h = flow depth, Q_0 = channel discharge, S_f = friction/energy slope, d = representative size of sediment, ρ_s = density of sand, ρ = density of water, g = acceleration due to gravity, ν = kinematic viscosity and q_s = seepage discharge. Equation (5.1) can be written by transforming discharges into the characteristics velocity as follows:

$$q_b = f(h, u, S_f, d, \rho_s, \rho, g, \nu, V_s) \quad (5.2)$$

where u is the channel flow velocity and V_s is the seepage velocity calculated as:

$$V_s = \frac{q_s}{PL} \quad (5.3)$$

Since the parameters ρ_s / ρ and S_f are two of the factors controlling the movement of the bed materials in the channel bed, these two parameters may be combined into one representing the relative mobility of the sediment. By introducing the shear velocity $u_* = \sqrt{ghS_f}$ and bed shear stress $\tau = \rho u_*^2$, Equation (5.2) can be transformed in non-dimensional form as follows:

$$\frac{q_b}{\sqrt{\left(\left(\frac{\rho_s}{\rho}\right) - 1\right)gd^3}} = f\left(\frac{u}{\sqrt{gh}}, \frac{u_*d}{\nu}, \frac{V_sd}{\nu}\right) \quad (5.4)$$

$$\text{Or } \phi = f(F, R_*, R_s) \quad (5.5)$$

Where $\phi = \left(\frac{q_b}{\sqrt{\left(\left(\frac{\rho_s}{\rho}\right) - 1\right)gd^3}} \right)$ is non-dimensional bed load transport, we have tried to get an

empirical relation for the bedload transport by using three different parameters (Froude number ($F = \frac{u}{\sqrt{gh}}$), friction Reynolds number ($R_* = \frac{u_*d}{\nu}$) and seepage Reynolds number ($R_s = \frac{V_sd}{\nu}$)) as

independent parameters. The non-dimensional bedload transport ϕ was assumed to be proportional to the product of the three different Reynolds number.

$$\phi = a(F)^b (R_*)^c (e^{R_{es}})^f \quad (5.6)$$

The exponential trends provided to seepage Reynolds number is to fulfill the boundary condition of an impermeable bed (zero seepage rates). In this study, the sediment size d is calculated according to d_{50} for each set of data (**Mohtar et al., 2016**). According to **Mohtar et al. (2016)**, the use of d_{50} rather than employing the effective sediment sizes is sufficient to calculate the bedload discharge which necessarily improves the bedload prediction. The set of experimental data series observed in the laboratory flume was used to calculate the values of empirical coefficient a , b , c and exponents f . The resultant equation for the dimensionless bedload transport becomes:

$$\phi = 1446.94(F_r)^{7.828} (R_*)^{-0.264} (e^{R_s})^{0.466} \quad (5.7)$$

Figure 5.2 shows the prediction of dimensionless bedload transport by Equation (5.7) with the observed values. Figure 5.2 pointed out that around 90% of the observed values lies within $\pm 10\%$ deviation from the prediction. Results of comparisons of bedload-transport predictions in non-uniform bedload transport cases are based on different efficiency criteria: The efficiency criteria are defined as mathematical measures of how a model simulation fits the available observations (**Beven, 2001**). In general, many efficiency criteria contain a summation of the error term (difference between the simulated and the observed variable at each time step) normalized by a measure of the variability in the observations. To avoid the canceling of errors of opposite sign, the summation of the absolute or squared errors is often used for many efficiency criteria. Further, different efficiency criterion like Nash-Sutcliffe efficiency (E), Mean Squared Error (MSE) and Index of Agreement (I_d) may be used to provide more information on the systematic and dynamic errors present in the model simulation.

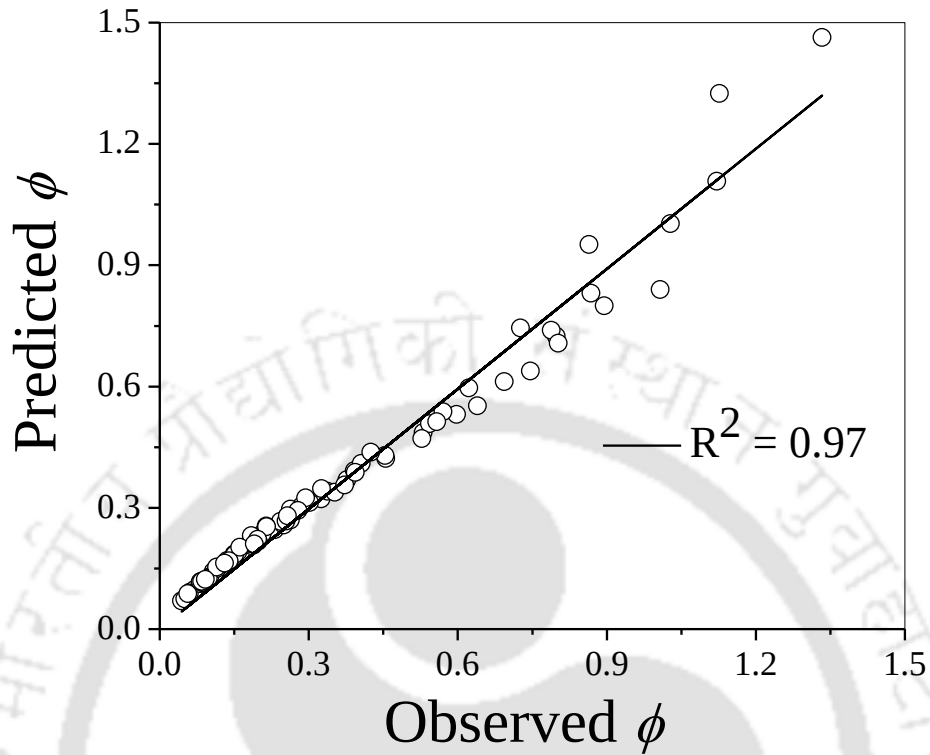


Figure 5.2: Predicted bedload transport by Equation (5.7) versus observed values

Coefficient of Determination (R^2)

The coefficient of determination R^2 can be expressed as the squared ratio between the covariance and the multiplied standard deviations of the observed and predicted values. Therefore it estimates the combined dispersion against the single dispersion of the observed and predicted series. The range of R^2 lies between 0 and 1.0 which describes how much of the observed dispersion is explained by the prediction. A value of zero means no correlation at all whereas a value of 1 means that the dispersion of the prediction is equal to that of the observation. The fact that only the dispersion is quantified is one of the major drawbacks of R^2 if it is considered alone. It is advisable that gradient of fitted regression and root mean squared errors (σ) should be given along with R^2 .

Nash-Sutcliffe efficiency (E)

The efficiency E proposed by **Nash and Sutcliffe (1970)** is defined as:

$$E = 1 - \frac{\sum_{i=1}^n (O_i - P_i)^2}{\sum_{i=1}^n (O_i - \bar{O})^2} \quad (5.8)$$

Where, O represents the observed and P represents the predicted values. The range of E lies between 1.0 (perfect fit) and $-\infty$. An efficiency of lower than zero indicates that the mean value of the observed time series would have been a better predictor than the model.

Mean Squared Error (MSE)

The mean squared error (MSE) of an observed value measures the average of the squares of the errors that is, the difference between the observed and what is predicted.

$$MSE = \frac{1}{n} \sum_{i=1}^n (O_i - P_i)^2 \quad (5.9)$$

Where, O represents the observed and P represents the predicted values. As, $MSE \rightarrow 0$, the difference between observed and predicted values is small.

Index of Agreement (I_d)

The index of agreement I_d was proposed by **Willmot (1981)** to overcome the insensitivity of E and R^2 to differences in the observed and predicted means and variances (**Legates and McCabe, 1999**). The index of agreement represents the ratio of the mean square error and the potential error (**Willmot, 1981**) and is defined as:

$$I_d = 1 - \frac{\sum_{i=1}^n (O_i - P_i)^2}{\sum_{i=1}^n (|P_i - \bar{O}| + |O_i - \bar{O}|)^2} \quad (5.10)$$

The potential error in the denominator represents the largest value that the squared difference of each pair can attain. The range of I_d is similar to that of R^2 and lies between 0 (no correlation) and 1.0 (perfect fit).

The performance of proposed Equation (5.7) was found to be very satisfactory with the observed values and the coefficient of multiple determinations (R^2) observed to be 0.97. We have used some other efficiency criterion such as Nash-Sutcliffe efficiency (E), Mean Squared Error (MSE) and Index of Agreement (I_d) in order to check the performance of Equation (5.7). For the present experimental data, we have observed the value of Nash-Sutcliffe efficiency (E), Mean Squared Error (MSE) and Index of Agreement (I_d) is 0.97, 0.0023 and 0.99 respectively which indicate that the predicted equation is well fitted with the experimental results.

5.4 Performance of Bedload Transport Formula

The exact measurement of bedload transport is very difficult. In the present scenario, no universal bedload transport equation exists (**Reid *et al.*, 1996**). Regardless of the available bedload transport equations, there is still considerable argument over their efficiency. In order to estimate the performance of proposed bedload equation, we have compared the observed values of bedload with the existing bedload transport formulas. Using data collected from present experiments, it is planned to determine which method best predicts bedload transport. Thus, for this study, twelve bedload empirical equations were used to evaluate the sediment transport rate based on their suitability for sand-bed rivers, the hydraulic conditions and to examine the efficiency of the recently developed bedload equations. The predicted bedload transport rates were compared to the measured values at each set of experiments. Twelve formulae were considered: **Myer Peter and Muller (1948)**, **Engeland and Henson (1967)**, **Ashida and Michiew (1972)**, **Englend Fredsoe (1976)**, **Madsen (1991)**, **Nielsen (1992)**, **Fredsoe and Deigaarrd (1992)**, **Van Rijn (1993)**, **Nino and Garcia (1994)**, **Julien (2002)**, **Abrahams and Gao (2006)** and **Camenon *et al.* (2006)**.

Table 5.1 lists the selected transport formulas analyzed in the present work where φ denotes the calculated non-dimensional bedload transport and u is the mean velocity. The initiation of motion of bed particles will occur when the Shield's parameter θ is higher than critical value, θ_c , which is defined as:

$$\left. \begin{aligned} \theta &= \frac{\tau}{(\rho_s - \rho)gd_{50}} \\ \theta_c &= \frac{\tau_c}{(\rho_s - \rho)gd_{50}} \end{aligned} \right\} \quad (5.11)$$

Where, τ is the shear stress and τ_c is the critical shear stress. Here, the value of Shields' critical shear stress θ_c is determined using equation given by **Paphitis (2001)**.

$$\theta_c = \frac{0.273}{1+1.2d_*} + 0.046(1-0.576e^{-0.05d_*}) \quad (5.12)$$

Where $d_* = d_{50} \left[\frac{g \left(\frac{\rho_s}{\rho} - 1 \right)}{\nu^2} \right]^{1/3}$ **(Yalin, 1977)**

Experimental data were applied in various bedload equations. We have investigated the best comparison curves (Φ observed versus Φ calculated) given by respective researchers with data observed in the present study for both no seepage and seepage runs. The experimental data displays a slightly scatter and could be described as a set rather than well-defined curves. For no seepage runs, data indicates that formulae of **Myer Peter and Muller (1948)** has predicted satisfactorily as shown in Figure 5.3 while for seepage runs shown in Figure 5.4, none of the equation performs well. The performance of each of the formulae was also assessed statistically, where the comparisons of the predicted and measured bedload discharge are given in Tables 5.2 for no seepage runs. Results of comparisons of bedload-transport predictions in no seepage case are based on efficiency criterion. Table 5.2 shows that the Equation of **Myer Peter and Muller (1948)** support the best prediction with the highest value of R^2 , E , I_d and with the lowest value of MSE . On the basis of statistical analysis, the Equation of **Myer Peter and Muller (1948)** best described the observed transport rates, typically within an order of magnitude of the observed values. Other formulae generally did not perform well. These comparisons pointed that the evaluation for bedload transport is the standard process for learning the sediment transport over the channel bed surface.

Table 5.1: Sediment transport models selected for comparison.

Sl. No.	Authors	Equation
1.	Myer Peter and Muller (1948)	$\phi = 8(\theta - \theta_c)^{1.5}$
2.	Engeland and Henson (1967)	$\phi = 0.05 \left(\frac{u}{u_*} \right)^2 (\theta)^{2.5}$
3.	Ashida and Michiew (1972)	$\phi = 17(\theta - \theta_c)(\theta^{0.5} - \theta_c^{0.5})$
4.	Englend Fredsoe (1976)	$\phi = 18.74(\theta - \theta_c)(\theta^{0.5} - 0.7\theta_c^{0.5})$
5.	Madsen (1991)	$\phi = (\theta - \theta_c)(\theta^{0.5} - 0.7\theta_c^{0.5})$
6.	Nielsen (1992)	$\phi = 12\theta^{0.5}(\theta - \theta_c)$
7.	Fredsoe <i>et al.</i> (1992)	$\phi = \frac{30}{\pi}(\theta - \theta_c)(\theta^{0.5} - 0.7\theta_c^{0.5})$
8.	Van Rijn (1993)	$\phi = \frac{0.053}{d_*^{0.3}} \left(\frac{\theta}{\theta_c} - 1 \right)^{2.1}$
9.	Nino and Garcia (1994)	$\phi = 52.17(\theta - \theta_c)(\theta^{0.5} - 0.7\theta_c^{0.5})$
10.	Julien (2002)	$\phi = \frac{18\sqrt{g}d^{1.5}\theta^2}{\sqrt{g\left(\frac{\rho_s}{\rho} - 1\right)}d^3}$
11.	Abrahams and Gao (2006)	$\phi = \theta^{1.5} \left(1 - \frac{\theta_c}{\theta} \right)^{3.4} \frac{u}{u_*}$
12.	Camenon <i>et al.</i> (2006).	$\phi = 12\theta^{0.5} \exp\left(-4.5 \frac{\theta_c}{\theta}\right)$

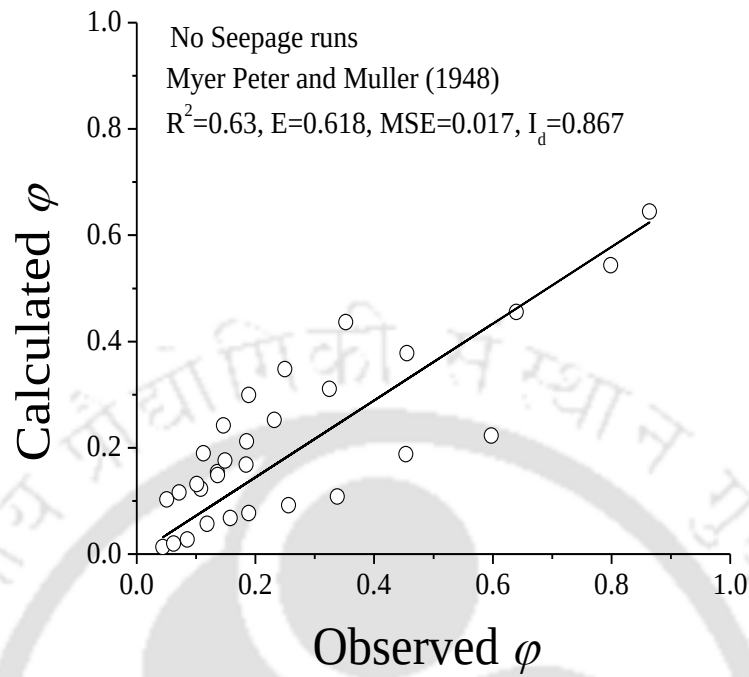


Figure 5.3: Comparison between calculated and observed bedload (Myer Peter and Muller, 1948) for no seepage runs

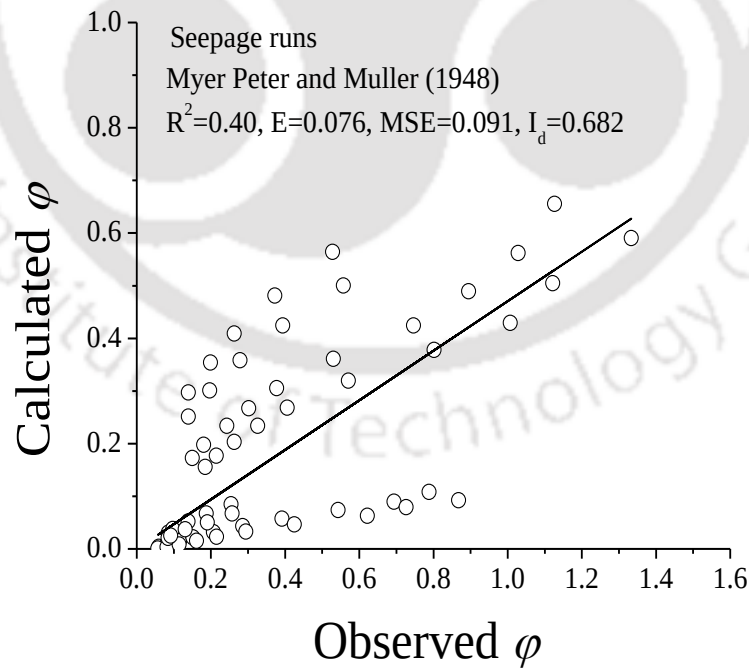


Figure 5.4: Comparison between calculated and observed bedload (Myer Peter and Muller, 1948) for seepage runs

Table 5.2: Comparison of different efficiency criteria for predicting bedload rate (no seepage)

Sl. No.	Authors	R^2	E	MSE	I_d
1.	Myer Peter and Muller (1948)	0.64	0.6186	0.0175	0.8677
2.	Engeland and Henson (1967)	0.48	0.3247	0.0570	0.8011
3.	Ashida and Michiew (1972)	0.47	-1.1367	0.3355	0.6815
4.	Englend Fredsoe (1976)	0.59	-0.1749	0.1384	0.7602
5.	Madsen (1991)	0.45	0.2946	0.0371	0.8077
6.	Nielsen (1992)	0.53	-4.6365	0.6416	0.5307
7.	Fredsoe <i>et al.</i> (1992)	0.57	0.0683	0.0429	0.6982
8.	Van Rijn (1993)	0.53	0.3355	0.0306	0.8048
9.	Nino and Garcia (1994)	0.53	-1.1093	0.0972	0.4587
10.	Julien (2002)	0.52	0.3734	0.0510	0.8037
11.	Abrahams and Gao (2006)	0.53	0.2566	0.0435	0.8072
12.	Camenon <i>et al.</i> (2006).	0.57	-0.2613	0.1164	0.6691

Similar performance study of each method (Table 5.3) has been done for seepage runs. This study also looked at the performance of the formulae with downward seepage and observed that none of the equation performs well. In the case of seepage runs, the efficiency criterions of all bedload transport equations are less as compared to no seepage runs. The results suggest that equations conforming to no seepage runs are not suitable for seepage case. However, present work establishes the bedload transport equation for an alluvial channel subjected to downward seepage which is based on several flow parameters including the seepage discharge.

Table 5.3: Comparison of different efficiency criteria for predicting bedload rate (Seepage)

Sl. No.	Authors	R^2	E	MSE	I_d
1.	Myer Peter and Muller (1948)	0.40	0.0764	0.0907	0.6822
2.	Engeland and Henson (1967)	0.37	0.3052	0.0683	0.8131
3.	Ashida and Michiew (1972)	0.36	-3.7071	0.4624	0.5867
4.	Englend Fredsoe (1976)	0.48	-0.2825	0.1260	0.7905
5.	Madsen (1991)	0.35	0.2770	0.0710	0.7453
6.	Nielsen (1992)	0.41	-8.3623	0.9197	0.4934
7.	Fredsoe <i>et al.</i> (1992)	0.46	-0.4799	0.1454	0.5499
8.	Van Rijn (1993)	0.41	-0.0204	0.1002	0.6479
9.	Nino and Garcia (1994)	0.41	-1.3162	0.2275	0.4567
10.	Julien (2002)	0.41	0.2664	0.0721	0.8153
11.	Abrahams and Gao (2006)	0.43	0.1872	0.0798	0.7522
12.	Camenon <i>et al.</i> (2006).	0.33	-0.3389	0.1315	0.6558

5.5 Temporal Variation of Bedload Transport

5.5.1 Overview

The movement of heterogeneous sediment mixture in turbulent flow has established high attention to the river engineers and sedimentologists in terms of basic problems related to the process of grain-sorting and sediment bed migration. Investigations have been carried out on the transporting bed material over the bed surface to calculate the bed migration for a large range of flow discharges and sediment inputs (**Best, 2005; McElroy and Mohrig, 2009**). Sediment transport rate is one of the crucial parameter for river engineering, channel hydrodynamics and fluvial hydraulics (**Recking, 2009**). Previous studies (**Chiew, 2004; Guo, 2011; Yang *et al.*, 2013**) suggest that the rate of sediment transport for the impermeable channel bed increases with time and finally attained an equilibrium condition. **Yang *et al.* (2013)** experimentally investigated the nature of sediment transport in a compound channel with impermeable

boundary, and observed that the bedload transport increases with time, achieve the maximum value then again gradually decrease to zero. Also they found that the bedload transport rate varied is a function of time in a log form. **Liu et al. (2014)** studied the behavior of sediment transport in the presence of downward seepage and developed the theoretical expression for the bedload transport rate by incorporating the seepage parameter as an explicit variable and the developed theoretical expression are validated with the experimental results.

The drawback of previous study (**Liu et al., 2014**) is that their work is restricted to the channel with local seepage (seepage zone length of only 2m out of flume length 30m), which does not satisfy the actual field condition as well as the natural rivers. The seepage zone length is important in the study of the time dependent sediment transport because sediment transport rate increases with seepage zone length (**Cao and Chiew, 2014**). Thus, the aim of the present section is to explore the time dependent sediment transport in a non- uniform sand bed channel with the whole permeable bed and to modify the empirical expression given by **Liu et al. (2014)** in order to relate the sediment transport rate with time.

5.5.2 Time Rate of Bedload Transport

Experiments were carried out to investigate temporal rate of bedload transport with two tests (no seepage flow) and downward seepage rates. Here, results have been presented for no seepage discharge of $0.0429 \text{ m}^3/\text{s}$ (test1) and $0.04065 \text{ m}^3/\text{s}$ (test2) and bed slope 0.0005 to avoid duplicity and redundancy in the findings. Figure 5.5 and 5.6 shows the measured bedload transport rates with time for both tests. As shown in Figure 5.5 and 5.6, the bedload transport rate increases quickly with time for approximately first 6 hrs for test1 and 1 hr for test2. After it achieved the maximum value, again it decreases with time. When the time exceeds around 10 hrs for test1 and 5 hrs for test2, the decreased trend of sediment transport rate slowed down till it attain the horizontal slope. This result recommends that the rate of bedload transport slowly decreases with time till it achieved the stable criteria, while the erosion depth remains same with time. We can say that the trend show a substantial decrease in q_b together with a discrete reduction in dq_b/dt . The results are in good agreement with impermeable bed alluvial channel (**Yang et al., 2013**).

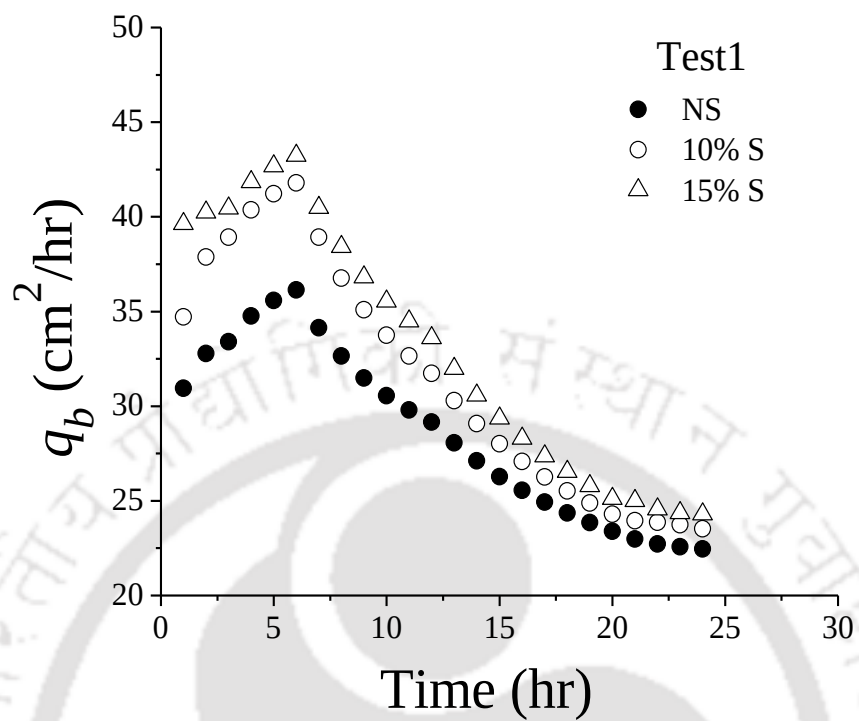


Figure 5.5: Bedload transport rate against time for test 1

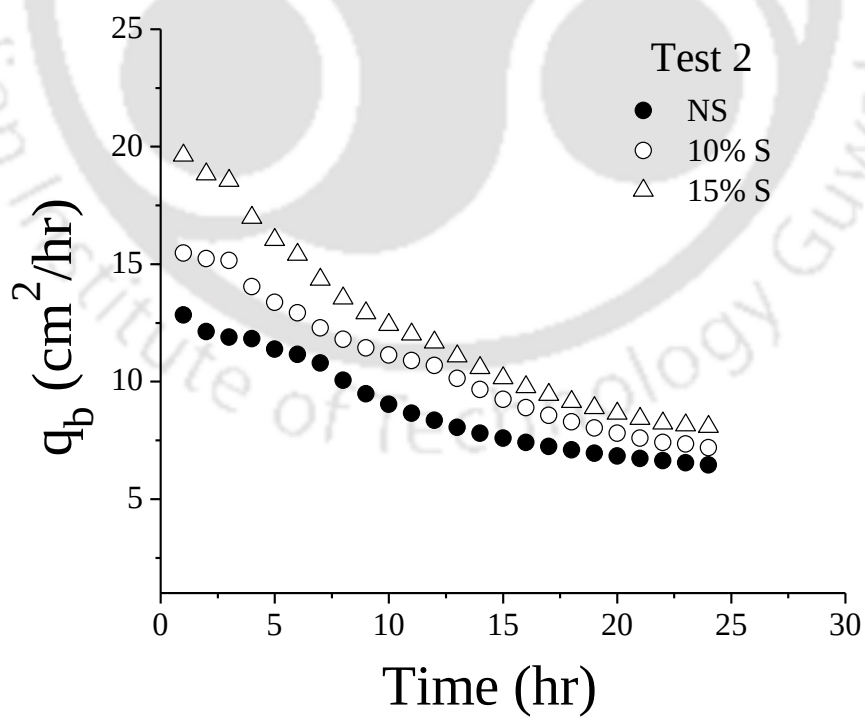


Figure 5.6: Bedload transport rate against time for test 2

To examine the temporal changes of falling limb of q_b , Figures 5.5 and 5.6 are re-plotted with the peak-value offset t_{off} (Equation 5.16) as shown in Figure 5.7 and 5.8. As shown in Figure 5.7 and 5.8, initially, the sediment transport rate increased with time and attains the maximum, and then decreased gradually. This happened due to the following reason. After the experiments start, the bed materials start to move in the channel bed with the presence of no seepage and downward seepage. **Chen and Chiew (2004)** proposed that the non-uniformity of bed shear stress is obtained over the entire length of the channel and has a higher value at the upstream zone of the channel length. Therefore, the bedload transport rate for both no seepage and seepage runs does not follow the uniformity, and has a maximum value at the upstream zone of the channel length. The higher rate of bed material was obtained at the upstream zone of no seepage and seepage flow which gradually move out of the test section along with time which further leads to gain in the bedload transport rate. This non uniformity of sediment transport rate results to the formation of rising limb shown in Figure 5.7 and 5.8.

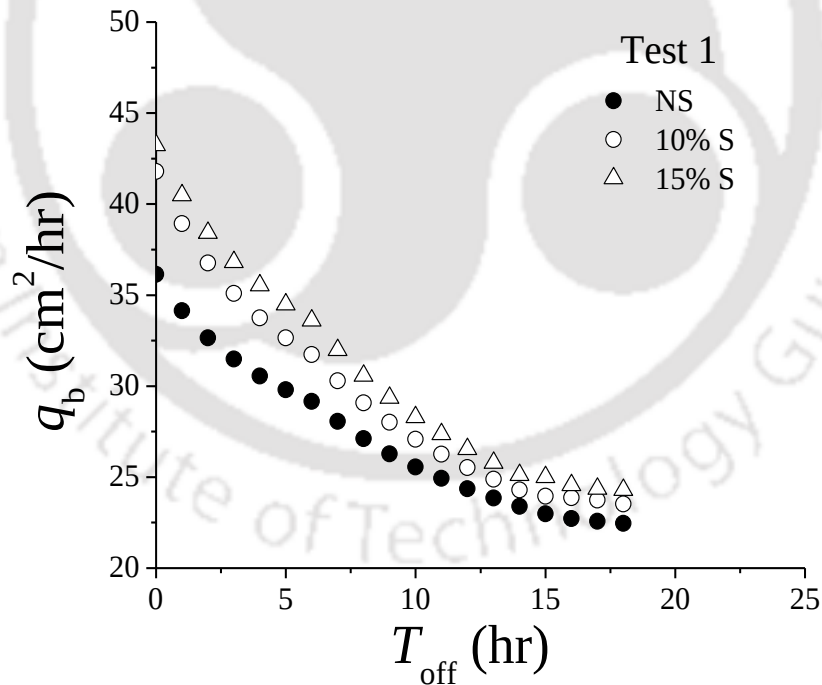


Figure 5.7: Sediment transport rate against time with peak value offset for test 1

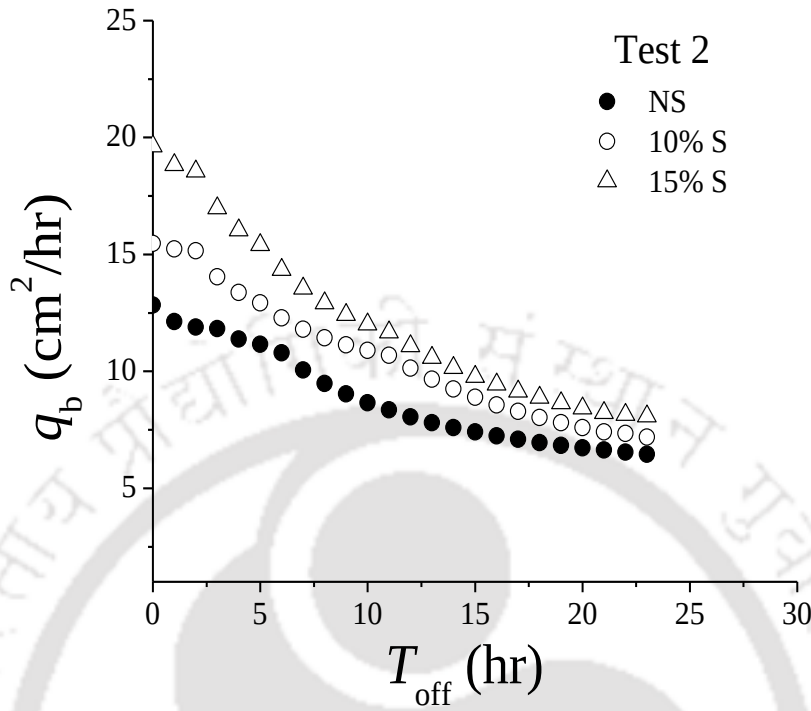


Figure 5.8: Sediment transport rate against time with peak value offset for test 2

As the upstream bed materials of the channel length progressed to the downstream end, the peak value of the bedload transport rate was obtained. Higher bed material transport rate at the upstream end of the channel, causes the erosion of the channel in that particular section. As the movement of bed materials continued, leading to an increase in bed erosion causes increase in the flow depth and corresponding decrease in the flow velocity. This happened because the tail gate at the downstream end of the flume was not disturbed in all experiments. **Cheng and Chiew (1998b)** proposed that this increased flow depth and decreased mean flow velocity results in the reduction of bed shear stress. Therefore, the bedload transport rate decreased with time after attaining the peak value.

The relationship between the reduction rate of sediment transport and the sediment transport rate are defined (**Liu et al., 2014**)

$$\frac{dY}{dT} = aY^{1-\frac{1}{b}} \quad (5.13)$$

Where a is the fitting parameter and b is the exponent. Y and T represent the dimensionless sediment transport rate and the dimensionless time which are expressed as follows:

$$Y = \frac{q_b}{q_{b-\text{peak}}} \quad (5.14)$$

$$T = \frac{t_{\text{off}} V_s}{L} \quad (5.15)$$

Where q_b is the volumetric sediment transport rate per unit width, $q_{b-\text{peak}}$ is the peak value of volumetric sediment transport rate per unit width. The term t_{off} in Equation (5.15) represents the peak-value offset given by

$$t_{\text{off}} = t - t_{\text{peak}} \quad (5.16)$$

Where t is time and t_{peak} is the time at which sediment transport rate reach the peak value. In Equation (5.15), V_s is the seepage velocity.

$$V_s = \frac{q_s}{p_s L} \quad (5.17)$$

Where q_s is the seepage discharge, p_s is wetted perimeter and L is seepage zone length.

The relationship between Y and T is obtained by rearranging Equation (5.13) and integrating both sides

$$Y = \left(1 + \frac{a}{b} T\right)^b \quad (5.18)$$

The experimental data sets are used to examine Equation (5.13), the central difference method are used in order to calculate the values of dY/dT between successive values of Y . Figure 5.8 display the profiles of dY/dT against Y . Fitting Equation (5.13) with the least-square method results in $b = -0.54$ and $a = -0.98$ with the coefficient of determination $R^2 = 0.93$. The resulting equation becomes:

$$\frac{dY}{dT} = -0.98Y^{2.84} \quad (5.19)$$

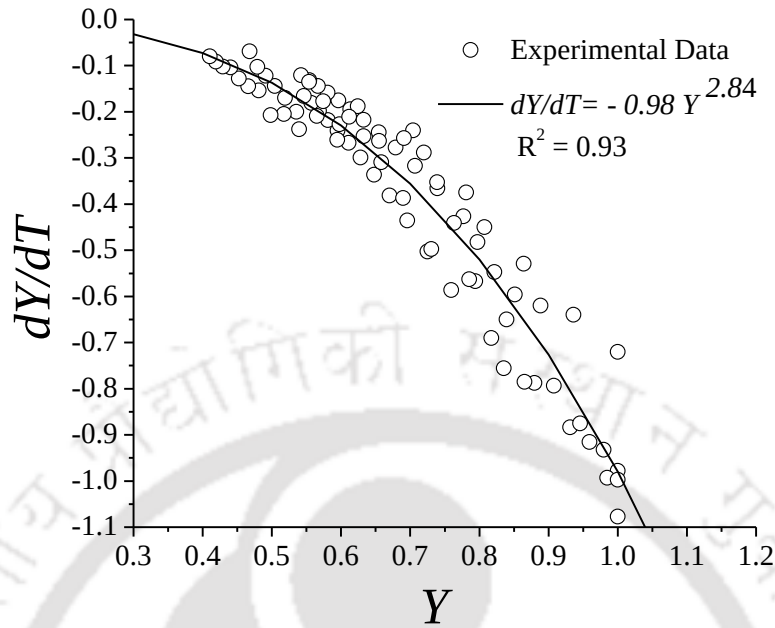


Figure 5.9: Test of hypothesis on rate of sediment transport

The obtained equation shown in Figure 5.9 agrees well with the experimental data. The scatter of data is mainly due to the large time steps during experimentation. It is clear from Figure 5.9 that the reduction rate of sediment transport (dY/dT) is large when the sediment transport rate start to decrease ($Y \rightarrow 0$), leading to rapid decrease in the bedload transport rate; and dY/dT approaches zero as the channel tends to the equilibrium state ($Y \rightarrow 1$), corresponding to the lower reduction in the bedload transport rate.

Putting the values of a and b in Equation (5.18) which becomes

$$Y = (1 + 1.81T)^{-0.54} \quad (5.20)$$

Figure 5.10 compares the Equation (5.20) with the experimental results. It is noticeable from Figure 5.10 that the bedload transport rate achieved the maximum value when $T \rightarrow 0$ and decreases continuously and when $T \rightarrow \infty$, the channel becomes the stable condition. It is notable that all the data for no seepage and seepage runs agreed well with Equation (5.20), indicating that the modified Equations (5.19) and (5.20) predicts well the temporal variation of the bedload transport rate subjected to downward seepage.

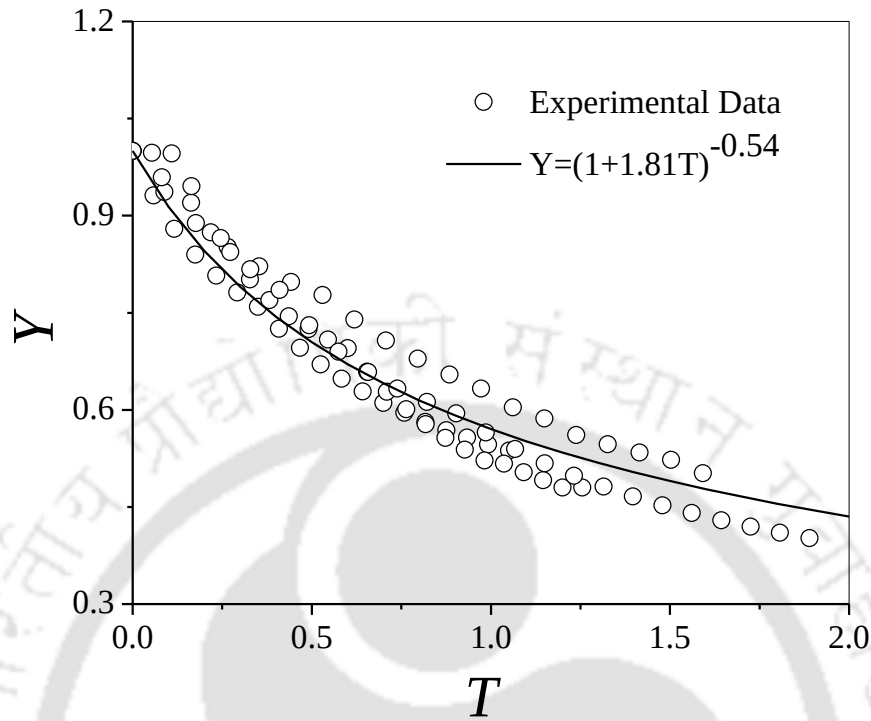


Figure 5.10: Test of relationship $Y(T)$

5.6 Analysis of Bedload Dynamic Characteristics

In alluvial channels, celerity shows the progradation of developing bedload along the flow. Here, dynamic characteristics of bedload are signified by their celerity. Alluvial bedload show a substantial variability in propagation velocity over the range of different spatial and temporal scales. To understand the effect of no seepage and seepage flow on this phenomenon, wavelet analysis has been carried out on the bed elevation series at different length scales and scale dependent celerity of bedload has been evaluated under no seepage and seepage scenarios. Here, results have been presented for no seepage discharge of $0.0429 \text{ m}^3/\text{s}$ and bed slope 0.0005 to avoid duplicity and redundancy in the findings. Here, bed elevation series were recorded at time intervals after 12 hrs of no seepage and seepage run using Ultrasonic Ranging System (Figure 5.11).

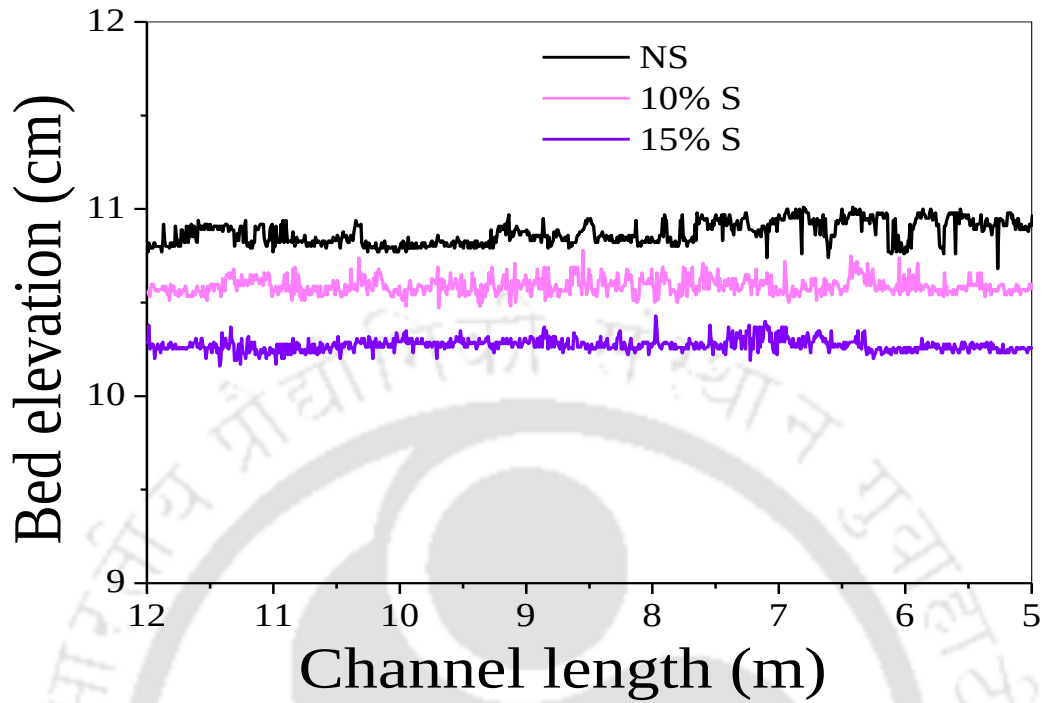


Figure 5.11: Bed elevation series for no seepage and seepage runs

5.6.1 Wavelet Cross-Correlation

Wavelet is a proficient method in order to analyse the scale to scale correlation of bed elevation series (**Kumar and Foufoula-Georgiou, 1997; Venugopal *et al.*, 2006**). It decomposes a signal into multiple signals and each signal represents the variability (energy) at a particular scale. In this method, Mexican hat wavelet was used, which is known to have optimal localization in both time and frequency domain (**Singh *et al.*, 2009**). At a particular scale, wavelet coefficients signify the fluctuations in the bed elevation series. Bed elevation series at two different times is decomposed to the same length scale using wavelet transform and cross-correlation analysis is performed on the obtained sets of wavelet coefficients to obtain celerity of bedload. Wavelet analysis has also been discussed in detail by **Singh *et al.* (2011)**. In the present study, the data recorded was spatially varied with 7.8 mm resolution at different time spans of the experimental run. Therefore, we could able to compute the length scale dependent celerity of the bedload. By following this approach, we can propose to eliminate the error owing to the incapability of the cross-correlation method based on time series data.

In this method, wavelet coefficients of bed elevation profiles at a particular scale were considered as bed elevation fluctuations. Fluctuations (wavelet coefficients) of bed elevations at different length scales were computed using the Mexican hat wavelet as the mother wavelet, which was the second order derivative of the Gaussian function. Coefficients, at the location b and scale a , were obtained as:

$$WC_f(a, b) = \frac{1}{\sqrt{a}} \int_R \psi\left(\frac{x-b}{a}\right) f(x) dx \quad (5.21)$$

Where $f(x)$ is the spatially varying bed elevation series, a is defined as the scaling parameter, b is defined as the location parameter, and R is the set of real number. The factor $1/\sqrt{a}$ is scaling factor of mother wave as discussed in earlier studies (**Mallat, 1998; Kumar and Foufoula Georgiou, 1997**). Mexican hat wavelet was chosen because of the resemblance of wavelet coefficients with the original bed elevation fluctuations. Length scales from 25 mm to 250 mm were chosen and the elevation series were transformed with a Mexican hat wavelet at those particular scales. The reason behind choosing 25 mm as the smallest characteristic length scale was the limitation of our sampled dataset to capture sediment transport below a characteristic length scale of 7.8 mm. This length scale was ensured to maintain reasonable accuracy for calculating the bedload celerity.

Wavelet coefficients represent bed elevation fluctuations at different characteristic length scales. Therefore, we have discretized signal into a range of length scales instead of time scales. The wavelets cross covariance between two signals $f_1(x)$ and $f_2(x)$ can be calculated as:

$$WCC_{f_1 f_2}(a, \Delta x) = \int_{-\infty}^{+\infty} W_{f_1}(a, b) W_{f_2}(a, b + \Delta x) db \quad (5.22)$$

Where $W_{f_1}(a, b)$ and $W_{f_2}(a, b + \Delta x)$ are the wavelet coefficients of two bed elevation series $f_1(x)$ and $f_2(x)$, respectively, at scale a and two adjacent locations b and $b + \Delta x$, respectively. The wavelet cross correlation was calculated by normalizing it with the variance of the signals. For a particular scale a , the lag corresponding to the maximum cross-correlation coefficient of two series of wavelet coefficients, was obtained from the wavelet transformation of two consecutive bed elevation series. Here, one lag is equal to 7.8 mm as our datasets were sampled at a

resolution of 7.8 mm. Thus, the migration velocity of bedload (bedload celerity at particular scale) was computed by dividing the lag by the time interval between the two consecutive datasets.

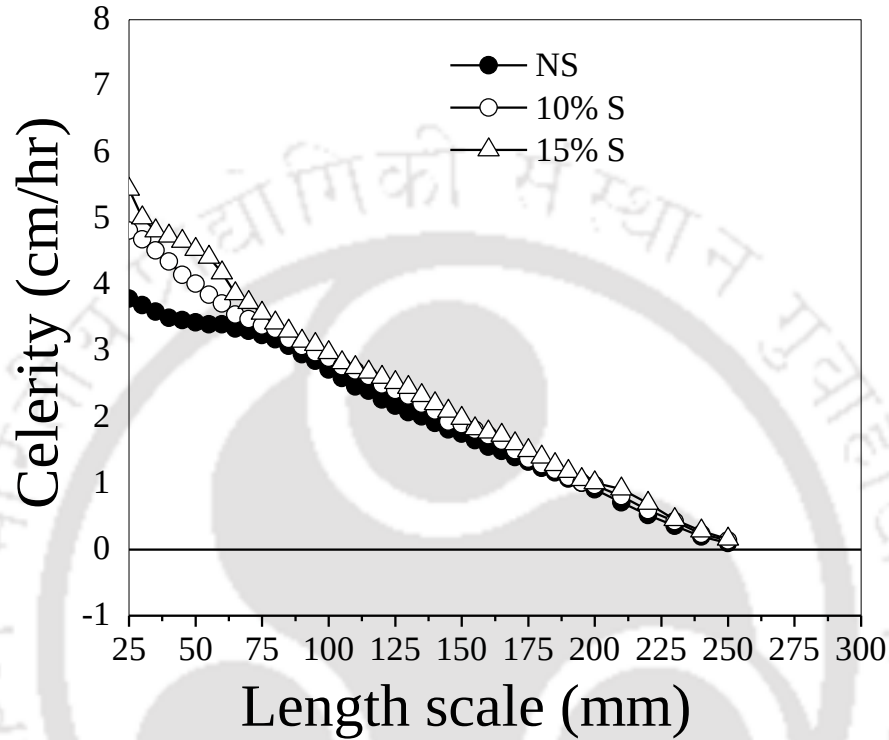


Figure 5.12: Celerity of sediment transport for no seepage and seepage runs

Celerity of the bedload, ranging from a length scale of 25 mm to 250 mm, was calculated using wavelet cross-correlation as shown in Figure 5.12. Results show an interesting observation regarding the celerity of bedload of the same size. Celerity of bedload having the same length scale was observed to be greater in the case of 15% seepage than 10% seepage experiment. It has been clearly shown that celerity of the bedload increases after increasing the percentage of seepage. Figure 5.12 also shows that the constancy achieved to the celerity of the larger length scales of bedload. In this regard, bedload size and shape govern the resistance to flow, which inhibits the further sediment transport at large scale. Observation suggests that the channel may have attained equilibrium in terms of sediment transport and length scale of bedload irrespective of the seepage percentages. The statistics of the bedload celerity is given in Table 5.4.

Table 5.4: Statistics of the bedload celerity

Length Scale (mm)	No seepage flow (NS)		Seepage flow (10% S)		Seepage flow (15% S)	
	Celerity (cm/hr)	Mean	Celerity (cm/hr)	Mean	Celerity (cm/hr)	Mean
25	3.78641	2.17	4.81605	2.42	5.43842	2.58
30	3.68932		4.68227		5.00493	
35	3.59223		4.51505		4.80788	
40	3.49515		4.34783		4.72906	
45	3.46278		4.14716		4.65025	
50	3.43042		4.01338		4.53202	
55	3.39806		3.84615		4.41379	
60	3.39806		3.71237		4.17734	
65	3.33333		3.54515		3.86207	
70	3.30097		3.47793		3.72857	
75	3.23625		3.37893		3.56799	
80	3.17152		3.32809		3.41922	
85	3.07443		3.17726		3.29506	
90	2.94498		3.06822		3.14533	
95	2.8479		2.97592		3.09557	
100	2.71845		2.89198		2.97093	
105	2.589		2.76655		2.82117	
110	2.45955		2.69933		2.74631	
115	2.39482		2.61572		2.6743	
120	2.26537		2.4903		2.59655	
125	2.16828		2.40669		2.52168	
130	2.0712		2.32307		2.4468	
135	2.00647		2.19766		2.32216	
140	1.90939		2.09558		2.19747	
145	1.8123		1.93044		2.08473	
150	1.74757		1.88863		1.9778	
155	1.65049		1.80501		1.82266	
160	1.5534		1.7214		1.77291	
165	1.48867		1.63779		1.72315	
170	1.39159		1.49599		1.59734	
175	1.32686		1.37056		1.48985	
180	1.22977		1.28696		1.39488	
185	1.16505		1.20452		1.28241	
190	1.06796		1.09615		1.17744	
195	1.00324		1.00878		1.05246	
200	0.90615		0.97107		1.00375	
210	0.71197		0.80167		0.9103	
220	0.5178		0.59264		0.6965	
230	0.35599		0.42541		0.45	
240	0.19417		0.234		0.276	
250	0.09709		0.134		0.156	

5.7 Hilbert-Huang Transform (HHT)

Hilbert-Huang transform (HHT) provides the distribution of physical parameters of sediment transport at different time scales. This transformation uses two steps: first, the Empirical Mode Decomposition (EMD) decomposes the datasets into several Intrinsic Mode Functions (IMF) and second, Hilbert Spectral Analysis (HSA) extracts instantaneous frequency data from the signal. The traditional signal analysis methods use either time domain or frequency domain analysis. In time domain analysis, the amplitude of the signal is represented as a function of time whereas in frequency domain analysis, it is represented as a function of frequency. But, in time domain analysis we completely lose the frequency resolution and frequency domain representation loses time resolution. With the advancement in mathematics, some powerful and effective time-frequency analysis methods have been developed. Among them, some predominant and widely used methods are short time Fourier transformation, wavelet analysis, bilinear time-frequency distribution, and modified Wigner distribution function etc. But most of these methods use a predefined function, which is dilated with a scaling factor and is slid through the signal to find out how well the actual signal matches with that predefined function. This somehow puts a limit to the applicability of these methods to any kind of datasets. To overcome this drawback, HHT can be used. Also, the EMD may be used to find time scales in the velocity signal. In this process, HHT decomposes into different modes, which are intrinsic to the function. These modes are known as IMF. Each IMF satisfies two conditions: first, the number of extrema and the number of zero crossings are equal or may differ at most by one and secondly, at any point, the mean value of the envelope, defined by the local extrema is zero. In contrary to the fast Fourier transformation, an IMF can have varying amplitude and frequency along the time axis. First, one IMF is extracted, and then it is subtracted from the original profile of sand waves. Then the next IMF is again extracted and it is subtracted from the previous resulting signal. In this way, the sifting process carries on until the standard deviation between two consecutive IMFs comes between 0.25-0.3. Each of the extracted IMFs is used to form an analytical signal, which is represented by a real part and an imaginary part. The real part is the IMF itself and the imaginary part is the IMF convoluted with $1/\pi t$. Now, the original signal can be expressed as a linear combination of the real parts of these analytical functions and a residue term. Let, $z_i(t)$ be the analytical function constructed from the i^{th} IMF then $z_i(t)$ can be expressed as:

$$z_i(t) = a_i(t)e^{j\theta_i(t)} = a_i(t)e^{j\int \omega_i(t)dt} \quad (5.23)$$

where $a_i(t)$ is defined as the local magnitude of the original signal and $\theta_i(t)$ is the instantaneous or local phase of the original signal. The instantaneous frequency $\omega(t)$ is represented as the derivative of the local phase (**Franceschini and Tsai, 2010**). We have used the EMD package (**Rilling et al., 2007**) to perform the HSA.

5.7.1 Hilbert Spectral Analysis (HSA)

In order to quantify the wide disparity in physical and dynamic characteristics of sediment transport among various ranges of scales, HSA helps to provide with a clearer and meaningful insight. Here, results have been presented for no seepage discharge of 0.0429 m³/s and seepage discharge of 10% of no seepage discharge to avoid duplicity and redundancy in the findings. The velocity time series data at near bed ($z=0.03h$) of no seepage and seepage run is shown in Figure 5.13 and 5.14. Time scale of velocity signal is characterized with the help of HSA at different time intervals. In the first step, EMD decomposes the velocity time series into various IMFs. The IMFs reveal the presence of multi scale signals in the filtered velocity time series of flow subjected to no seepage (Figure 5.15). It is seen from Figure 5.15 that the first IMF1 is descriptive of extremely fluctuating components and frequency of fluctuation is reduced with the increase of IMF. Thus, IMF13 represents the signals of lowest fluctuating values. The last diagram represents the residual, which displays the major part in the time series datasets after the noise removal. It is obtained from the residual function that the predominant trend is decreasing with the increase of time scale. Similar observation is found in the flow velocity time series datasets subjected to flow associated with seepage. It should be noted that the lowest measurable time scale is at least equal to the sampling time interval (0.01 sec in this study), that may be suitable for data collection and acquisition.

In the second step, HSA was used to get more insight into time-frequency-energy of the signals by using Hilbert spectrum. Figure 5.16-5.17 shows the Hilbert spectrum with different colors and each color denotes a specific energy level over time (x-axis) and over the frequency (y-axis). Darker points represent higher energy levels. The frequency corresponding to the maximum number of high energy level points signifies the dominant time scale. The scattered points near

high frequency correspond to different individual time scales. The equation used to evaluate dominant time scales: Dominant time scale = (1/dominant normalized frequency) x measuring time interval. It should be noticed that most energy points are located in the dominant relative frequency (y-axis) and the dominant time scale (x-axis) is not constant but fluctuates with time.

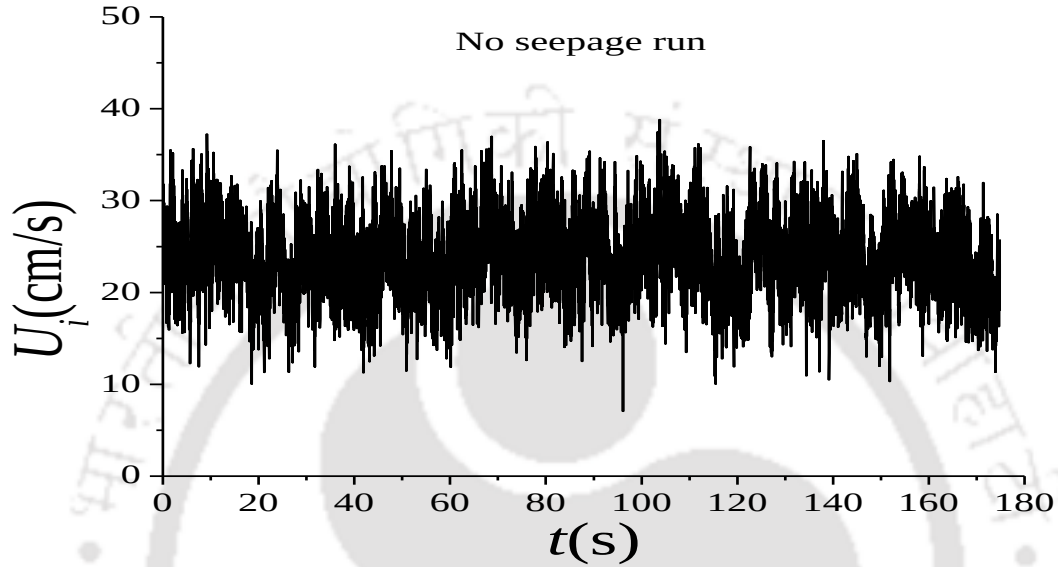


Figure 5.13: Original velocity time series datasets for no seepage run

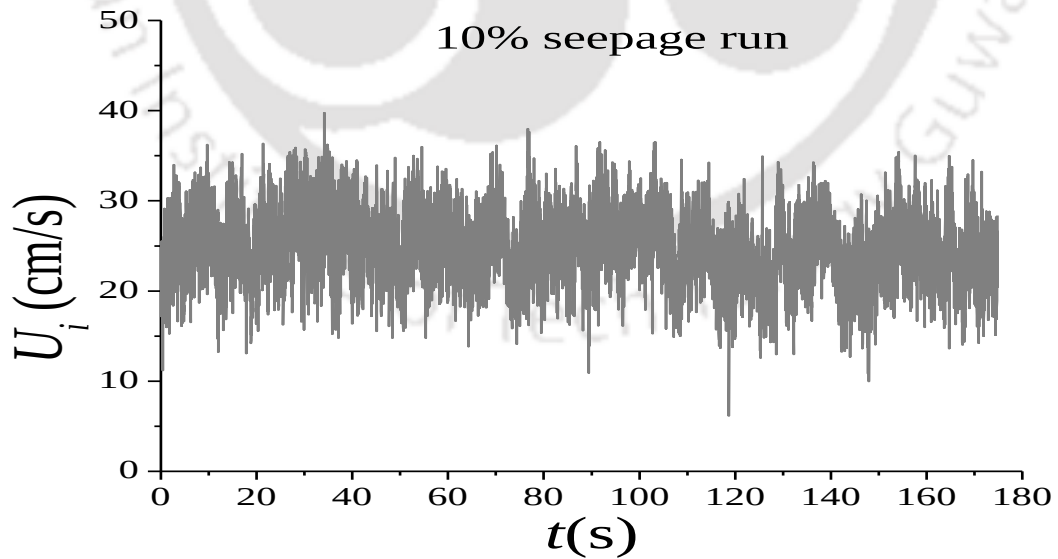
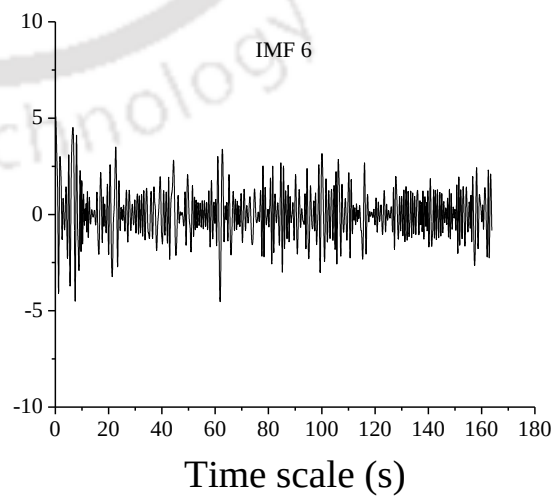
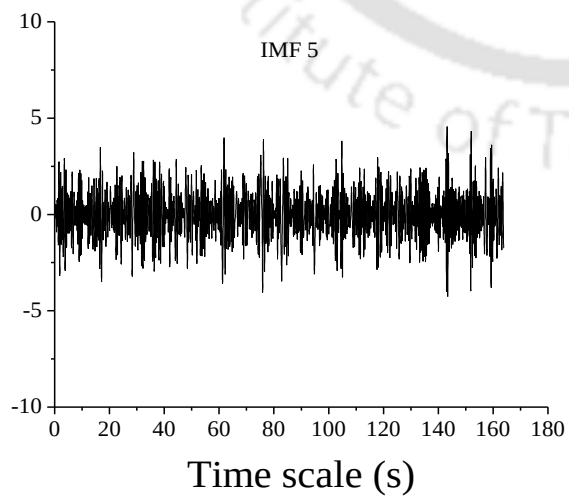
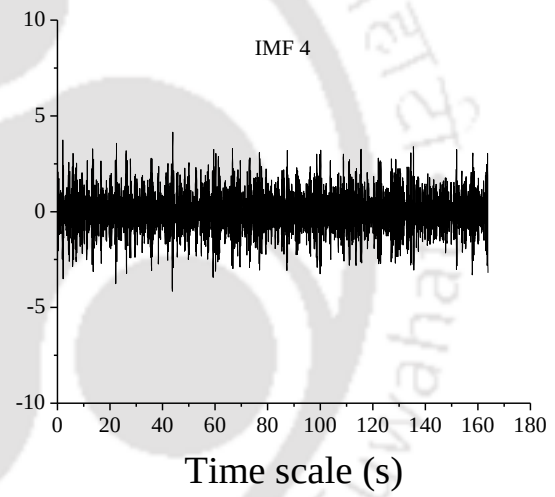
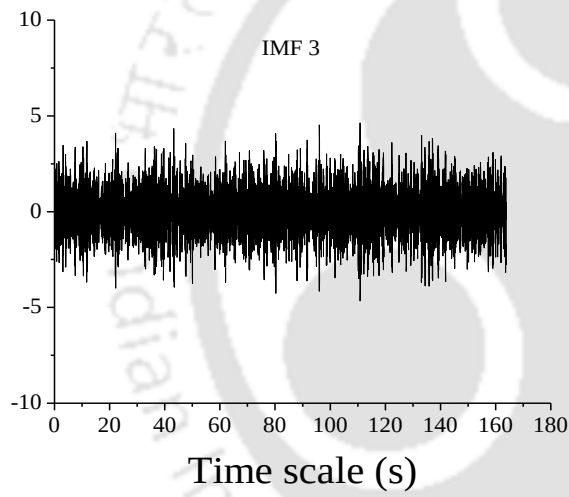
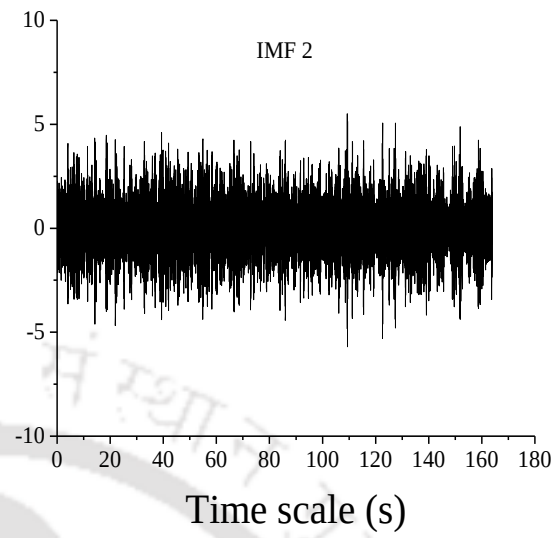
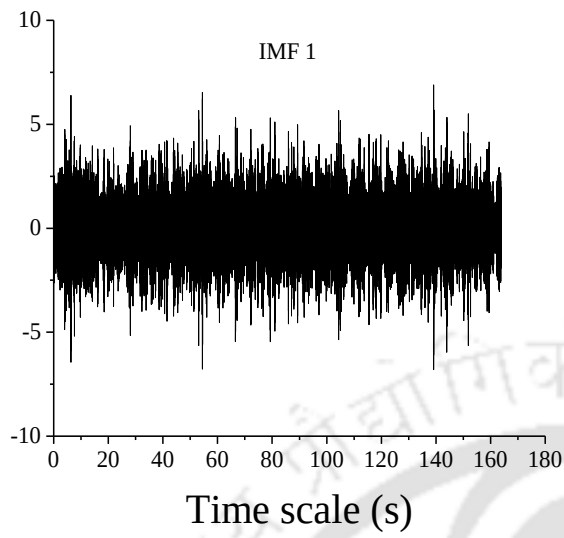
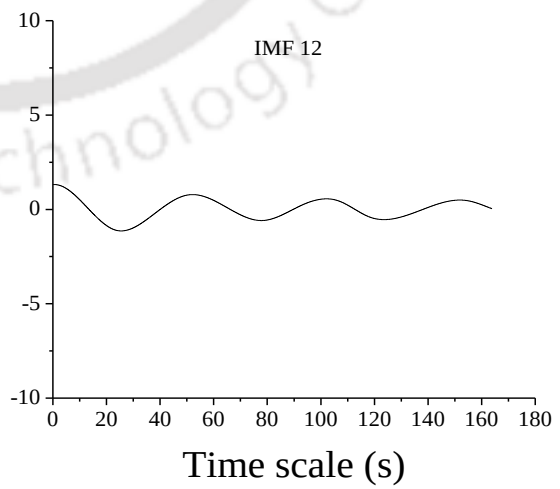
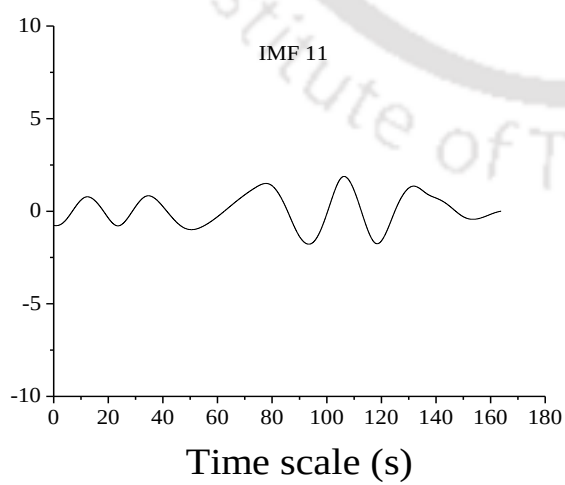
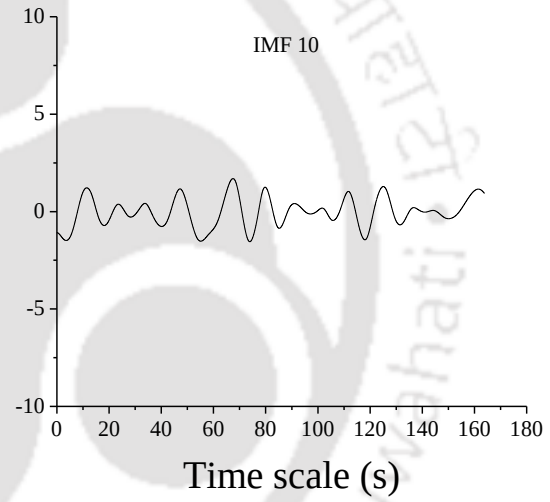
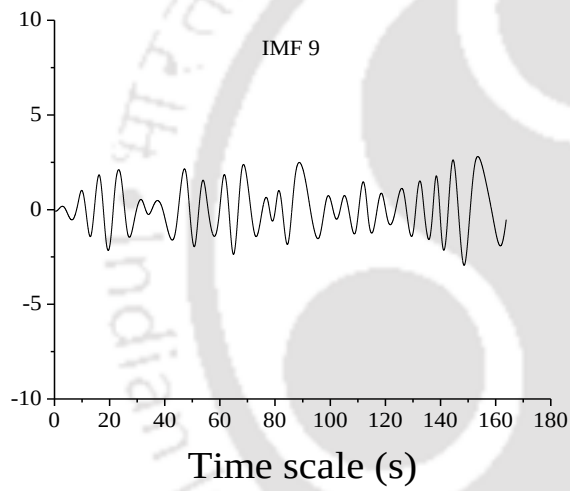
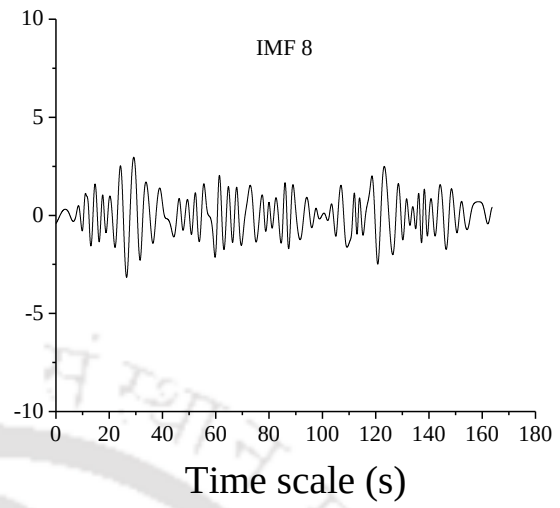
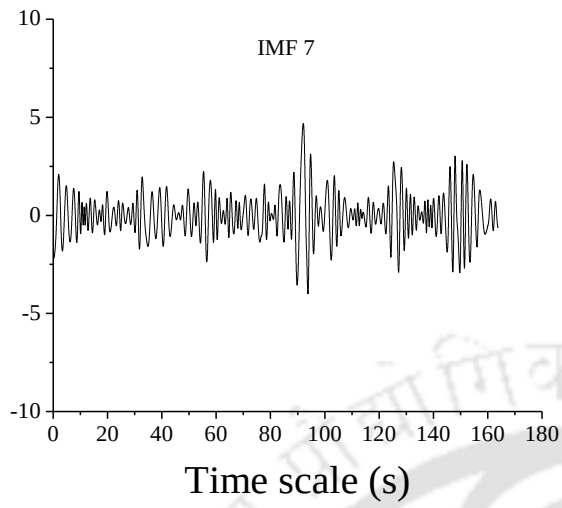


Figure 5.14: Original velocity time series datasets for 10% seepage run





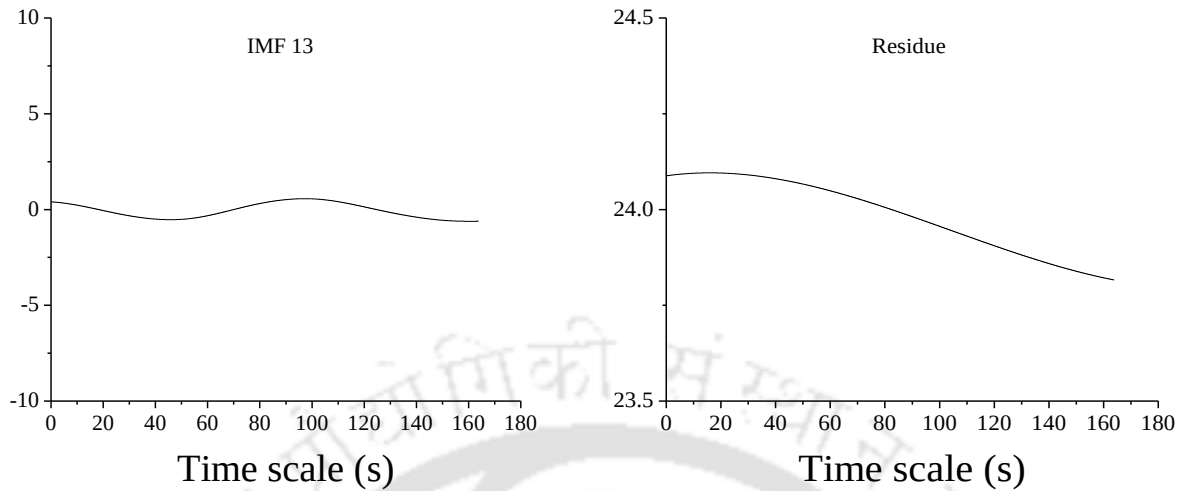


Figure 5.15: IMFs and residue extracted from the velocity time series of no seepage flow

The small sampling time interval of 1/100 s has presented the high intermittence (high frequency part) to the frequency–time spectrum (Figure 5.16 and 5.17). We can also see that greater energy points are accumulated near low frequency zones. In addition to this, we can see that high energy level signals are more scattered or shows a wider range of frequencies. **Huang *et al.* (1998)** reported that the turbulent flows are dominated by common properties like intermittency and frequency modulations. These evident the effect of turbulent flow on the instantaneous sediment transport at a small time scales. Most of the turbulent fluctuations of velocity signal are filtered out by the high acquisition rate. Even in steady flow nature, we have seen that the velocity time series rate can be highly unsteady until the data sampling duration is small enough. Hence, the velocity signal fluctuates with time scales. It can be found from the spectrum that the dominant time scale of no seepage and seepage run detects in the zone with normalized frequency equal to 0.007 and 0.008. The time interval between two data sets was 1/100 s in the present experimental study, so the dominant time scale is $(1/\text{dominant relative frequency}) (\text{measuring time interval}) = (1/0.007) (1/100) = 1.43 \text{ s}$ for no seepage flow and 1.25 s for seepage flow. The observed dominant time scale for no seepage flow is near to the approximation of holding time of bed particle mobility (**Wu and Yang, 2004**). A large time scale corresponds to a slower variation of the physical quantity in time (**Cao *et al.*, 2007**). We have seen that dominant time scale for seepage flow is lower as compared to no seepage flow which suggests that rate of sediment transport is increased in seepage flow. In the field, extraction of such type of information can be very useful to monitor changes in alluvial channels.

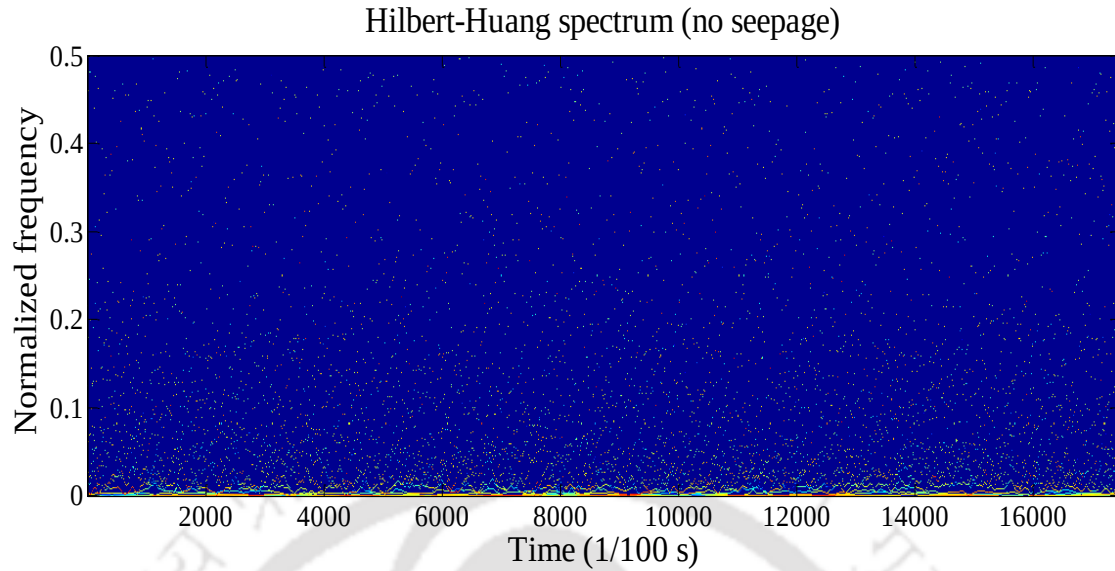


Figure 5.16: Identification of varying time scales from the Hilbert-Huang spectrum for no seepage

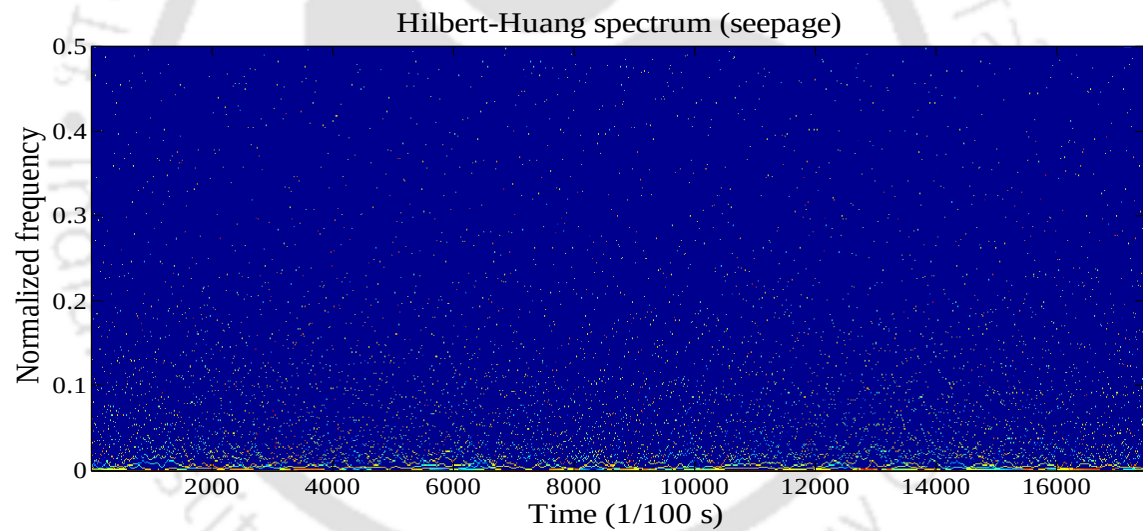


Figure 5.17: Identification of varying time scales from the Hilbert-Huang spectrum for seepage

5.8 Wavelet Based Semblance Analysis

The importance of similarity measures is increasing continuously in order to compare the multiple signals from different field. The measure of semblance analysis gives similarity between two datasets based on correlations between their phase angles in terms of frequency. **Cooper and Cowan (2008)** have used wavelet built semblance analysis for comparing series of synthetic data. In the present study, we have calculated semblance using continuous wavelet transform (CWT) because CWT based semblance analysis permits the local phase interactions

between the two signals in terms of both scale and time. The process of comparison between two time series datasets using CWT can be expressed as **(Torrence and Compo, 1998)**

$$W_{f1,2} = W_{f1} \times W_{f2} \quad (5.24)$$

The quantity $W_{f1,2}$ is a complex nature with an amplitude X and local phase θ is given by

$$X = |W_{f1,2}| \quad (5.25)$$

$$\theta = \tan^{-1} \{ \text{Imaginary}(W_{f1,2}) / \text{Real}(W_{f1,2}) \} \quad (5.26)$$

When the local phase θ of two datasets are calculated, the semblance at each frequency is measured as **(Cooper and Cowan, 2008)**

$$S = \cos^n(\theta) \quad (5.27)$$

where n is an odd number. The advantage of using S compared with θ is that the values of S will be varied from -1 to 1. Values of $S=-1$ and $S=1$ indicates inversely correlation and perfectly correlation between two datasets respectively, while $S=0$ represent un-correlation between signals. Because S and θ gives information of phase angles only rather than amplitude, the advantage of comparison of two signals using S and θ is that they (signals) are not necessary to require the same units. The absence of amplitude information leads to noise sensitivity for which we can further express as

$$D = \cos^n(\theta) |W_{f1} \times W_{f2}| \quad (5.28)$$

When $n=1$, the value of D is approximately equal to the vector dot product of two wavelet coefficient for each scale and position. D indicates the information of amplitude X with information of phase S that can be beneficial if the phase correlations of the higher amplitude components of the signal are of the best noticeable. In the present study, semblance analysis is done for two datasets at a section which contains velocity time series datasets of no seepage and seepage flow. In first dataset of Figures 5.18, the real part of the complex CWT indicates the occurring of velocity time series for the duration of the dataset in no seepage flow. In the second dataset of Figures 5.18, the velocity time series dataset of flow in seepage exist but this dataset differs in phase angle from those in the first dataset in terms of position. Figures 5.18 (a) shows the semblance S and dot product D (calculated with $n = 1$) for main flow discharge. In time

portion (0–60) of the data series, the longer and shorter wavelength component respectively represents the perfectly correlated ($S = 1$) and inversely correlated ($S = -1$).

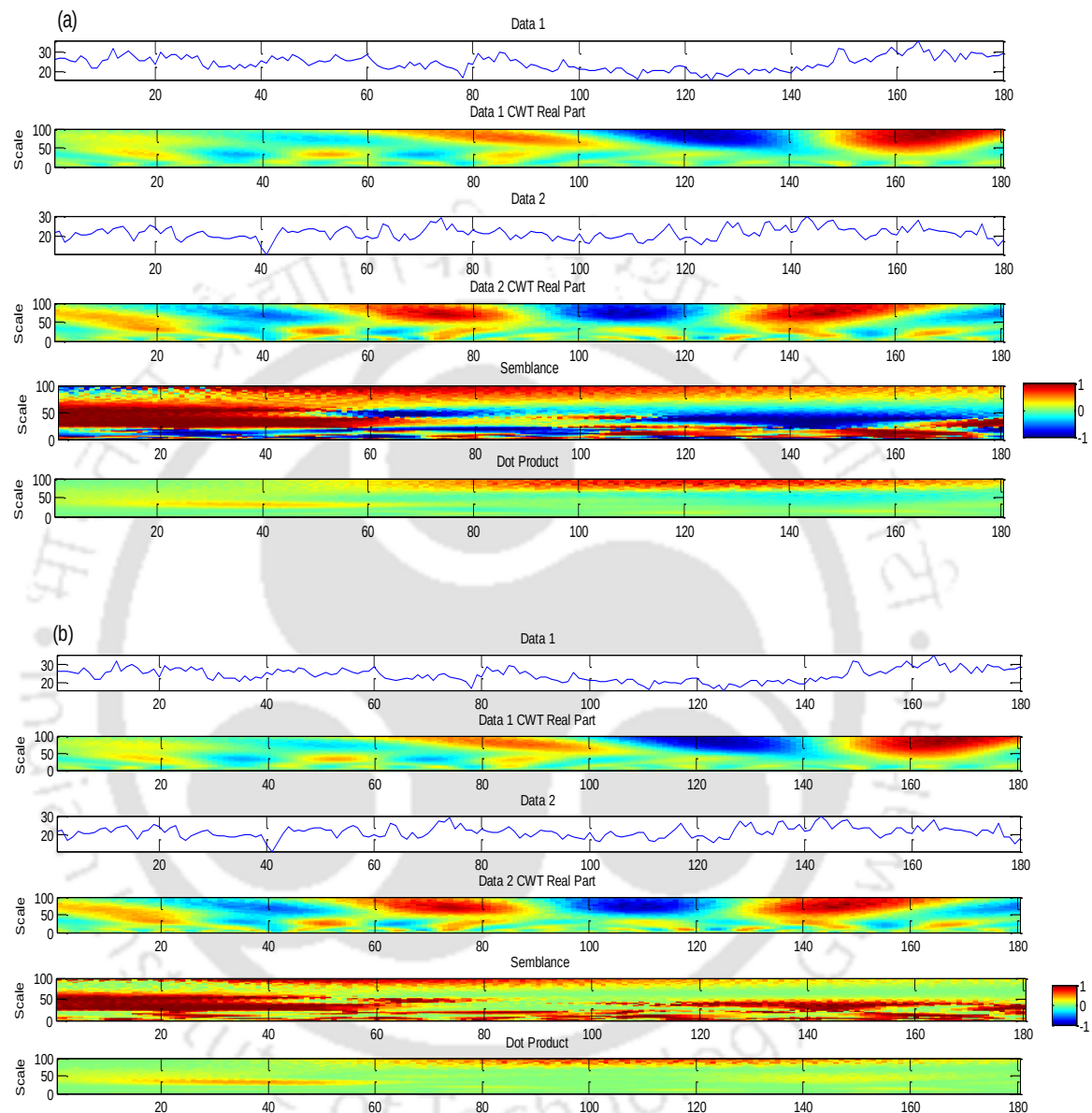


Figure 5.18 Semblance analyses of velocity time series for order (a) $n=1$ and (b) $n=3$

Figure 5.18 (a) shows a broad blue patch in the portion (0–60) at a wavelength of around 25 units, representing a negative correlation between the two data series. In the portion (60–120) of the plot, the phase relationship between the velocity time series components of the two data series was changed so that they are correlated, uncorrelated or inversely correlated as a function of position. In time portion (120–180) of the data series, the larger wavelength component is

inversely correlated ($S = -1$) while the shorter wavelength component shows a broad red patch at a wavelength of approximately 15 units, representing a positive correlation between the two data series. There is improvement in the simplicity of plot by dot product when information of amplitude is incorporated concurrently. However, if we will interest to compute the phase relationship of signals corresponding to small values of amplitude, then displaying this plot might not be the correct way. The longitudinal defining restrictions of this methodology are scale dependant that is visible longitudinal deviations in semblance is sharper at lower wavelengths as compared to higher wavelengths. These happen because of wavelet transform nature. Figure 5.18 (b) shows the effect of increasing the parameter n on the plots of S and D . As n is increased to 3, the spreading in the areas of positive and negative correlation is gradually decreased. The higher values of n will have a tendency to grind down the degree of these areas mainly at higher wavelengths creating the selection of an optimum value dependant on the signals.

5.9 Conclusions

The experimental investigations were carried out to observe the changes in the bedload discharge in seepage affected alluvial channel comprising of non-uniform sediment mixture. Close observation of the turbulent parameter suggest an increment in stream wise velocity and Reynolds stresses near the bed, which causes increased sediment transport after the application of downward seepage (Chapter 3). The median grain size and standard deviation of bedload is increased with the increase of seepage percentage, which indicates that non uniformity of bedload material increases with the presence of downward seepage. Based on modified median grain size and relevant flow parameters including seepage discharge, we have developed the empirical bedload transport equation for seepage affected alluvial channel. The proposed empirical equation is the function of the Froude number, friction Reynolds number and seepage particle Reynolds number. The various efficiency criteria have been used to checked the performance of empirical equation and found that the equation is well fitted with the observed values of bedload transport. We further compared the performance of proposed empirical equation with existing bedload transport formulas. For the range of sediment type and distribution discussed in this study, the best predictions were obtained using Myer Peter and Muller (1948) method for no seepage runs, while in case of seepage runs, none of the models perform well. The newly proposed formulas provide better predictions than several existing

methods. Further research is required to generalize the proposed empirical equation based on laboratory and field observations to enhance the accuracy and reliability and to broaden its usability in the field application.

The temporal variation of bedload transport rate in the presence of downward seepage was discussed in this chapter. In the present chapter, we have considered the non-uniform sand bed channel with whole permeable bed. The experimental results indicate that the bedload transport rate quickly increased with time and reach the maximum value then again started to decrease. The obtained results conclude that, the bedload transport rate slowly reduced with time till the channel reaches stable position, while the bed degradation remains constant. The temporal variation of falling limb of the bed load transport was studied by incorporating the dimensionless bedload transport rate and the dimensionless time. We have proposed the empirical expression for the temporal variation of bedload transport rate subjected to downward seepage which is modification of theoretical expression given by Liu *et al.* (2014). The proposed equation is based on the experimental data over non-uniform sand bed channel with whole permeable bed which is applicable for canals and rivers. These new results provide insight to explain the distinctive morphology of the non-uniform grained bedload transport subjected to no seepage and seepage runs in non-uniform sand bed channel.

Experimental study has been performed to compare the earth-surface morphology of no seepage and seepage flow. Present results reveal that average celerity of bedload increases with increasing percentage of seepage. This may happen due to the dominance of high rate of bedload transport with downward seepage. Interestingly, bedload of same length scale also show increment in their celerity with downward seepage. Presence of the numerous time scales in the velocity time series becomes a significant part in the modelling of flow velocity prediction. The efficiency of flow velocity formulas may be highly influenced by the unsuitable demonstration of the time scales series. The used spectrum analyses tools in the present chapter can support classify various time scales in the case of velocity time series datasets. Thus, an HHT analysis is a suitable method to observe multiple time scales in velocity time series datasets. Also, we have performed wavelets built semblance analysis of time series datasets to compare their correlations in terms of both scale and time. The methodology of semblance analysis is applicable to extensive range of problems for fluvial hydraulics.

6 Conclusions and Recommendations for Future Work

Experimental investigation was carried out for studying the flow characteristics in non-uniform sand bed channel on two sand beds of median diameter 0.425 mm and 0.50 mm. The experiment framework was designed in two categories: no seepage and with seepage experiments. In this study, role of no seepage and downward seepage on the development of bed-features in a non-uniform sand bed channel was studied. Experiments were conducted for no-seepage, 10% seepage and 15% seepage. In order to get an insight into the physics behind the initiation of bed-features, turbulent flow parameters were investigated when no seepage and downward seepage was applied to the channel. With the application of no seepage and downward seepage, bed-features such as sheet flow layer were observed on both the sand beds. Further, temporal and spatial variations of developing bed-features are analyzed in the presence of no seepage and downward seepage. Important conclusions are drawn from the present study which is presented below:

6.1 Boundary Layer Development over Non-uniform Sand Bed Channel with Downward Seepage

An experimental study has been carried out to investigate boundary layer development using the measurements of velocity profiles in non-uniform sand bed channel subjected to no seepage and seepage flows.

- The experiment presented herein reproduces the development of a boundary layer in a water flume over a non-uniform sand bed channel at high Reynolds numbers. The velocity measurements were performed via a three-dimensional acoustic Doppler velocimeter (ADV) technique in the vertical symmetry plane of the flow. Measured velocity profiles are used to determine the boundary layer thickness and development length.

- For no seepage and seepage flow conditions, the maximum development length was obtained at $x=6.5$ m and $x=7.5$ m respectively. Therefore, it can be accepted that the flow was fully developed beyond that point.
- No seepage flow gives higher thickness of boundary layer and smaller boundary layer development length values as compared to seepage flows. The boundary layer development varies between 56 to 65 times of the water depth for all flow conditions.
- The boundary-layer growth was established on the basis of the mean longitudinal velocities yielding 0.8 power law in agreement with previous results for the growth of an internal boundary layer. However, it appears that coefficient is smaller than that for a typical internal boundary layer.
- The present chapter introduces empirical equations in order to predict boundary layer thickness and flow development length in seepage affected alluvial channel under present experimental conditions.

6.2 Structure of Turbulence over Non-uniform Sand Bed Channel with Downward Seepage

An experimental study has been carried out to observe changes in the turbulent characteristics of flow when downward seepage was applied to an alluvial channel composing of non-uniform sediment mixture. The present study addresses on the flow zone in the vicinity of the bed surface. Close observation of the turbulent structure of flow leads to the conclusion which causes increase of bed material transport after the application of downward seepage to the channel.

- The variations of stream wise velocity with vertical distance in flows with downward seepage in the near bed region are greater than no seepage flow. This increase in stream wise velocities was sufficient for increased rate of sediment transport with seepage.
- The profiles of Reynolds shear stress is slightly scattered in general and increased with seepage which signifies the greater momentum transfer towards the boundary. The distribution of Reynolds shear stress undergoes a damping due to a decreasing level of turbulence fluctuation within the wall shear layer.

- The thickness of roughness sub layer and shear velocities were increased with the application of downward seepage. The reduced value of von Karman's constant corresponds to increase in the bed load transport with seepage.
- In presence of seepage, increase in stream wise and vertical turbulence intensities is observed than turbulence intensities in no-seepage. The turbulence is strongly anisotropic being seepage independent and varying approximately linear with flow depth.
- The turbulent diffusivity and the mixing length decreases with seepage. Decrease in the slope of turbulent mixing length corresponds to lower mixing length of turbulent eddies with seepage.
- Application of downward seepage increases the flux transport in the downward direction and then carried the sediments along the flow direction, therefore highlighting the dominance of sweep event over ejection event.
- The distributions of third order moment suggest that the sweeps are prevalent over the near bed flow depth associated with the arrival of high speed fluid parcel. Increased fourth order correlation in stream wise direction within near bed surface represents higher level of turbulence intermittency with seepage.
- Seepage influenced higher turbulence production and higher pressure energy diffusion rate, lower turbulent diffusion and lower TKE dissipation rate due to which the level of turbulence in the flow has been increased with the application of downward seepage.
- The integral time scale and length scales were increased significantly with seepage, which results the increased energy and momentum transfer caused by the larger eddy size in the near bed region.
- The computed and experimental PDF distributions of turbulent parameters for both no seepage and seepage case are in good agreement except that a slight departure of inward and outward interactions is observed which may be due to weaker events.
- The present study develops the theoretical expression for the turbulence intensity which is good agreement with the experimental results and follows the universality which is applicable for no seepage and seepage runs.

6.3 Sheet Flow Hydrodynamics over Non Uniform Sand Bed Channel

An experimental study has been carried out to observe changes in the turbulent characteristics of flow in the presence of sheet layer in an alluvial channel composing of non-uniform sediment mixture. The present chapter addresses the vertical distribution of turbulent flow for no seepage and seepage runs. Close observation of the turbulent structure of flow leads to the conclusion that the sheet material transport increases after the application of downward seepage to the channel.

- The variations of stream wise velocity with sheet layer are greater than without sheet layer but the result is quite opposite in case of vertical velocity. Similar trends are observed in no seepage and seepage flows except stream wise and vertical velocities are increased with the application of seepage. This increase in stream wise velocities was sufficient for increased rate of sediment transport which corresponds to increased rate of sheet layer transport with seepage.
- The profiles of Reynolds shear stress are slightly scattered in general and increased with sheet layer which signifies the greater momentum transfer towards the boundary. Similar trends are observed in no seepage and seepage flow and Reynolds shear stress is increased with the application of seepage.
- In presence of sheet layer, increased in stream wise and vertical turbulence intensities are observed than turbulence intensities without sheet layer. Similar trends are observed for both no seepage and seepage flow and it is increased with the application of seepage.
- Sheet flow influenced higher turbulence production and lower TKE dissipation rate due to which the level of turbulence in the flow has been increased with the presence of sheet layer.
- The centre line profile of the sheet flow movement suggests that initially, the sheet layer moves faster, but with the period of time its movement slows down. The net effect of sheet layer development is to increase the cross-sectional area until the developing shape is about to attain equilibrium where thickness of sheet flow have very minimal variation.
- The rate of sheet flow movement has been increased with the application of downward seepage. Reasons behind the increments of sheet layer movement with the application of downward seepage have been linked to turbulent flow measurements.

- Average celerity of sheet flow increases with increasing percentage of seepage. This may happen due to the dominance of high rate of sheet flow movement with seepage in downward direction. Sheet flow of same length scale also shows increment in their celerity with downward seepage.
- Non uniformity in the resultant collected sheet material (sheet material after the experiment) is increased with the increase in seepage percentage.

6.4 Bedload Transport of Non-uniform Sediment in Seepage Affected Alluvial Channel

The experimental investigations were carried out to observe changes in the bedload discharge in seepage affected alluvial channel comprising of non-uniform sediment mixture. Close observation of the turbulent parameter suggest an increment in stream wise velocity and Reynolds stresses near the bed, which causes increased sediment transport after the application of downward seepage (Chapter 3).

- The median grain size and standard deviation of bedload is increased with the increase of seepage percentage, which indicates that non uniformity of bedload material increases with the presence of downward seepage.
- Based on modified median grain size and relevant flow parameters including seepage discharge, we have developed the empirical bedload transport equation for seepage affected alluvial channel. The proposed empirical equation is the function of the Froude number, friction Reynolds number and seepage particle Reynolds number.
- The various efficiency criteria have been used to checked the performance of empirical equation and found that the equation is well fitted with the observed values of bedload transport.
- We further compared the performance of proposed empirical equation with existing bedload transport formulas. For the range of sediment type and distribution discussed in this study, the best predictions were obtained using Myer Peter and Muller (1948) method for no seepage runs while in case of seepage runs, none of the models perform well. The newly proposed formulas provide better predictions than several existing methods.

- The temporal variation of bedload transport rate in the presence of downward seepage was discussed. The experimental results indicate that the bedload transport rate quickly increased with time and reach the maximum value then again starts to decrease till the channel reaches stable position, while the bed degradation remains constant.
- The temporal variation of falling limb of the bed load transport was studied by incorporating the dimensionless bedload transport rate and the dimensionless time. We have proposed the empirical expression for the temporal variation of bedload transport rate subjected to downward seepage which is modification of theoretical expression given by Liu *et al.* (2014).
- Present results reveal that average celerity of bedload increases with increasing percentage of seepage. This may happen due to the dominance of high rate of bedload transport with downward seepage. Interestingly, bedload of same length scale also show increment in their celerity with downward seepage.
- The HHT spectrum analyses tools can support classify various time scales in the case of velocity time series datasets. Thus, an HHT analysis is a suitable method to observe multiple time scales in velocity time series datasets.
- We have performed wavelets built semblance analysis of time series data to present their correlations in terms of both scale and time.

6.5 Motivations

Results show that no seepage and downward seepage influences the flow hydrodynamics, sediment transport, and bed morphodynamics in a non-uniform sand bed channel. These results provide the distinctive morphology of the non-uniform grained bed-features and turbulent flow structure in a non-uniform sand bed channel. The transition of plane bed condition to development of bed-features in the presence of no seepage and seepage would have effect on resistance characteristics of natural channel. The results of such transition report in the present work will help in deciphering the different sediment modes and mechanism of seepage affected natural channel. This study can also be implemented in order to investigate the flow resistance, grain sorting, and sediment erosion, in seepage affected natural channels. Besides, these analysis

techniques can be easily applied to any real dataset of natural streams bed profile as these are purely statistical analyses of bed feature geometry.

6.6 Recommendations for Future Work

The present study, understanding of bed features dynamics and flow process on the laboratory flume in the presence of no seepage and seepage, opens the door for further detailed investigation regarding bed morphology in natural channels. In this experimental work, only two types of bed sediments were used i.e., non-uniform sand (median diameter = 0.42 mm and 0.5 mm) bed materials. Therefore, more investigation is needed on different types of grain sizes for getting overall understanding of bed-features dynamics with downward seepage. The present study can be further extended to quantify and signify the bed resistance because of developing bed-features in the presence of no seepage and seepage for improving the understanding and behavior of alluvial channels.

More experiments in the laboratory by using different grain sizes and seepage percentages can be performed to get more clarification about sheet flow dynamics and progradation on a non-uniform sand bed channel. In this study, the proposed equation for the resultant bed load and temporal variation of bed load through the experimental study is empirical in nature. Further research is required to generalize the proposed empirical equation based on laboratory and field observations to enhance the accuracy and reliability and to broaden its usability in the field application.

From the experimental channels and natural river channels, it is observed that there is some threshold condition of flow at which sediment particle start to move. The threshold condition of flow is defined as bed shear stress and if bed shear stress is greater than stabilizing resistance to the sediment particles, the sediment can be mobilized. The specific velocity or bed shear stress at which the particles on the bed surface start to move, called the threshold velocity or threshold shear stress. **Shields (1936)** in his pioneering study characterized this flow condition as the condition of incipient motion for the uniform sediment. Future researchers can incorporate the incipient motion conditions into the theories of non-uniform sediment transport in seepage affected alluvial channel.

Appendix A

Double Averaged Flow Turbulence Characteristics of Mobile Bed Channel with Downward Seepage

1. Introduction

Even if large numbers of studies have been carried out with time averaged turbulence structure of flows with downward seepage (section 1.4), the flow near the sand bed is considerably heterogeneous in space. Therefore, the double averaged (DA) method is essential for valid calculation for the seepage affected near bed turbulence characteristics. The DA method consists of averaging the Navier Stokes equations in time and space over an area parallel to the stream wise direction. This method, therefore, takes into account the spatial heterogeneity in the mean flow quantities introduced by the heterogeneous sand bed geometry. As DA method can be applicable for flow over the sand bed, it potentially can offer new insights into the turbulence characteristics of an alluvial river. **Nikora *et al.* (2007a, 2007b), Manes *et al.* (2007), Mignot *et al.* (2009), Sarkar and Dey (2010), Dey *et al.* (2011), Dey and Das (2012) and Martino *et al.* (2012)** introduced the details of DA method in their works. In the previous studies, DA methods originally developed for atmospheric boundary layer studies (**Wilson and Shaw, 1977; Raupach and Shaw, 1982**) have been revisited in the field of river hydraulics (**Nikora *et al.*, 2001; Aberle, 2007; Coleman *et al.*, 2007**). **Lopez and Garcia (2001)** used DA method to study the turbulence structure in an open channel flow over a vegetation canopy.

Experimental flume studies have done by previous researchers to investigate double averaged turbulent characteristics of flow which were carried out in either gravel bed or vegetation canopy. Also, their study primarily deals with small seepage zone length as compared to the channel length. The investigation about the effect of downward seepage on the DA flow structures in sand bed with the large seepage zone length are yet to be explored. Thus, the focus of the present study is to explore the effect of downward seepage on the DA turbulent structures

of flow over the sand bed and to analyse the physics behind sediment transport with downward seepage in alluvial channels.

2. Material and Methods

Different main flow discharges (three in number) were considered for experimentation. The reference discharge was calculated as per incipient motion condition. At incipient motion condition, the sediment particles were at the threshold of motion i.e., the bed shear stress was approximately equal to the critical bed shear stress. **Yalin (1976)** criteria were used to determine the threshold condition. A summary of experimental values of flow parameters are given in Table 1 where Q_0 , h , R_e and q_s represent main flow discharge, flow depth, flow Reynolds number and seepage discharge respectively. Data were collected at the centre line of the channel cross section at a distance of 8.5m, 8m, 7.5m and 7 m from the downstream end of the flume.

Table 1: Experimental value of flow parameters

Q (m ³ /s)	h (m)	R_e	q_s (% Q_0)
0.0364	0.117	36387	0, 10, 15
0.0429*	0.125	42875	0, 10, 15
0.0465	0.129	46440	0, 10, 15

*Runs used for the analysis of turbulence characteristics because incipient motion condition is achieved at this particular depth and discharged.

3. Results and Discussion

3.1. Mean Velocity

The time averaged flow behavior is three dimensional due to bed irregularities. In order to deal with the spatial heterogeneity, the double averaged (DA) flow quantities are required. The decomposition of the local time averaged flow velocity based on the intrinsic spatial averaging is given by $u = \langle u \rangle + \tilde{U}$ where $\tilde{U}$ is the deviation of local time averaged flow velocity (u) from the DA flow velocity $\langle u \rangle$. The intrinsic spatial averaging is noted by single bracket $\langle \rangle$ which is proposed as area integration over a surface parallel to the bed at variable elevation (**Nikora et al., 2007a, 2007b**). Figure 1(a) shows the variations of DA stream wise velocity with normalized

vertical distance (z/h) in flows subjected to no seepage and seepage for 10% and 15%. It is interesting to note in Figure 1(a) that the velocity increased with the application of downward seepage as compared to no seepage for threshold incipient motion condition. Quantitative analysis suggests that at near bed, velocity increases on an average value of 8-12% with the application of 10% seepage. It is also observed that in the lower flow region, the velocity for 15% seepage case is slightly higher with an average increasing value of 0.6-1% than the velocity for 10% seepage.

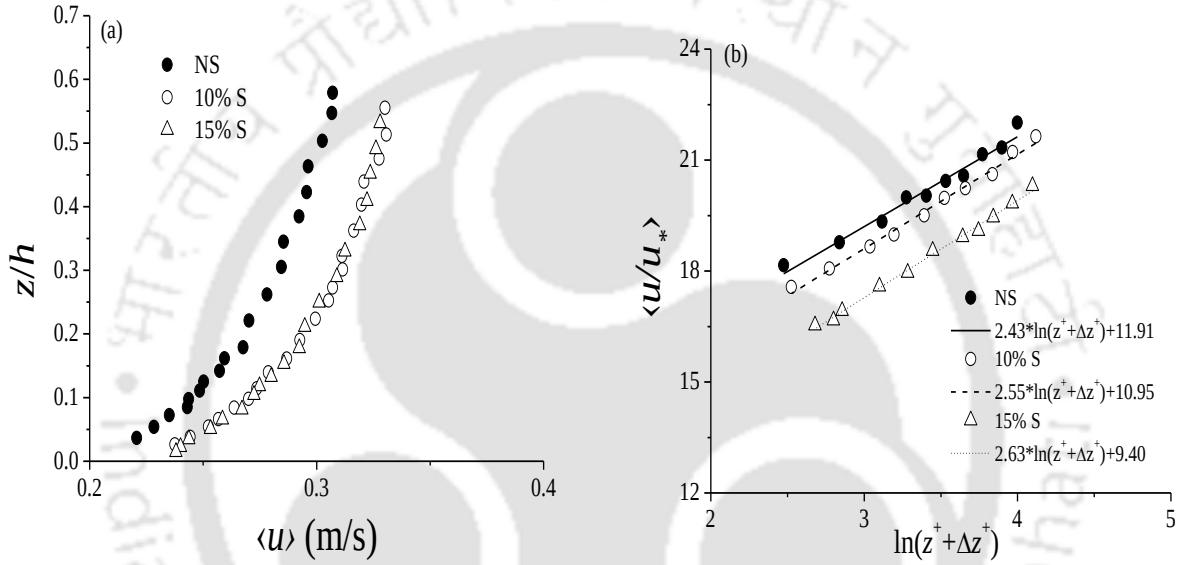


Figure 1: (a) Vertical profiles of mean stream wise velocity (b) Velocity logarithmic law

In order to plot the experimental data, **Deshpande and Kumar (2016a)** log law expressed in following non-dimensional form is being considered.

$$\left\langle \frac{u}{u_*} \right\rangle = \frac{1}{k} \ln \left(\frac{\langle z^+ \rangle + \Delta z^+}{\varepsilon^+} \right) \quad (1)$$

Where, $z^+ = z/d_{50}$, $\Delta z^+ = \Delta z/d_{50}$, Δz is depth of virtual bed level from the bed, $\varepsilon^+ = z_0/d_{50}$, z_0 is zero velocity level, u_* is friction velocity, d_{50} ($=0.418\text{mm}$) is median grain size, k is von Karman constant. The modified logarithmic law for no seepage condition as shown in Figure 1(b) does not hold for seepage condition as there is exchanged of momentum between the flow and sand bed. As a result, the modified logarithmic law is developed for both 10% and 15% seepage condition as shown in Figure 1(b). Average values of the von Karman's constant, friction

velocity, depth of the virtual bed level and the zero velocity level are tabulated in Table 2. Value of the k for no seepage condition has been observed to be same with the universal value (0.41). With the application of downward seepage, bed particles start to move rapidly (Table 3) and value of k reduce to 0.39 and 0.38 under the condition of 10% S and 15% S respectively. The depth of virtual bed level and the zero velocity level from the bed increased after the application of downward seepage, suggesting an exposure of increased velocity component in the stream wise direction to the particles on the bed.

Table 2: Coefficient value observed from velocity logarithmic profiles

Flow ($Q=0.0429 \text{ m}^3/\text{s}$)	u_*	k	Δz (mm)	z_0 (mm)
NS	0.01216	0.41	0.418	3.175×10^{-3}
10% S	0.01352	0.39	1.92	4.368×10^{-3}
15% S	0.01540	0.38	2.93	10.1×10^{-3}

Table 3: Observed values of sediment transport rate

Q_0 (m^3/s)	q_s (% Q_0)	Sediment transport rate (gram/hour)
0.0364	0	2.00
	10	3.12
	15	3.75
0.0429	0	15.55
	10	33.56
	15	40.75
0.0465	0	31.78
	10	59.86
	15	68.98

3.2 Reynolds Shear Stress

In the momentum balance (**Manes *et al.*, 2007**), the DA total shear stress $\langle \tau \rangle$ is

$$\langle \tau \rangle = -\rho \langle \overline{u'w'} \rangle - \rho \langle \tilde{u}\tilde{w} \rangle + \rho \nu \left\langle \frac{\partial u}{\partial z} \right\rangle \quad (2)$$

Where $\tau_{uw} = -\rho \langle \overline{u'w'} \rangle$ is Reynolds shear stress, $\tau_f = -\rho \langle \tilde{u}\tilde{w} \rangle$ is form induced stress, $\tau_{vis} = \rho \nu \left\langle \frac{\partial u}{\partial z} \right\rangle$ is the viscous stress, ρ is density of fluid, ν is kinematic viscosity of fluid. In the above Equations (2), the straight over bar and the angle brackets denote the time and spatial average of flow variables respectively whereas the prime and the tilde denote the time fluctuations and the spatial disturbances respectively i.e., $u' = U - u$ and $\tilde{u} = u - \langle u \rangle$ where U is the instantaneous velocity measurement in the stream wise direction. The spatial averaging procedure introduces additional terms known as form induced stress with respect to the traditional time averaged Reynolds equations. In Figures 2, the vertical profiles of all these quantities are shown in DA form in flows subjected to no seepage (0%S), 10% seepage (10%S) and 15% seepage (15%S), where the vertical ordinate (z) has been made non-dimensional with the flow depth (h). As observed from Figure 2, the τ_f and τ_{vis} are negligible with respect to the τ_{uw} over the entire flow depth for all flow condition. With the application of downward seepage, τ_f at 10%S and 15%S is greater than that with no seepage and the negative value of τ_f with no seepage indicates a reduction in total shear stress $\langle \tau \rangle$. In contrast, the viscous shear stress τ_{vis} is increased towards the bed and are of comparable magnitude over the entire flow condition. On the other hand, the Reynolds shear stress τ_{uw} dominates along the flow depth, having a damping within the near bed due to a reduction in turbulence level (Figure 2c). All together $z/h > 0.05$, the total shear stress $\langle \tau \rangle$ reasonably reproduces a linear profile (Figure 2d). Figure 2(d) shows that the total stress increases towards the wall boundary are associated with the provided momentum from the main flow to maintain sediment particle motion overcoming the bed resistance and attain a maximum value in between $0.05 < z/h < 0.2$ and then again decrease towards the boundary because of the presence of a roughness sub layer in the near bed region. Position of maximum total stress from the bed is shifted upwards with the application of downward seepage ($z = 16.5$ mm) as compared to no seepage run ($z = 15.6$ mm) indicating increase in thickness of the roughness sub layer under seepage.

The friction velocity u_* is obtained from double averaged Reynolds shear stress (RSS) profiles by extending the linear projection of RSS on the crest level of the bed particles, $u_* = \left(\tau_{uw} / \rho \right)_{z=0}^{0.5}$ suggested by **Nezu (1977)**. The maximum value of τ_{uw} is about 24% greater with 10%S and about 68% greater with 15%S, as compared to that with no seepage. This

increment in the RSS results in a subsequent increase in the friction velocities by 11.19% and 26.5% in case of 10%S and 15%S respectively as compared to that with no seepage. The value of friction velocity (u_*) are shown in Table 2.

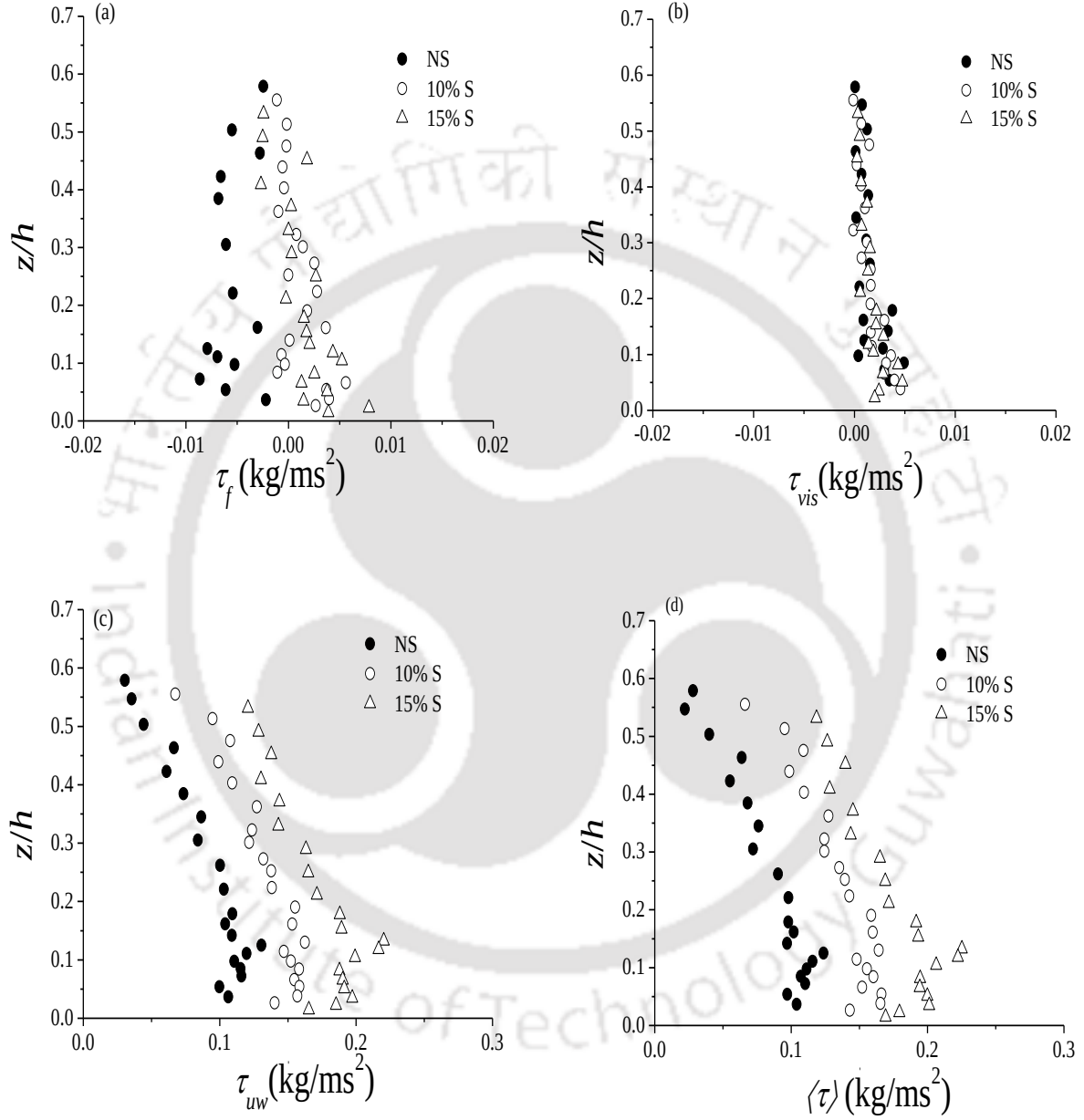


Figure 2: Vertical profiles of (a) form induced shear stress (b) viscous shear stress (c) DA Reynolds shear stress and (d) DA total shear stress

The vertical distribution of the non-dimensional RSS ($\tau_{uw}^+ = \frac{\tau_{uw}}{\rho u_*^2}$) for NS, 10%S and 15%S runs is plotted in Figure 3. Figure 3 show that the distribution of the RSS in the layer just above the bed ($z/h > 0.05$) for NS, 10%S and 15%S runs and are reasonably consistent with the linear law of RSS for free surface flows with zero pressure gradient.

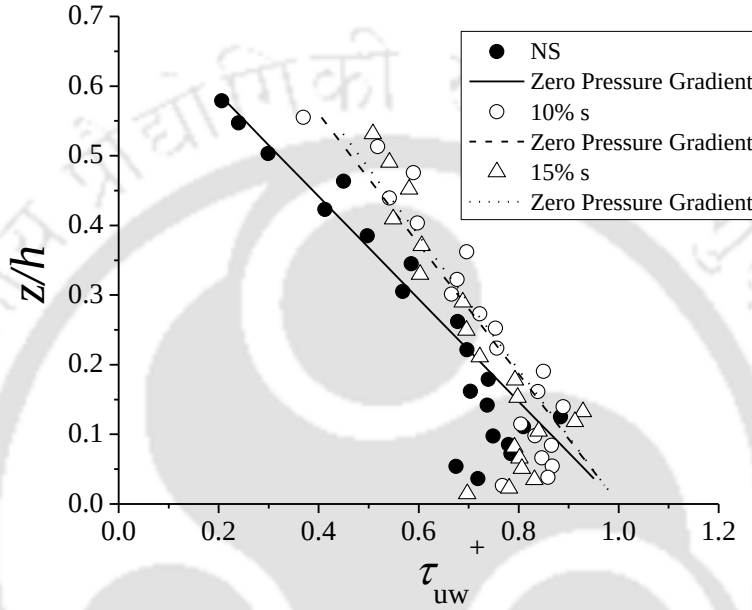


Figure 3: Vertical profiles of normalised RSS

The DA stream wise and vertical turbulence intensities are defined by $\langle \sigma_u \rangle = \left\langle \left(\overline{u'^2} \right)^{0.5} \right\rangle$ and $\langle \sigma_w \rangle = \left\langle \left(\overline{w'^2} \right)^{0.5} \right\rangle$ respectively. The vertical profiles of their normalized values $\langle \hat{\sigma}_u \rangle = \langle \sigma_u \rangle / u_*$ and $\langle \hat{\sigma}_w \rangle = \langle \sigma_w \rangle / u_*$ with NS, 10%S and 15%S are presented in Figure 4. From the present results, normalized turbulence intensities are smaller with downward seepage as compared to those with no seepage. This decreased in the non-dimensional turbulent intensities with seepage may be due to corresponding increase in the shear velocities. The normalized turbulent intensities near bed decreases by an average value of 3% for $\langle \hat{\sigma}_u \rangle$ and 1% for $\langle \hat{\sigma}_w \rangle$ with 10%S and decreases by 12% $\langle \hat{\sigma}_u \rangle$ and 9% for $\langle \hat{\sigma}_w \rangle$ with 15%S as compared to no seepage.

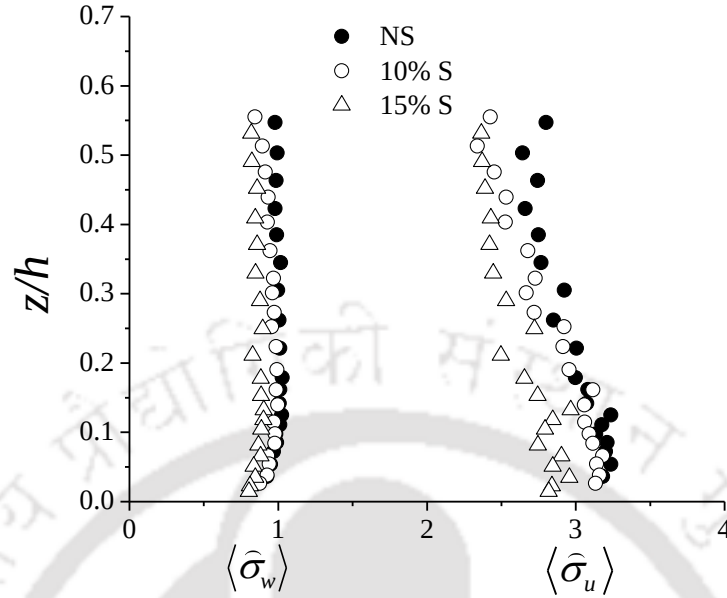


Figure 4: Vertical profiles of normalized DA turbulence intensities ($\langle \hat{\sigma}_u \rangle$ and $\langle \hat{\sigma}_w \rangle$)

Several laws have been suggested describing the vertical distribution of turbulence intensities in uniform open channel flows (Nezu and Nakagawa, 1993; Nikora and Goring, 1998). Since these laws provided by them are not being based on the double averaged equations and considered only time averaging, therefore, double averaged turbulent intensities in stream wise direction for no seepage and seepage runs (shown in Figure 5) are not fitted to the laws provided by Nezu and Nakagawa (1993) and Nikora and Goring (1998). In order to fill up the aforesaid gap, Regression analysis is used to derive new empirical constants (Table 4) for flows over plane boundaries with no seepage and seepage affected channels. The coefficient D_u is observed to be decreased with seepage by (4-7) % as compared to no seepage run and the coefficient C_u is increased with seepage by 10% than those of no seepage run.

$$\langle \hat{\sigma}_u \rangle = D_u \exp\left(-C_u \frac{z}{h}\right) \quad (3)$$

The level of flow anisotropy is measured by the ratio $\frac{\langle \hat{\sigma}_w \rangle}{\langle \hat{\sigma}_u \rangle}$. Figure 6 shows the vertical

distribution of flow anisotropy and achieved that flow is highly anisotropic ($\frac{\langle \hat{\sigma}_w \rangle}{\langle \hat{\sigma}_u \rangle} < 1$).

Table 4: Values of empirical constants for proposed DA stream wise turbulent intensity $\langle \hat{\sigma}_u \rangle$

Flow ($Q=0.0429 \text{ m}^3/\text{s}$)	D_u	C_u
NS	3.27	-0.40
10%S	3.12	-0.44
15%S	2.9	-0.45

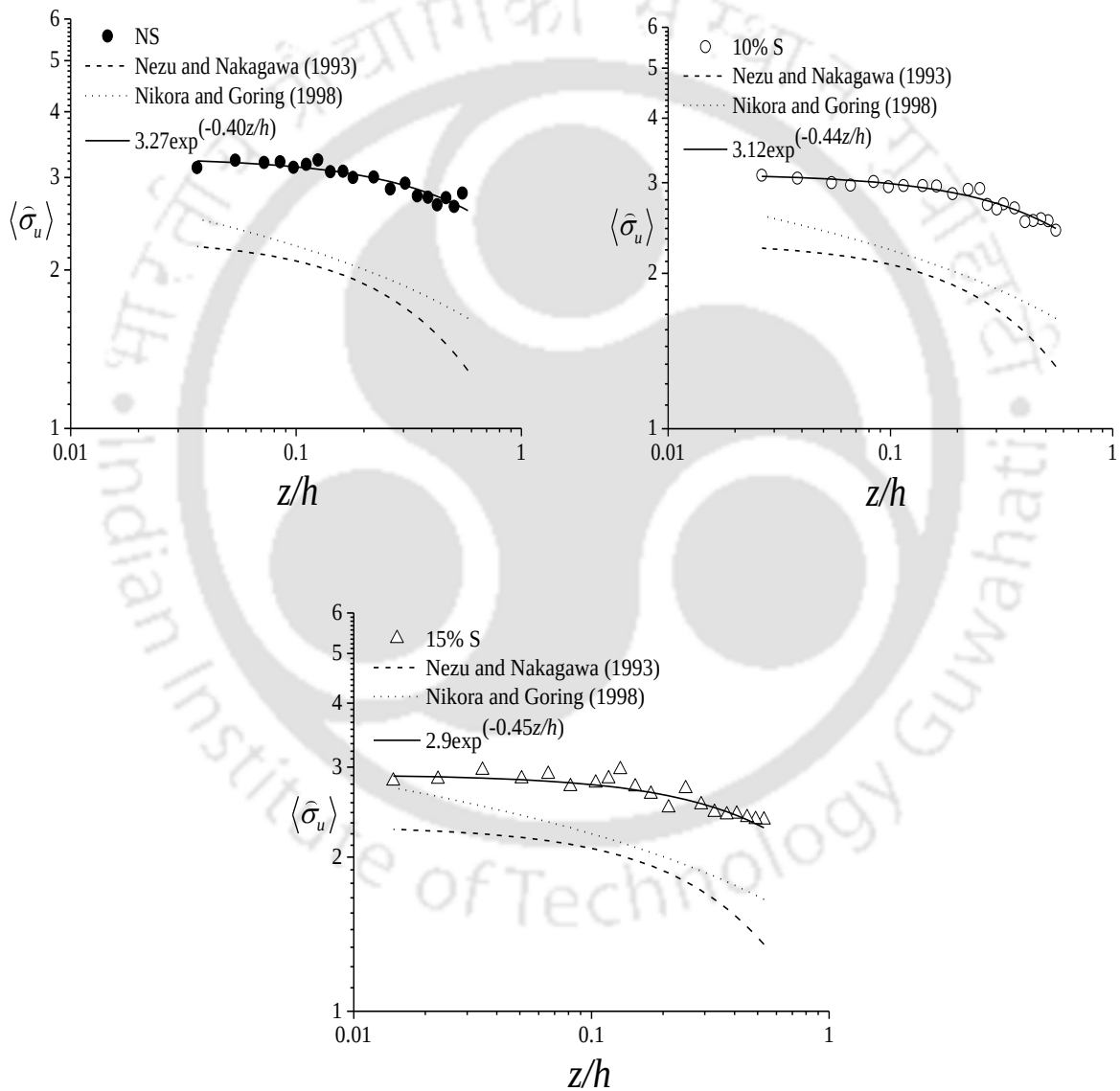


Figure 5: Comparison of DA stream wise turbulent intensity with Nezu and Nakagawa (1993) and Nikora and Goring (1998)

Nezu and Nakagawa (1993) observed that the ratio is universal and has a constant value of 0.55 for smooth boundary. On the contrary, the present findings show that in flow over a plane sand bed, $\langle \hat{\sigma}_w \rangle / \langle \hat{\sigma}_u \rangle$ remains less than those observed by **Nezu and Nakagawa (1993)**. However, the influence of seepage on the profiles of $\langle \hat{\sigma}_w \rangle / \langle \hat{\sigma}_u \rangle$ is not apparent. In the present study, it is observed that the ratio is a function of depth and does not have a constant value which is also obtained by **Deshpande and Kumar (2016a)** in curved cross section sand bed channel. Independent of flow condition (no seepage and seepage runs), the flow anisotropy has a minimum value ($\frac{\langle \hat{\sigma}_w \rangle}{\langle \hat{\sigma}_u \rangle} \cong 0.28$) near the channel bed and increase linearly up to $\frac{z}{h} \cong 0.25$.

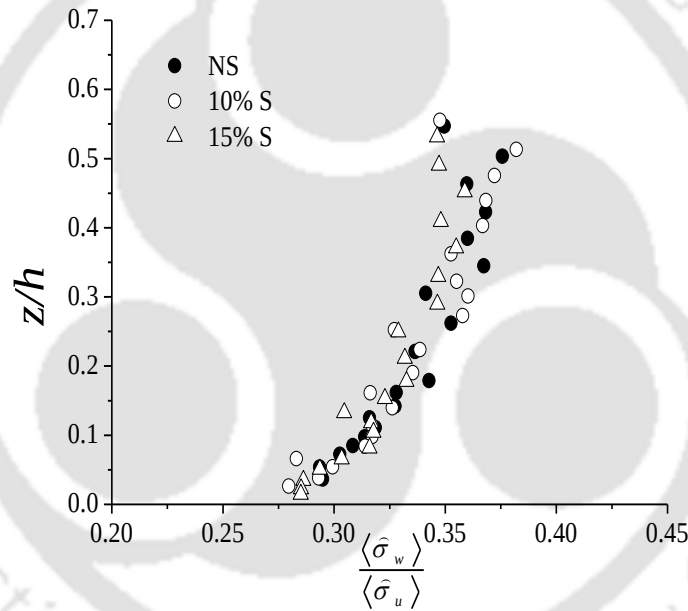


Figure 6: Vertical distributions of flow anisotropy

3.3. Turbulent Kinetic Energy (TKE) Flux

The vertical distributions of double averaged non-dimensional stream wise and vertical TKE flux

components $\langle F_{ku} \rangle = \frac{\langle f_{ku} \rangle}{u_*^3}$ and $\langle F_{kw} \rangle = \frac{\langle f_{kw} \rangle}{u_*^3}$ for no seepage and seepage runs are shown in

Figure 7 where f_{ku} and f_{kw} can be calculated as (**Raupach, 1981; Krogstadt and Antonia, 1999**):

$$\langle f_{ku} \rangle = 0.75 \left\langle \left(\overline{u'^3} + \overline{u'w'^2} \right) \right\rangle \quad (4)$$

$$\langle f_{kw} \rangle = 0.75 \left\langle \left(\overline{w'^3} + \overline{w'u'^2} \right) \right\rangle \quad (5)$$

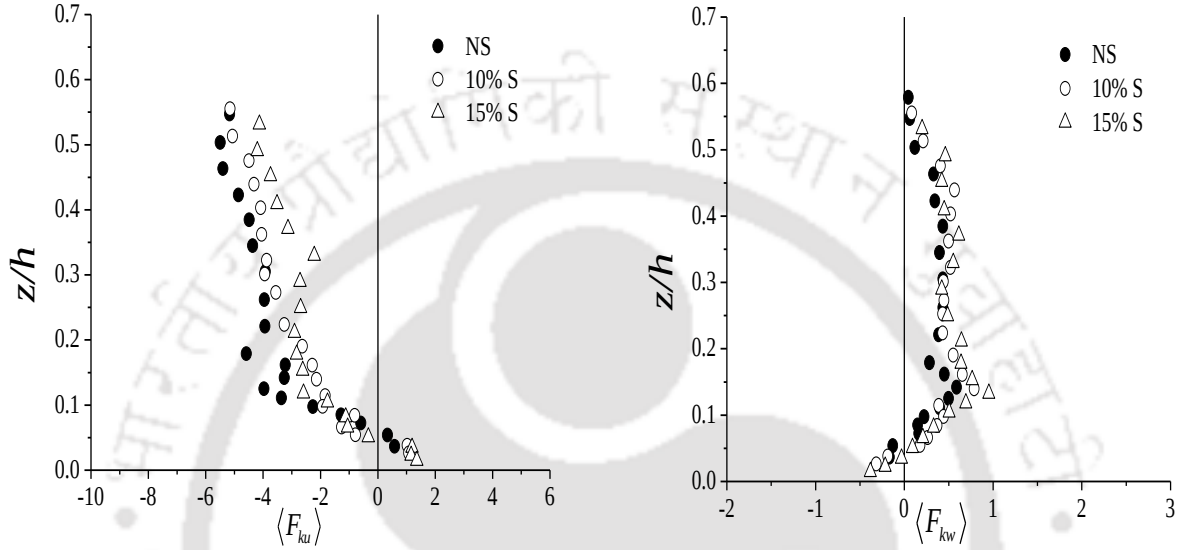


Figure 7: Vertical distributions of TKE Flux components ($\langle F_{ku} \rangle$ and $\langle F_{kw} \rangle$)

In NS runs, $\langle F_{ku} \rangle$ is positive at near bed and changing to negative value at away from the bed while $\langle F_{kw} \rangle$ is negative at near bed and changing to positive value at away from the bed. This implies that at near bed region, positive values of $\langle F_{ku} \rangle$ indicate the transport of turbulent kinetic energy transport in the stream wise direction while, negative values of $\langle F_{kw} \rangle$ suggest the transport of turbulent kinetic energy in the downward direction. While away from the bed, negative values of $\langle F_{ku} \rangle$ show the transport of turbulent kinetic energy is against stream wise direction and positive values of $\langle F_{kw} \rangle$ suggest the transport of turbulent kinetic energy is in the upward direction. The negative and positive values of the flux $\langle F_{ku} \rangle$ are related to the ejection–sweep character of the Reynolds stress (Maity and Mazumder, 2014). However, the behavioral characteristics of $\langle F_{ku} \rangle$ and $\langle F_{kw} \rangle$ for seepage runs are similar to those for no seepage flow.

Therefore, the most significant characteristic of a seepage flow is that $\langle F_{ku} \rangle$ starts with greater positive value in near bed region and less negative for the rest of the depth as compared to that of no seepage runs. Near bed, the distance of occurrence of negative $\langle F_{kw} \rangle$ and positive $\langle F_{ku} \rangle$ increases in the seepage runs. Positive $\langle F_{ku} \rangle$ and negative $\langle F_{kw} \rangle$ in no seepage runs at near bed are forming an accelerating effect as an inrush of fluid parcel which indicate the prevailing condition of the incipient motion of the bed particles while, greater positive $\langle F_{ku} \rangle$ and greater negative $\langle F_{kw} \rangle$ in near bed region for seepage runs imply the mobile bed condition with the application of downward seepage.

3.4. Quadrant Analysis

The conditional nature of the velocity fluctuations u' and w' is useful to quantify the bursting events. To quantify the bursting events, it is thus necessary to plot u' and w' pairs on $u' w'$ plane (**Lu and Willmarth, 1973**). The hyperbolic *hole region* H represents the threshold level and is determined by the curve $|u'w'| = H \left(\overline{u'^2} \right)^{0.5} \left(\overline{w'^2} \right)^{0.5}$ (**Nezu and Nakagawa, 1993**). A hole size parameter H categorizes to differentiate the larger contributions to $-\overline{u'w'}$ from each quadrant leaving the smaller u' and w' corresponding to more quiescent periods. Hence, a clear difference is obtained between the strong and the weak events depending on the hole size. The bursting events are generalised by four Quadrants (i) outward interactions ($i=1, u'>0, w'>0$), (ii) ejections ($i=2, u'<0, w'>0$), (iii) inward interactions ($i=3, u'<0, w'<0$) and (iv) sweeps ($i=4, u'>0, w'<0$) where i represent the Quadrants. At any point, the contribution to the total Reynolds stress from events excluding a hole size H is:

$$\langle u'w' \rangle_{i,H} = \lim_{T \rightarrow \infty} \frac{1}{T} \int_0^T u'(t)w'(t) I_{i,H}[u'(t)w'(t)] dt \quad (6)$$

Where t is time, T is sampling duration, $I_{i,H}$ is the indicator function define as

$$I_{i,H}[u'(t)w'(t)] = \begin{cases} 1, & \text{if } (u', w') \text{ is in quadrant } i \text{ and if } |u'w'| \geq H \left(\overline{u'^2} \right)^{0.5} \left(\overline{w'^2} \right)^{0.5} \\ 0, & \text{otherwise} \end{cases} \quad (7)$$

The double-averaging quadrant analysis provides the estimation of stress fraction transported by the contribution is

$$\langle S_{i,H} \rangle = \left\langle \frac{\langle u' w' \rangle_{i,H}}{\overline{u' w'}} \right\rangle \quad (8)$$

Here, $\langle S_{i,H} \rangle$ is positive when $i=2$ and 4 (Q2 and Q4) and $\langle S_{i,H} \rangle$ is negative when $i=1$ and 3 (Q1 and Q3). Hence, at a point, a sum of contributions from different bursting events is unity that is

$$\sum_{i=0}^{i=4} [\langle S_{i,H} \rangle]_{H=0} = 1. \text{ The DA fractional contributions } \langle S_{i,H} \rangle \text{ for } H=0 \text{ and } H=2 \text{ are plotted in}$$

Figure 8 respectively for flows subjected to no seepage and seepage runs. It is necessary to analyse all the data pairs ($H=0$) because the downward seepage affects the near bed flow by slowly extracting fluid through the bed. Still, the hole size $H=2$ is used to determine the stronger events. In Figure 8 with $H=0$, most of the contribution comes from ejection (Q2) and sweep (Q4) events while, outward (Q1) and inward interactions (Q3) contribute weakly towards the Reynolds shear stress production. Divergence in between Q2 and Q4 events is associated with the large scale of eddies. This trend is agreement with flows over smooth and rough beds (**Fer et al., 2004; Ojha and Mazumder, 2008; Mazumder et al., 2009**). In Figure 8 (NS case) with $H=0$, Q2 and Q4 events in the vicinity of bed contribute about 89% and 90% respectively towards the production of RSS while Q1 and Q3 events contribute as 44% and 35% respectively. The present results conclude that Q2 and Q4 events at near bed are nearly equal and opposite in nature, hence effectively cancel the effects of one another during their appearances in succession. This suggests that succession of high speed fluid parcel from the upper portion of near bed flow layer is compensated by the arrival of low speed fluid parcel within the near bed region. While away from the bed, Q2 events dominate in contributing to the Reynolds shear stress production. In contrast, for 10%S (Figure 8) with $H = 0$, Q2 and Q4 events within the near bed contribute about 82% and 86% respectively towards the production of RSS while Q1 and Q3 events contribute minimal as 36% and 30% respectively. With the increase of seepage to 15%, contribution from Q4 and Q2 events at near bed becomes 85% and 75% respectively. The result pointed that dominance of sweep events over the ejection events at near bed is responsible for sediment transport with seepage. When downward seepage is applied, the high speed fluid parcel becomes more persistently preserving close to the bed when compared with no seepage.

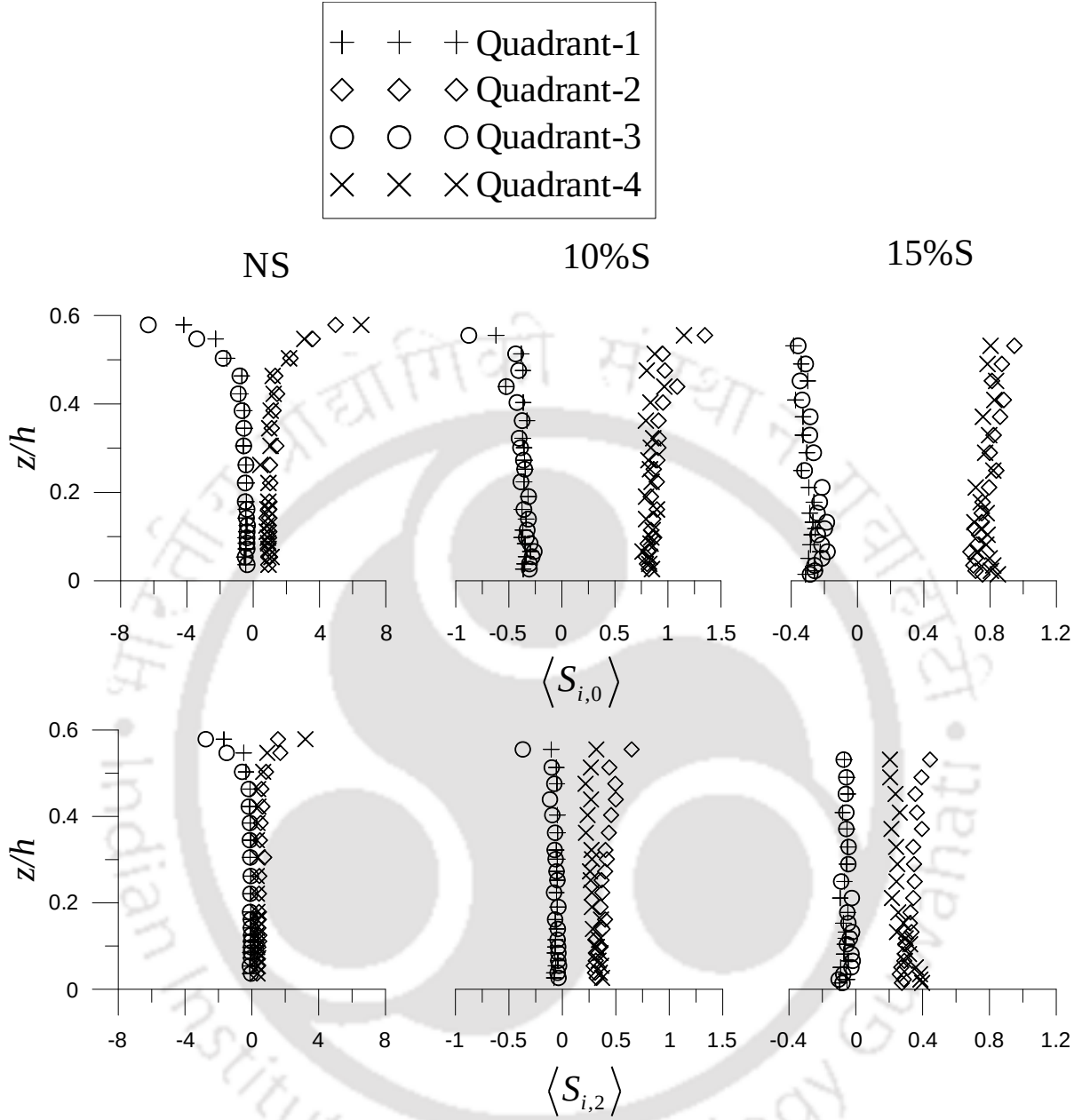


Figure 8: Profiles of conditional DA Reynolds shear stresses $\langle S_{i,0} \rangle$ for $H = 0$ and $\langle S_{i,2} \rangle$ for $H = 2$

In order to observe the vertical distribution of larger contributions, the analysis is also carried out with $H=2$ for both no seepage and seepage run (Figure 8). The most energetic Q2 and Q4 events have specific behaviour throughout the flow depth, since increased contributions of Q4 events are noticeable. However, in Figures 8 for $H=0$ and 2, there remains a similar predominating feature of Q2 and Q4 events within the near bed. In contrast, the contributions from Q1 and Q3 events remain insignificant. An important noticeable point is that $\langle S_{i,H} \rangle$ profiles are not matched

with τ_{uw}^+ profiles as $\langle S_{i,H} \rangle$ profiles do not introduce a damping within the near bed, as they represent the fractional contributions to the Reynolds shear stress. Overall, Q1 and Q3 events remain modest with the effect of seepage while reduction of Q2 events and an increment of Q4 events occur in a sequence: no seepage (NS) $\rightarrow$ 10% seepage (10%S) $\rightarrow$ 15% seepage (15%S).

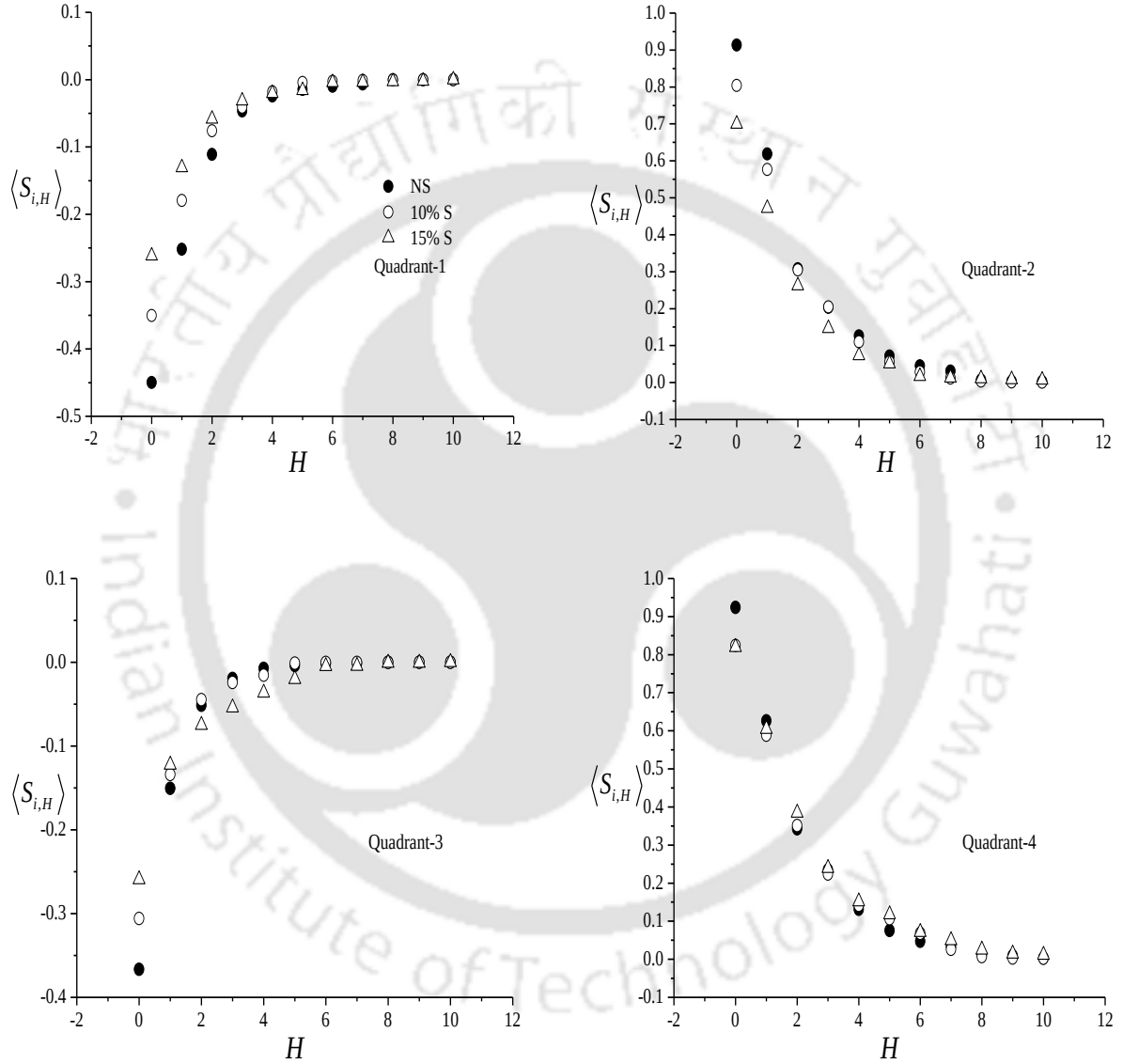


Figure 9: Variations of $\langle S_{i,H} \rangle$ with H

Variation of stress fraction $\langle S_{i,H} \rangle$ with the different hole size (H) for each of the four quadrants are plotted in Figure 9 to understand the contribution of larger events at $z/h=0.04$ (near the boundary) subjected to no seepage and seepage experiment. In no seepage case (Figure 9) for

different H , still governing events are Q2 and Q4 while Q1 and Q3 feebly contributes towards the production of RSS. Close observation of Figure 9 in no seepage case reveals that the fractional contribution from Q1 and Q3 events vanishes for $H > 7$. On the other hand, contribution from the Q2 and Q4 events vanishes for $H > 9$ and $H > 10$ respectively. However, $\langle S_{i,H} \rangle$ is slightly influenced by the seepage. An assessment of the variations of $\langle S_{i,H} \rangle$ with H reveals that reduction of Q2 events and increment of Q4 events occur in a sequence: no seepage (NS) $\rightarrow$ 10% seepage (10%S) $\rightarrow$ 15% seepage (15%S). As a result of the withdrawal of momentum associated with downward seepage in the near boundary region causes an increase in Q4 events, resulting in arrival of a high speed fluid parcel due to acceleration of fluid in stream wise direction. Also a slight reduction of contribution from Q2 events is apparent. In seepage runs, the contributions from both outward and inward interactions cease for $H > 8$ (10%S case) and $H > 9$ (15%S case). Ejection and sweep events dominate even for $H > 10$ with sweeps still being the largest contributor towards the momentum transfer.

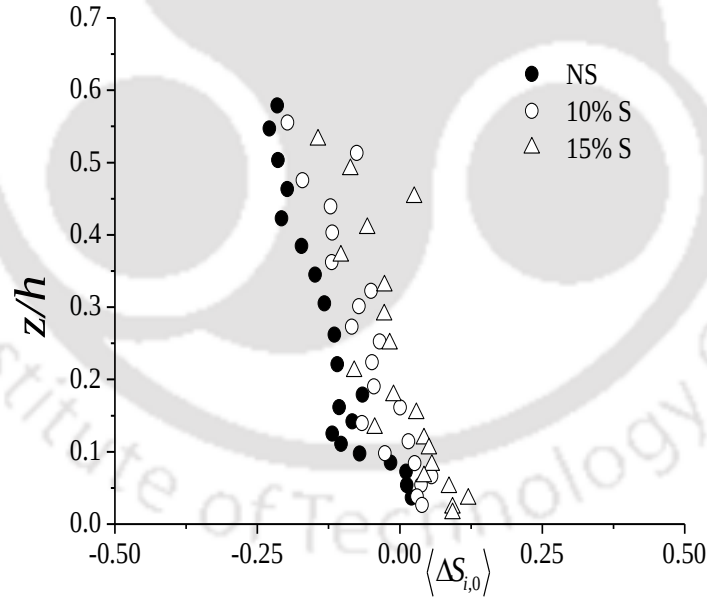


Figure 10: Vertical distributions of $\langle \Delta S_{i,0} \rangle$

Vertical distributions of the difference in the fractional contributions towards Reynolds shear stress production from Q4 and Q2 events ($\langle \Delta S_{i,0} \rangle = \langle S_{4,0} \rangle - \langle S_{2,0} \rangle$) is presented in Figure 10. It

is clear from Figure 10 that due to increase in thickness of roughness sub layer with the seepage runs, $\langle \Delta S_{i,0} \rangle$ also increases which clearly indicates the dominance of the sweep events in contribution towards the production of Reynolds shear stresses with the application of seepage.

3.5. DA Third Order Correlations of Velocity Fluctuations

There is a lack in understanding the third order correlations in flows subjected to seepage even though many studies on the time averaged turbulent flow characteristics over smooth and rough boundaries are carried out. To get the information on the flux of stress caused by the turbulence, DA third order correlation is essential. Third order correlation is directly proportional to the turbulent coherent structures due to conservation of their signs and produce necessary information on the temporal characteristics of the velocity fluctuations (**Gad-el-Hak and Bandyopadhyay, 1994**). The DA third order moments of velocity fluctuations can be expressed as (**Raupach, 1981**)

$$\left. \begin{aligned} M_{jk} &= \overline{\hat{u}^j \hat{w}^k} \text{ where } j+k=3 \\ \hat{u} &= \frac{u'}{(\overline{u'u'})^{0.5}}, \hat{w} = \frac{w'}{(\overline{w'w'})^{0.5}} \end{aligned} \right\} \quad (9)$$

Where $M_{30} = \overline{\hat{u}^3}$ represents the transport of $u'u'$ in stream wise direction, $M_{03} = \overline{\hat{w}^3}$ defines the transport of $w'w'$ in wall normal direction, $M_{12} = \overline{\hat{u}\hat{w}^2}$ represents the transport of $w'w'$ in stream wise direction, $M_{21} = \overline{\hat{u}^2\hat{w}}$ characterising the transport of $u'u'$ in wall normal direction (**Agelinchaab and Tachie, 2006**). Figures 11a-11d displays the individual profiles of the DA third order correlations $\langle M_{jk} \rangle$ for sets of no seepage and seepage run whose distributions are simply changed to some extent by the influence of seepage. In flows with no seepage and seepage, $\langle M_{30} \rangle$ initiates with small positive value and $\langle M_{03} \rangle$ starts with small negative value near the boundary as shown in the Figure 11. Magnitude of the positive $\langle M_{30} \rangle$ near the bed is increased with the increase of downward seepage suggesting the strong influence of increased stream wise flux of stream wise RNS which increases the particle mobility with the increased

seepage. Similar trend in the distribution of $\langle M_{12} \rangle$ is observed which reflects the increase in advection of vertical RNS in stream wise direction in seepage run.

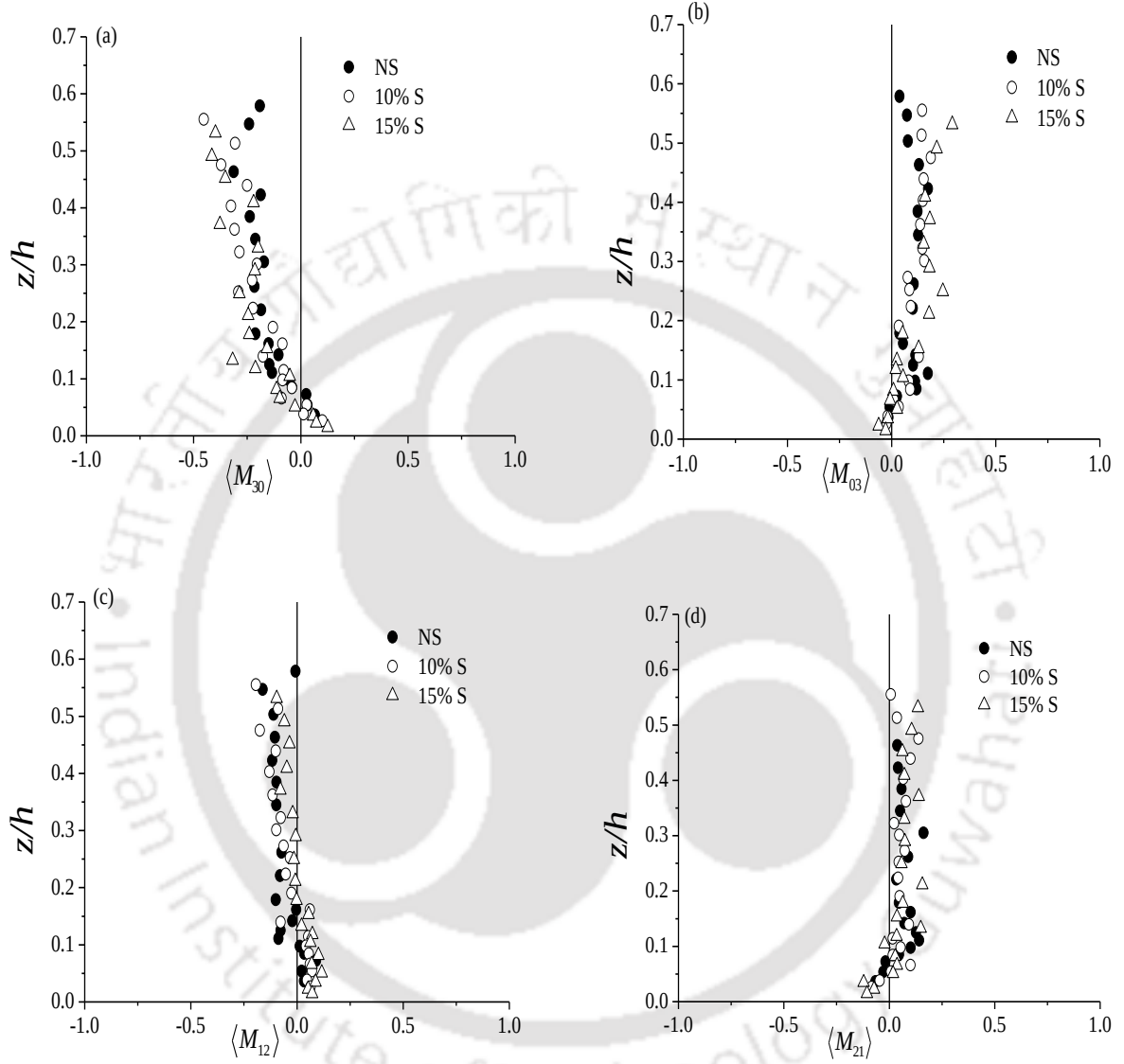


Figure 11: Vertical profiles of DA third order correlations

Magnitude of the negative values of $\langle M_{03} \rangle$ and $\langle M_{21} \rangle$ in the near bed region increases with the increased of downward seepage as compared to their negative values in no seepage run, suggests the increase in vertical flux of vertical RNS and also the increase in advection of stream wise RNS in downward direction. The effective features of the bursting events at a point can also probably recognized by the third order correlation (Nezu and Nakagawa, 1993). It is cleared

from Figures 11 that $\langle M_{30} \rangle$ and $\langle M_{03} \rangle$ are positive and negative respectively within the near bed implying the occurrence of Q4 events for both no seepage and seepage run. Away from the bed, the dominance of Q2 events causes $\langle M_{30} \rangle$ to be negative and $\langle M_{03} \rangle$ to be positive for the rest of the flow depth. This features were also observed by **Grass (1971)** and **Krogstadt and Antonia (1999)**. In addition, the transport of the Reynolds normal stresses is examined from $\langle M_{12} \rangle$ and $\langle M_{21} \rangle$ profiles (Figure 11). Within the near bed, the flows with no seepage and seepage correspond to a transport of vertical RNS in the stream wise direction and stream wise RNS in the vertical direction. Therefore, the present scenario provide a clear message that for no seepage and seepage runs, a stream wise acceleration is associated with a downward flux causing Q4 events. On the other hand, a stream wise acceleration is associated with a transport in the upstream direction, representing Q2 events.

As mentioned above, in order to understand the characteristics of bursting events, third order moments of velocity fluctuations are important parameters as they exhibit similar trend with $\langle \Delta S_{i,0} \rangle$ (**Raupach, 1981**). Variation of $\langle M_{30} \rangle$ with $\langle \Delta S_{i,0} \rangle$ is plotted in Figure 12 for no seepage and seepage runs.

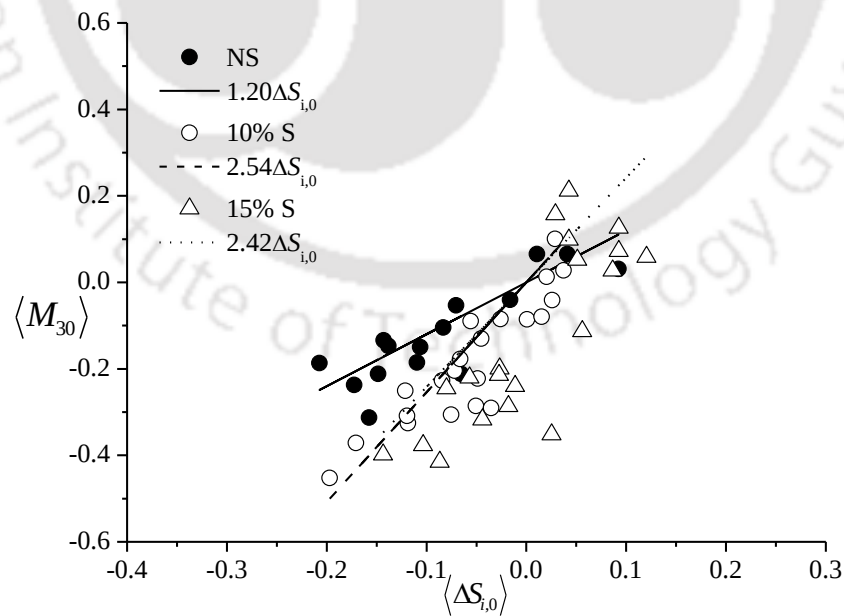


Figure 12: Relation between $\langle M_{30} \rangle$ and $\langle \Delta S_{i,0} \rangle$

It is cleared from the Figure 12 that respective of the roughness sub layer caused by the change in flow conditions (application of downward seepage), there exist a linear relationship between $\langle \Delta S_{i,0} \rangle$ and $\langle M_{30} \rangle$. However, after applying downward seepage of different percentage on a plane sand bed, the linear relationship between $\langle \Delta S_{i,0} \rangle$ and $\langle M_{30} \rangle$ becomes

$$\langle M_{30} \rangle = \begin{cases} 1.2 \langle \Delta S_{i,0} \rangle & \text{for NS case} \\ 2.54 \langle \Delta S_{i,0} \rangle & \text{for 10\%S case} \\ 2.42 \langle \Delta S_{i,0} \rangle & \text{for 15\%S case} \end{cases} \quad (10)$$

3.6. DA Turbulent Kinetic Energy (TKE) Budget

The DA TKE budget for a uniform flow over a sand bed is given as follows (**Mignot et al., 2009**)

$$-\langle \overline{u'w'} \rangle \frac{\partial \langle u \rangle}{\partial z} = \langle \varepsilon \rangle + \frac{\partial \langle f_{kw} \rangle}{\partial z} + \frac{1}{\rho} \frac{\partial \langle \overline{p'w'} \rangle}{\partial z} - \nu \frac{\partial^2 \langle K \rangle}{\partial z^2} \quad (11)$$

Where $t_p = -\langle \overline{u'w'} \rangle \frac{\partial \langle u \rangle}{\partial z}$ is TKE production rate, ε is TKE dissipation rate, $t_D = \frac{\partial \langle f_{kw} \rangle}{\partial z}$ is TKE

diffusion rate, $p_D = \frac{1}{\rho} \frac{\partial \langle \overline{p'w'} \rangle}{\partial z}$ is pressure energy diffusion rate, $v_D = \nu \frac{\partial^2 \langle K \rangle}{\partial z^2}$ is viscous

diffusion rate, ν is kinematic viscosity, p' is the pressure fluctuations and K is the TKE. The viscous diffusion v_D contributes insignificantly in turbulent flow over the rough boundary. The turbulent dissipation ε is evaluated by methods as follows (**Krogstadt and Antonia, 1999**):

$$\varepsilon = \frac{15\nu}{u^2} \left(\frac{\partial u'}{\partial t} \right)^2 \quad (12)$$

The pressure energy diffusion rate p_D is calculated from Equation (11) as $p_D = t_p - \varepsilon - t_D$. The

parameters of TKE budget are normalized in the form as $T_p, E_D, T_D, P_D = (t_p, \varepsilon, t_D, p_D) \times \left(\frac{h}{u_*^3} \right)$.

Figures 13a-13d presented the DA TKE budget in flows subjected to no seepage and seepage

run. **Nezu and Nakagawa (1993)** gave a theoretical expression for the TKE production rate and TKE dissipation rate in the normalized form as

$$T_p = \frac{1}{k} \left(\frac{1 - \hat{z}}{\hat{z}} \right) \quad (13)$$

$$E_D = \frac{9.8}{(\hat{z})^{0.5}} \exp(-3\hat{z}) \quad (14)$$

Where, k is the von Karman constant and $\hat{z} = \frac{z}{h}$. In Figure 13a, the theoretical $\langle T_p \rangle$ with no seepage and seepage case is in good agreement with the experimental data plots. $\langle T_p \rangle$ is increased especially in the near bed region with the seepage run as compared to no seepage, suggesting the increased level of turbulence with the application of downward seepage. This increased level of turbulence is connected with the increased turbulence intensities and increased momentum transfer with downward seepage. Dissipation of turbulent kinetic energy ($\langle E_D \rangle$) is reduced in magnitude or becoming less noticeable. With the application of seepage, more energy is transferred to turbulent fluctuations; the gain in turbulent kinetic energy becomes higher while, its dissipation reduces. Figure 13b clearly indicates that in seepage run, dissipation of turbulent kinetic energy is less with the increased seepage as compared to that with no seepage. Over the entire flow depth, the distributional patterns of $\langle T_p \rangle$ and $\langle E_D \rangle$ have a difference such that for a given z/h , $\langle T_p \rangle < \langle E_D \rangle$ for no seepage case. With the presence of downward seepage, the distributional patterns of $\langle T_p \rangle$ and $\langle E_D \rangle$ have a difference such that $\langle T_p \rangle > \langle E_D \rangle$ in the vicinity of bed and the difference between them decreases with an increase in z/h . Positive $\langle T_p \rangle$ indicates the conversion of TKE from the time averaged flow to turbulence where negative $\langle T_p \rangle$ corresponds to conversion of TKE from turbulence to time averaged flow. Turbulent transport or diffusion plays an important role in the TKE budget. Turbulent diffusion has the most important role in governing the ejection and sweep events, which are the key phenomena for generating vortices (**Devi and Kumar, 2016**). Figure 13c pointed out that turbulence diffusion ($\langle T_D \rangle$) is reduced in the near bed region with respect to seepage run due to gain in turbulence production and subsequent reduction in the dissipation of turbulent kinetic energy with the application of

downward seepage. **Nezu and Nakagawa (1993)** observed that T_P nearly equals E_D and T_D is almost balanced by P_D in flows over smooth and rough beds. The most interesting feature lies on the near bed distributions of $\langle P_D \rangle$ that it is increased with respect to seepage which suggest the mobility of bed particles due to gain in turbulence production (Table 3).

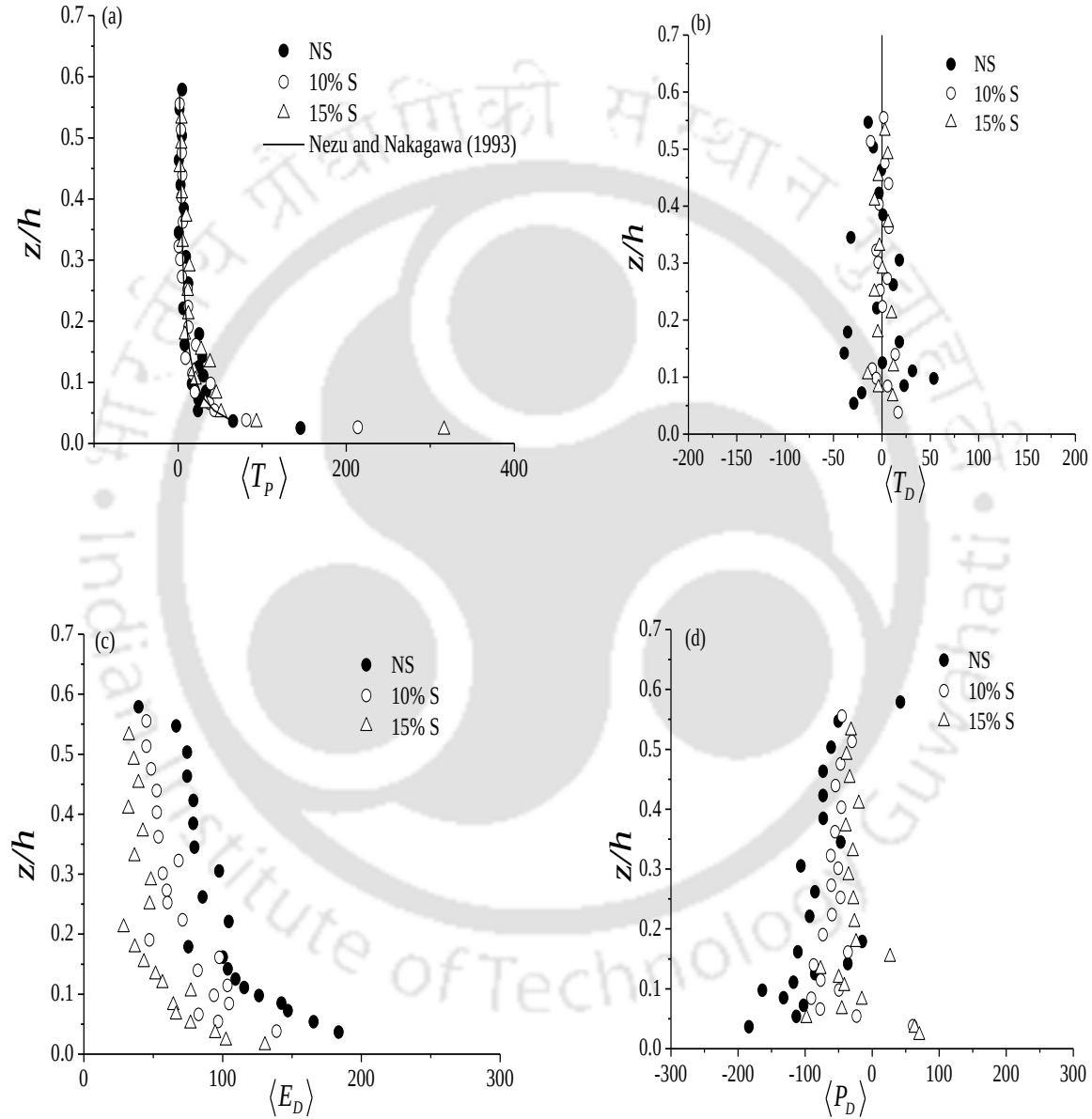


Figure 13: Vertical profiles of DA TKE budget

3.7. DA Fourth Order Moments (Kurtoses)

Fourth order moments represent the intermittency of turbulence. Figure 14 shows the vertical

distribution of fourth order moment $\langle K_u \rangle = \left\langle \frac{\overline{u'^4}}{\sigma_u^4} \right\rangle$ in the stream wise direction subjected to no

seepage and seepage run. It is observed that kurtosis becomes minimum at $\frac{z}{h} \cong 0.08$ after which

the value is going to be increased in both inner and outer region of turbulent boundary layer which was also found by **Andreopoulos et al. (1984)**. It suggests that turbulence is highly

intermittent in both layers which are in conformity with **Grass (1971)** that sweep and ejection provides strong intermittent contribution to the turbulence production. Close examination of

Figure 14 suggest that $\langle K_u \rangle > 3$ in both inner and outer layer denotes to a distribution with a peaky signal characteristic of intermittent turbulent events, while at the point of contra flexure of inner and outer layer i.e at $\frac{z}{h} \cong 0.08$, $\langle K_u \rangle < 3$ states to a flat characteristic. Kurtosis is increased

in the vicinity of bed with the application of seepage which indicates that level of turbulence intermittency is higher with seepage as compared to that with no seepage.

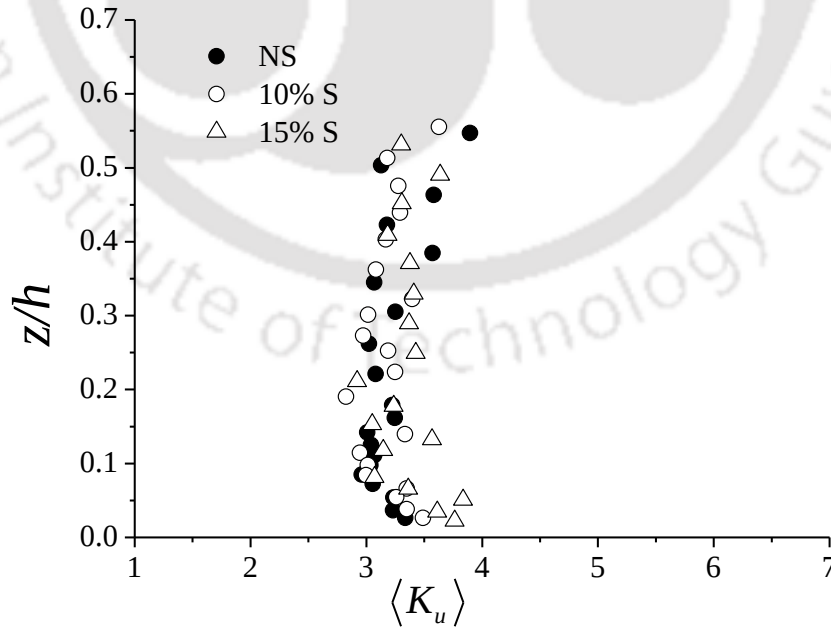


Figure 14: Vertical profiles of DA Kurtoses

3.8. DA Taylor Micro Scale Distributions

Taylor scale is calculated based on the Kolmogorov's second similarity hypothesis (Pope, 2000) which states that in every turbulent flow at sufficiently high Reynolds number, the statistics of motion of scale have an universal form that is uniquely determined by turbulent dissipation rate (ε) and independent of kinematic viscosity (ν). Taylor micro scale λ_t defines eddy size in the inertial sub range and can be expressed as:

$$\langle \lambda_T \rangle = \left(15\nu \left\langle \frac{\sigma_u^2}{\varepsilon} \right\rangle \right)^{0.5} \quad (14)$$

Where, ε is the TKE dissipation rate estimated from the Equation (12). The normalized Taylor scale $\langle \hat{\lambda}_T \rangle = \frac{d_{50}}{\langle \lambda_T \rangle}$ is the variation of the ratio of sediment size to Taylor micro scale with respect to depth as shown in Figure 15 subjected to no seepage and seepage run.

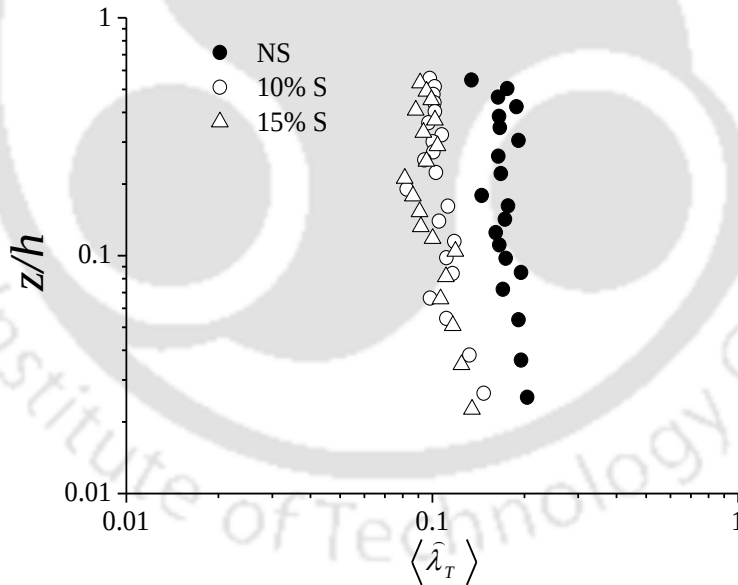


Figure 15: Vertical profiles of ratio of particle size to Taylor micro scale $\langle \hat{\lambda}_T \rangle$

The decrease in the turbulent kinetic energy dissipation rate (ε) and decrease in normalized turbulent intensity in stream wise direction with seepage flow leads to the reduction of $\langle \hat{\lambda}_T \rangle$ as compared to that of no seepage throughout the flow depth. Also it is observed that $\langle \hat{\lambda}_T \rangle$ for both

no seepage and seepage case almost decreases away from the bed. The values of $\langle \lambda_T \rangle$ near bed is about 47% greater with 10%S and about 59% greater with 15%S, as compared to no seepage. This increased in the eddy size close to the bed causes the subsequent increase in bed particle mobility in seepage flow as compared to no seepage run (Table 3)

3.9. DA Integral Scale

Gyr and Schmid (1989) reported that initiation of bed features are formed because of the dominance of turbulent sweep events in the near bed region are directly correlated with coherent structures of the turbulent flow. **Best (1992)** pointed out that these bed features are developed by coherent structure which keep the bed in mobile condition. The origin of bed deformation is linked with integral scale of flow in near bed region (**Venditti et al., 2005; Patel et al., 2015**). In recent study, **Roy et al. (2017)** investigated the distribution of eddy scales and intermittency of turbulence at different levels. In the present experiments, bed load transport was observed at the downstream end of the channel for both no seepage and seepage run. The time scale and integral length scale are calculated for the near bed velocity of the no seepage, 10% seepage and 15% seepage runs in order to describe the changes in the bed load transport condition after the application of seepage. Integral time scale and length scale are calculated at ~4.5mm above the bed. An integral length scale indicates the large eddy size in the flow and corresponding time scale suggests the eddy turnover time at a given point. The transfer of momentum and turbulent kinetic energy in the flow are associated with these eddies. The Eulerian integral time scale is expressed as

$$\langle E_T \rangle = \left\langle \int_0^a R(t) dt \right\rangle \quad (15)$$

Where, $R(t)$ is the autocorrelation function, dt is the lag distance, and a is the time at which $R(t)$ started to oscillate about zero (**Tennekes and Lumley, 1972**). In order to convert the time series into regularly spaced events, Autocorrelation is calculated using linear interpolation. Value of a is calculated based on autocorrelation at which $R(t) \approx 0.01$ and lag time is 0.01 s. The Eulerian integral length scale (**Taylor, 1935**) is defined as:

$$\langle E_L \rangle = \langle E_T u \rangle \quad (16)$$

Table 5: Values of DA integral length scale and time scale

Flow ($Q_0=0.0429 \text{ m}^3/\text{s}$)	Eulerian time scale $\langle E_T \rangle$	Eulerian length scale $\langle E_L \rangle$
	(sec)	(m)
NS	0.287	0.063
10%	0.388	0.094
15%S	0.409	0.102

Value of $\langle E_T \rangle$ and $\langle E_L \rangle$ are represented in Table 5 which shows that eddy length and eddy turnover time in the vicinity of bed are increased significantly with downward seepage. **Venditti et al. (2005)** suggested that larger size of eddies in the vicinity of bed is responsible for initiation of bed features. Increased eddy sizes in the near bed region correspond to higher momentum and energy transfer and less destruction of turbulent motions. Thus higher levels of turbulence prevail near the bed zone with an increased eddy size, which results in higher bed load transport with downward seepage (Table 3).

3.10. DA Turbulent Mixing Length

Turbulent Mixing length (L) is the distance at which fluid parcels keeps its original characteristics before dispersing into surrounding fluid, is given by **(Krogstadt and Antonia, 1999)**:

$$\langle l^+ \rangle = \frac{(\langle \tau_{uw}^+ \rangle)^{0.5}}{\left\langle \frac{\partial u^+}{\partial z^+} \right\rangle} \quad (17)$$

Where, $l^+ = \frac{l}{h}$, $u^+ = \frac{u}{u_*}$ and $z^+ = \frac{z}{h}$.

The measured double averaged velocity profiles are used to calculate the DA velocity gradients

$\left\langle \frac{\partial u^+}{\partial z^+} \right\rangle$ by smooth curve fitting to the data, and the values of $\langle \tau_{uw}^+ \rangle$ are obtained directly from the

measured RSS distributions in order to determine the mixing length $\langle l^+ \rangle$ from Equation (17).

The distribution of the DA non-dimensional turbulent mixing length is depicted in Figure 16 for no seepage and seepage runs.

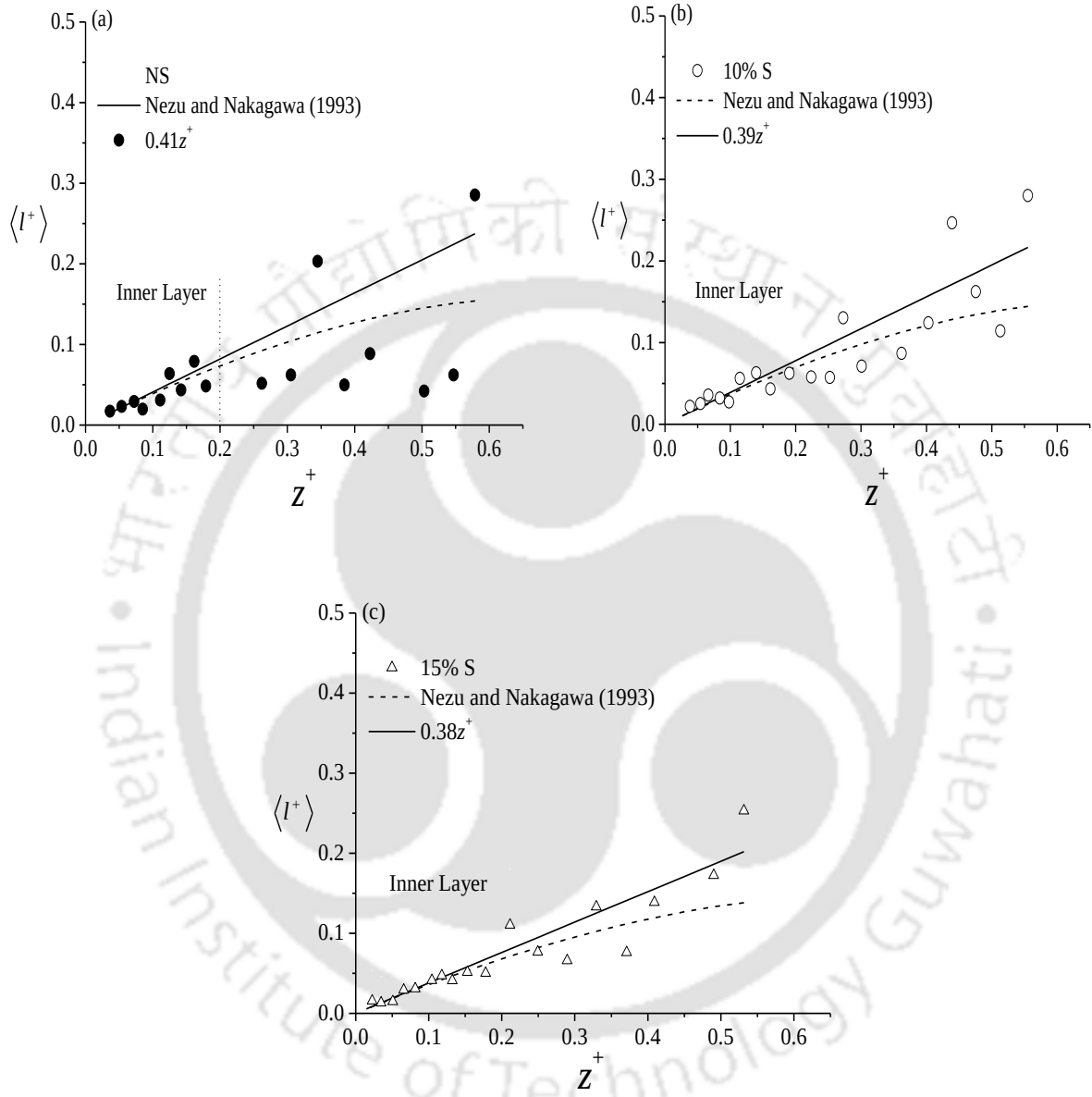


Figure 16: Distribution of Turbulent mixing length of flow for (a) NS (b) 10%S and (c) 15%S

In the inner flow layer ($0 < z/h < 0.2$) there is an acceptable linear fit between $\langle l^+ \rangle$ and z^+ :

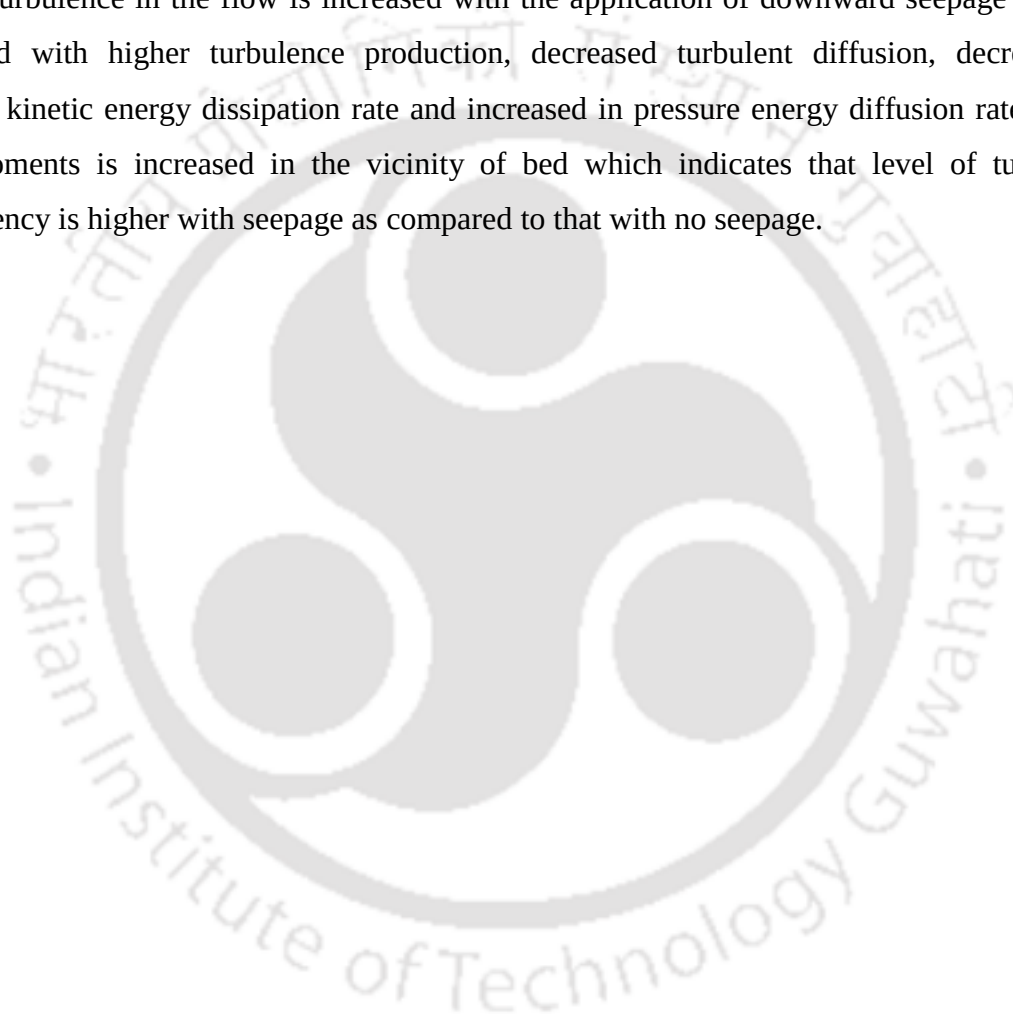
$$\langle l^+ \rangle = kz^+ \quad (18)$$

Where k is von Karman constant and slope of this linear fit, the experimental data points for no seepage and seepage flows collapse reasonably on a single band, which is in conformity with Prandtl's hypothesis. Also, data points collapse satisfactorily on the curves obtained from the theoretical equation $\langle l^+ \rangle = kz^+(1 - z^+)^{0.5}$ given by **Nezu and Nakagawa (1993)**. From Figure 16a-16c, it is clear that the slope of the linear fit (von Karman's constant) decreases with the effect of seepage. This decreased in the slope suggests that the traversing length of turbulent eddy decreases, which signifies the increase in bed load transport in the channel under the action of downward seepage. It is interesting point to note from Figure 16a-16c that observed value of von Karman constant is same as observed value from the velocity logarithmic law (Figure 1b) for both no seepage and seepage cases.

4. Conclusions

In the present study, the experimental results of the DA turbulence characteristics in sand bed streams subjected to no seepage and downward seepage are analysed. The profiles of DA stream wise velocities in the near bed region increases with the application of downward seepage. Increase in the depth of the virtual bed level and the zero velocity level reflects that the particles resting on the channel boundary are exposed to higher component of stream wise velocity. Value of von Karman's constant reduces with seepage, which corresponds to increase in the bed load transport. The profile of total shear stress is also increased with seepage which signifies the enhanced momentum transfer towards the boundary. Near bed, the profiles of DA Reynolds shear stress undergo a damping due to a decreasing level of turbulence fluctuation. The maximum value of Reynolds shear stress is greater with seepage and subsequent increase in shear velocities suggests greater momentum transfer towards the boundary in seepage runs. The form induced shear stress has a decelerating effect with no seepage and an accelerating effect with seepage. The stream wise and vertical turbulence intensities in presence of seepage are greater than the corresponding turbulence intensities in no seepage. The turbulence is strongly anisotropic being invariant of seepage; varying almost linearly with vertical distance. Both stream wise and vertical flux of kinetic energy in seepage are greater than those of no seepage. Quadrant analysis for stronger events shows that contribution of sweep events towards Reynolds shear stress production is increased near the boundary with seepage. The occurrences of the bursting events obtained from the quadrant analysis are also confirmed by the analyses of the

third order correlations corresponding to different seepage conditions. The distributions of third order moment suggest that sweeps are prevalent close to the bed associated with the arrival of high speed fluid parcel. The application of seepage influences the near boundary data of third order correlation by increasing stream wise flux and the vertical advection of stream wise Reynolds normal stress and also increasing the vertical flux and the stream wise advection of vertical Reynolds normal stress. The effect of seepage on the TKE budget is pronounced. The level of turbulence in the flow is increased with the application of downward seepage which is associated with higher turbulence production, decreased turbulent diffusion, decreased in turbulent kinetic energy dissipation rate and increased in pressure energy diffusion rate. Fourth order moments is increased in the vicinity of bed which indicates that level of turbulence intermittency is higher with seepage as compared to that with no seepage.



Appendix B

Probability Distribution Functions of Turbulence Using Multiple Criteria over Non-uniform Sand Bed Channel

1. Introduction

Probability density function (PDF) of an arbitrary variable is a function that defines the relative likelihood for this random variable to deal with a given value. Probability distribution function can be represented by Gaussian distribution, Log normal distribution and many other etc. These distributions are based on the interval over which the convergence is finite. The advantage of Gram Charlier (GC) series to other series is that it is applicable to series containing interval over which the application of convergence is infinite (**Milne, 1929**). Also PDFs of GC series is developed considering the higher order moments up to four (**Cramer, 1999**). The knowledge of PDF of turbulent statistics such as velocity fluctuation and Reynolds shear stress helps in understanding the related turbulent events (**Nakagawa and Nezu, 1977; Afzal et al., 2009**). Considering non-Gaussian nature of velocity fluctuation in stream wise and vertical direction, **Antonia and Atkinson (1973)** followed GC series expression in order to study the PDFs of velocity fluctuation achieved by reversing a Gaussian based characteristics function. **Nakagawa and Nezu (1977)** used GC based PDF in the form given by **Feriet and Kampe (1966)**. The problems of calculating the probability density using a sequence of random variables has received the significant courtesy during the last decades. Many studies have been recommended including the kernel method of **Rosenblatt (1956)** and **Parzen (1962)**, the orthogonal series method of **Kronmal and Tarter (1968)**, the histogram method of **Van Ryzin (1973)**, the polynomial interpolation of **Wahba (1971, 1975)**, the Fourier transform method of **Blum and Susarla (1976)**, and the Hermite series method of **Schwartz (1967)**. Most of the methods are based on cumulants such as kurtosis and skewness. Cumulant-based probability measures can be inspired by the concept of Hermite polynomials. In the field of mathematics, the Hermite

polynomials are a standard orthogonal polynomial sequence. The polynomials arise in different study such as probability, combinatorics, numerical analysis, and physics and systems theory. When an appropriate polynomial chain is used to estimate the PDF near a Gaussian density, cumulants are found obviously as the coefficients in the chain (**Jones and Sibson, 1987; Comon, 1994**). The advantage of this method is that we can keep a random number of terms in the series and thus it may be possible to get very general approximations. **Maity and Mozumdar (2014)** used GC series Hermite polynomials for PDFs of turbulent parameters in the region of crescentic scour. In the recent study, **Sharma and Kumar (2016)** experimentally investigated the PDFs of flow parameters in alluvial channel and observed that PDF obtained with the experimental data followed the theoretical curves based on GC series exponential distribution.

The PDFs can be used to compute the change of turbulent flows extending from shear layer of incompressible turbulent flows (**Pope, 1985**) to turbulence flows with high velocity fluctuations. As observed in open channel flow, turbulence seems to be quite changeable and offers a large series of scales. The statistical characteristics of any signal observed in turbulence for experimental conditions are highly reproducible (**Andreotti and Douady, 1999**). This elementary remark has motivated researchers to search for a probabilistic explanation of turbulence. Thus, PDFs of numerous turbulent features (velocity fluctuations and Reynolds shear stress) should be measured both experimentally and theoretically. As discussed in the literature review, GC series exponential distribution and GC series Hermite polynomials are important in the calculation of PDFs in their respective field, but the question arises which method is more superior in the field of sediment transport in an alluvial channel. Hence, the primary objective of this study is to use both exponential and Hermite GC series PDFs separately in order to estimate the probability distribution function of turbulence quantities such as velocity fluctuation and Reynolds shear stress. These theoretical based PDFs will be compared with observed data sets for evaluation of PDFs given by exponential and Hermite GC series for the complete range of data sets exist in a time series.

2. Theoretical Background of PDFs

In order to calculate the joint probability density of velocity fluctuation (u' , w'), we have used the GC series exponential distribution and GC series Hermite polynomials at which u' and w' is normalized as $\hat{u} = u'/\sigma_u$ and $\hat{w} = w'/\sigma_w$ where, σ_u and σ_w are the stream wise and vertical

turbulence intensity respectively. The joint probability density function (PDF) based on GC series exponential distribution for stream wise and vertical velocity fluctuation is expressed in the following form of equations (Nezu, 1977),

$$P_E(\hat{u}, \hat{w}) = \frac{1}{4\pi^2} \sum_{j=0}^{\infty} \sum_{k=0}^{\infty} i^{j+k} \alpha_{jk} I_j(\hat{u}) I_k(\hat{w}) \quad (1)$$

Where $i = \sqrt{-1}$, considering GC series of third order ($j+k=3$), the integral $I_j(\hat{u})$ and $I_k(\hat{w})$ are evaluated as per Gradshteyn and Ryzhik (1980) formulas:

$$\left. \begin{aligned} I_0(\hat{u}) &= \pi \exp(-|\hat{u}|) \\ I_1(\hat{u}) &= -\frac{i\pi}{2} \hat{u} \exp(-|\hat{u}|) \\ I_2(\hat{u}) &= \frac{\pi}{8} (1 + |\hat{u}| - \hat{u}^2) \exp(-|\hat{u}|) \\ I_3(\hat{u}) &= -\frac{i\pi}{48} \hat{u} (3 + 3|\hat{u}| - \hat{u}^2) \exp(-|\hat{u}|) \end{aligned} \right\} \quad (2)$$

$$\left. \begin{aligned} I_0(\hat{w}) &= \pi \exp(-|\hat{w}|) \\ I_1(\hat{w}) &= -\frac{i\pi}{2} \hat{w} \exp(-|\hat{w}|) \\ I_2(\hat{w}) &= \frac{\pi}{8} (1 + |\hat{w}| - \hat{w}^2) \exp(-|\hat{w}|) \\ I_3(\hat{w}) &= -\frac{i\pi}{48} \hat{w} (3 + 3|\hat{w}| - \hat{w}^2) \exp(-|\hat{w}|) \end{aligned} \right\} \quad (3)$$

The coefficient α_{jk} in Equation (1) is related to third order moment $M_{jk} = \overline{\hat{u}^j \hat{w}^k}$ in such a way that:

$$\left. \begin{aligned} \alpha_{00} &= M_{00}, \alpha_{10} = M_{10}, \alpha_{20} = \frac{1}{2} M_{20} - 1, \alpha_{30} = \frac{1}{6} M_{30} - 2M_{10}, \alpha_{12} = \frac{1}{2} M_{12} - M_{10} \\ \alpha_{11} &= M_{11}, \alpha_{01} = M_{01}, \alpha_{02} = \frac{1}{2} M_{02} - 1, \alpha_{03} = \frac{1}{6} M_{03} - 2M_{01}, \alpha_{21} = \frac{1}{2} M_{21} - M_{01} \end{aligned} \right\} \quad (4)$$

The time averaged of $u'w'$ derived the Reynolds stress $\tau = -\overline{\rho u'w'}$ due to which the PDFs of τ depends upon the joint PDFs of u' and w' . To study the PDFs of τ , normalized random variable $\hat{\tau} = \hat{u}\hat{w}$ has been considered. Nezu (1977) derived the PDFs of $\hat{\tau}$ as:

$$P_E(\hat{\tau}) = k_0(2\tau_1) - \frac{1}{8}(\alpha_{20} + \alpha_{02})(1 - \tau_1^2)k_0(2\tau_1) + \frac{1}{4}\alpha_{11}\hat{\tau}k_0(2\tau_1) \quad (5)$$

Where $\tau_1 = (|\hat{\tau}|)^{0.5}$, k_0 is the modified Bessel function of second kind and order 0. The coefficient in the above expression can be estimated by the Equation (4).

The joint probability density function (PDF) based on GC series Hermite polynomial for stream wise and vertical velocity fluctuation is expressed in the following form of equations (**Maity and Mazumder, 2014**)

$$P_H(\hat{u}, \hat{w}) = \psi(\hat{u}, \hat{w}) \left[1 + \sum_{j+k=3}^4 \frac{q_{jk}}{j!k!} H_{jk}(\hat{u}, \hat{w}) \right] \quad (6)$$

Where q_{jk} represents the cumulant of the corresponding third order moment M_{jk} and $\psi(\hat{u}, \hat{w})$ is the Gaussian distribution for two independent variables and can be calculated as

$$\psi(\hat{u}, \hat{w}) = \frac{1}{2\pi\sqrt{1-S^2}} \exp \left\{ -\frac{\hat{u}^2 + 2S\hat{u}\hat{w} + \hat{w}^2}{2(1-S^2)} \right\} \quad (7)$$

Where S is the correlation coefficient and can be expressed as

$$S = -\left(\frac{\overline{u'w'}}{\sqrt{u'u'}\sqrt{w'w'}} \right) \quad (8)$$

The $H_{jk}(\hat{u}, \hat{w})$ in Equation (6) is a Hermite polynomial of order $(j+k)$ and can be expressed as (**Gradshteyn and Ryzhik, 1980**)

$$\left. \begin{aligned} H_{12} &= \hat{u}\hat{w}^2 - \hat{u}M_{02} - 2\hat{w}M_{11} \\ H_{21} &= \hat{u}^2\hat{w} - \hat{w}M_{20} - 2\hat{u}M_{11} \\ H_{03} &= 8\hat{w}^3 - 12\hat{w} \\ H_{30} &= 8\hat{u}^3 - 12\hat{u} \end{aligned} \right\} \quad (9)$$

In order to calculate the PDFs of Reynolds stress, we have normalized the Reynolds stress

$\tau_n = \frac{u'w'}{u'u'}$ of random variable. The probability distribution $P_H(\tau_n)$ of τ_n can be expressed as

(**Maity and Mazumder, 2014**)

$$P_H(\tau_n) = \frac{S}{\pi} \exp \left(\frac{S^2 \tau_n}{1-S^2} \right) \frac{k_0 \left(\left| \frac{S \tau_n}{(1-S^2)} \right| \right)}{\sqrt{1-S^2}} \quad (10)$$

3. Verification with Experimental Data

Here, results have been presented for no seepage discharge ($0.0488\text{m}^3/\text{s}$) at a bed slope 0.001 to avoid duplicity and redundancy in the findings. The above mentioned theory permitted to calculate the joint PDFs of velocity fluctuation and Reynolds stress by GC series exponential and Hermite polynomial method for two variables. This analysis was performed only at a location ($z/h=0.4$) to estimate the contributions of events to the joint PDFs of velocity fluctuation and Reynolds shear stress. Since PDFs obtained from experimental data will provide a complete numerical description in the areas of turbulence at a definite sum of points, it should be useful to clarify facts of PDFs.

3.1. Joint PDFs of Velocity Components

In this section, we will calculate joint PDFs of velocity fluctuation based on GC series exponential distribution and Hermite polynomials and verified with experimental data. The experimental data sets are used to determine the coefficient α_{jk} mentioned in Equation (1). The joint relative frequencies $f(\hat{u}, \hat{w})$ of arbitrary variable $(\hat{u}, \hat{w})$ are calculated from the experimental data at a vertical distance z to estimate the theoretical curves for the joint PDFs of $P_E(\hat{u}, \hat{w})$. In order to calculate the moment M_{jk} , the PDF $P_E(\hat{u}, \hat{w})$ is approximated by $f(\hat{u}, \hat{w})$ and the composite Simpson's rules are used to evaluate the integrals. In this manner, Equation (4) is used to estimate the coefficient α_{jk} and the joint PDF is computed to produce the theoretical curve for $P_E(\hat{u}, \hat{w})$. The coefficient α_{jk} are tabulated in Table 1. Because of inhomogeneity and anisotropic nature of turbulent flows, it is difficult to model for such flows in near bed zone, a measurement location ($z=0.4h$) is selected for the study of joint PDF $P(\hat{u}, \hat{w})$. The contours of $P_E(\hat{u}, \hat{w})$ and $P_H(\hat{u}, \hat{w})$ for both no seepage, 10% seepage and 15% seepage computed from Equation (1) and Equation (6) respectively are drawn in Figure 1, 2 and 3 respectively. The experimental data trends are used to calculate joint PDFs for both no seepage and seepage cases. The results shows that contour of joint PDFs distributions for velocity fluctuations based on GC series exponential distributions and Hermite polynomial follows the universality that it is applicable for no seepage as well as seepage cases. Figure 1, 2 and 3 shows that as the PDFs calculated from GC series

based Hermite polynomial and GC series based exponential distribution, there is slight variation of shape from the circular to square, whereas experimentally calculates joint PDFs displays extreme variation of shape to elliptic shape inhabiting the second and fourth quadrants suggesting the downward momentum flux (Maity and Mazumder, 2014) . It is noticed that the contour orientation has been changed cyclically from quadrant to quadrant based on bottom shear stress. The contour of $P_H(\hat{u}, \hat{w})$ shown in Figures 1a, 2a and 3a represents a circular shape which equally favours all four quadrants and follow the experimental results shown in Figures 1b, 2b and 3b while contour of $P_E(\hat{u}, \hat{w})$ shown in Figures 1c, 2c and 3c represents a square shape which is slightly deviated from the experimental results because of observing a discontinuity point at each of the four quadrants which is minor disadvantage of using GC series exponential distribution to calculate the joint PDFs of velocity fluctuation. The results conclude that joint PDFs distributions for velocity fluctuations based on GC series Hermite polynomial is more superior method as compared to GC series exponential distribution.

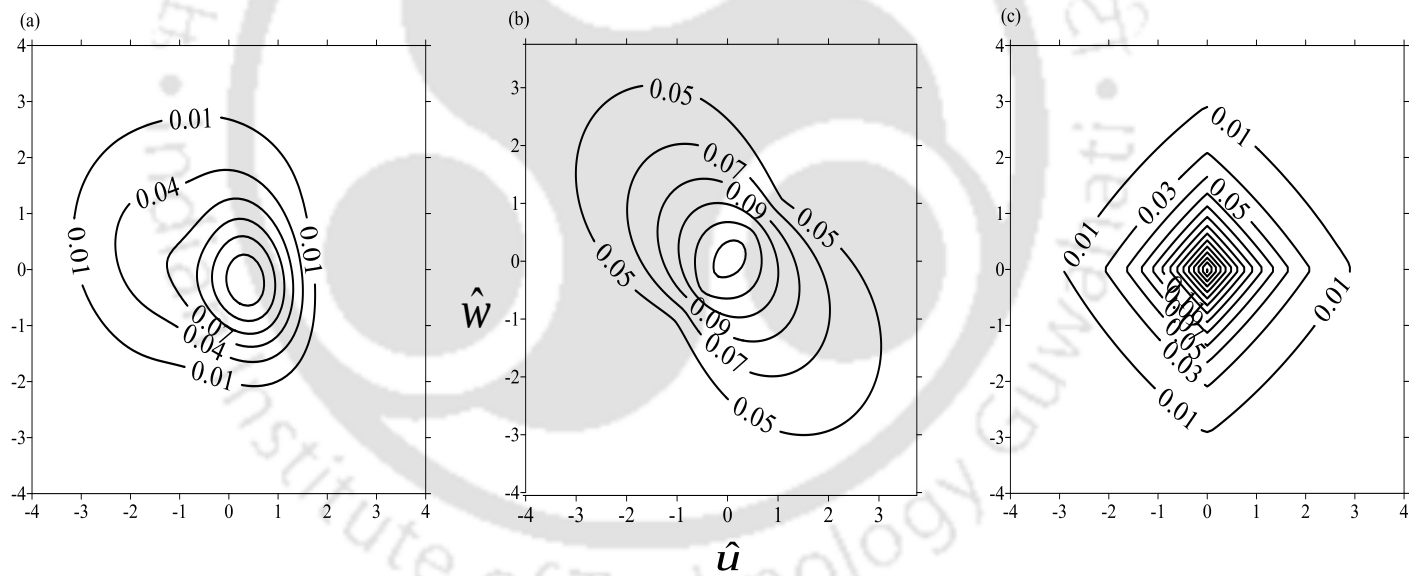


Figure 1: Comparing contour plots of joint probability density function for no seepage case using
(a) GC series Hermite polynomial, (b) Experimentally calculate, (c) GC series exponential
distribution

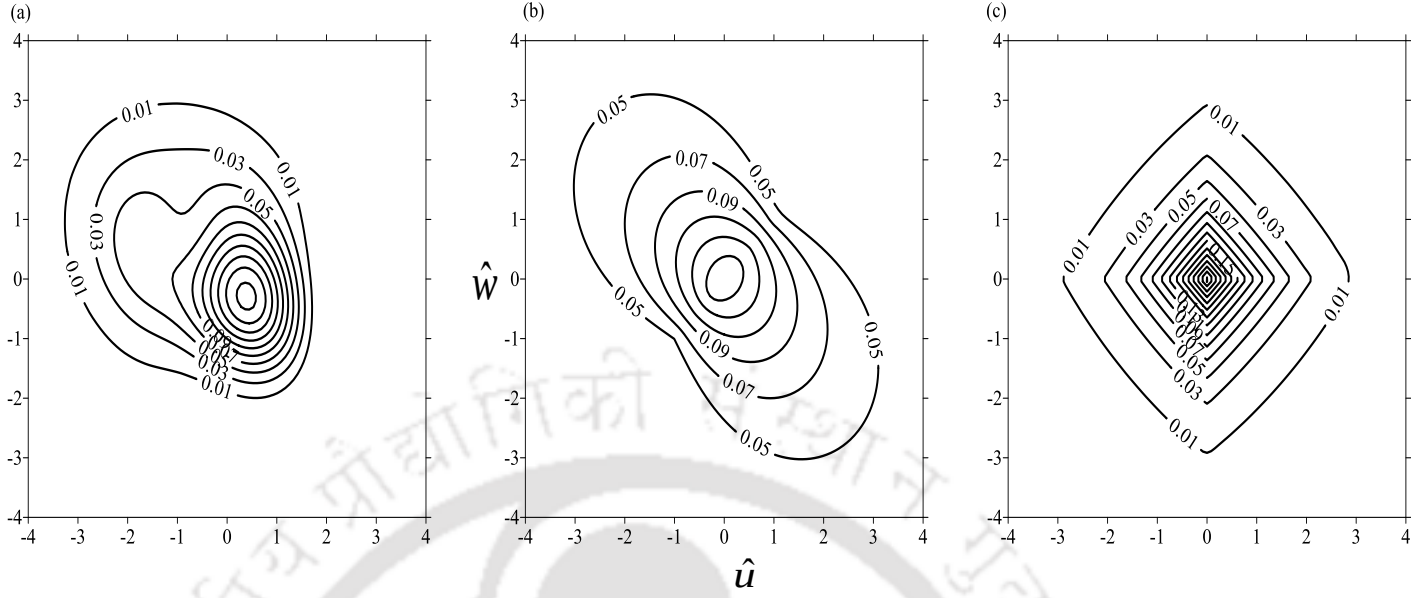


Figure 2: Comparing contour plots of joint probability density function for 10% seepage case using (a) GC series Hermite polynomial, (b) Experimentally calculate, (c) GC series exponential distribution

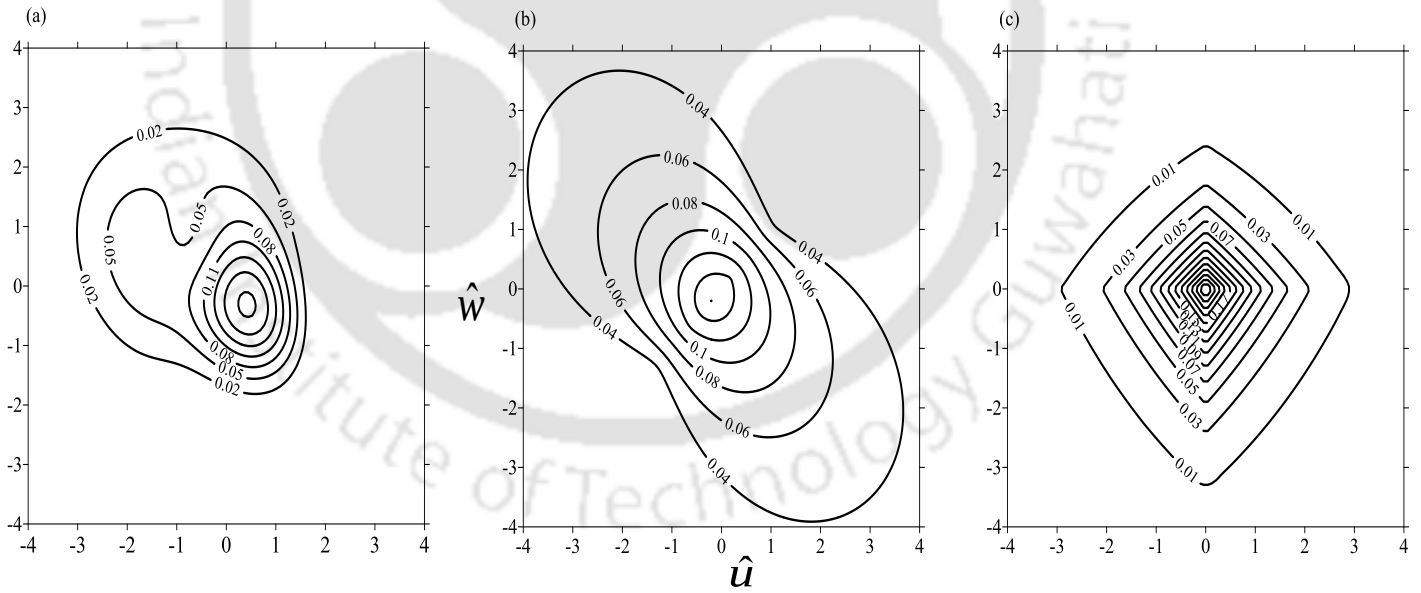


Figure 3: Comparing contour plots of joint probability density function for 15% seepage case using (a) GC series Hermite polynomial, (b) Experimentally calculate, (c) GC series exponential distribution

Table 1: Coefficient α_{jk} for the computation of $P_E(\hat{u}, \hat{w})$

Flow	α_{10}	α_{20}	α_{30}	α_{01}	α_{02}	α_{03}	α_{11}	α_{12}	α_{21}
NS	-0.00033	-0.5	-0.037	-0.0014	-0.5	0.0164	-0.194	-0.0368	0.0131
10%S	-0.00065	-0.5	-0.084	-0.0021	-0.5	0.0299	-0.291	-0.0878	0.0643
15%S	-0.000186	-0.5	-0.054	-0.2371	-0.5	0.436	-0.256	0.0569	0.1740

3.2. PDF of Reynolds Stress

The theoretical curve for joint PDFs of Reynolds shear stress is estimated by approximating the relative frequency of arbitrary variables $\hat{u} \hat{w}$ which was estimated through data sets at a vertical distance z from the bed surface. The required coefficient is calculated with the equations (4) and (9) to yield the theoretical expression for $P_E(\hat{\tau})$ and $P_H(\tau_n)$ given by equations (5) and (10). Figure 4 depicts computed and observed $P_E(\hat{\tau})$ and $P_H(\tau_n)$ distributions at a purpose depth ($z=0.4h$) for both no seepage and seepage cases. The values of coefficient are tabulated in Table 1. It is clear from the Figure 4 that experimentally calculated PDFs followed the theoretical curves which suggests that PDFs of Reynolds shear stress is characterized by GC series exponential distribution and GC series Hermite polynomial for both no seepage and seepage flows. The distribution profiles of probability density function are almost identical for both the GC series but maximum value of PDFs reduces with the GC series Hermite polynomial. It may be focused that $\hat{\tau} = 0$ and $\tau_n=0$ for both GC series have singularity at the origin for no seepage and seepage flows. There are various methods to check the similarity between PDFs of random variables. In the present study, we have used the Jensen-Shannon divergence (JSD) method to measure the similarity between theoretical and experimental PDFs (Lin, 1991). As, $JSD \rightarrow 0$, divergence in between observed and the computed value is small. It is clear from Figure 4 that the value of JSD lies between (0.0011-0.0018) for $P_E(\hat{\tau})$ while the value of JSD lies between (0.00076-0.00095) for $P_H(\tau_n)$ which suggest that degree of divergence is reduced in the case of PDFs based on GC series Hermite polynomial. Thus, the PDFs of Reynolds shear stress based on GC series Hermite polynomial predicts well with the experimental data as compared to PDFs based on GC series exponential distribution.

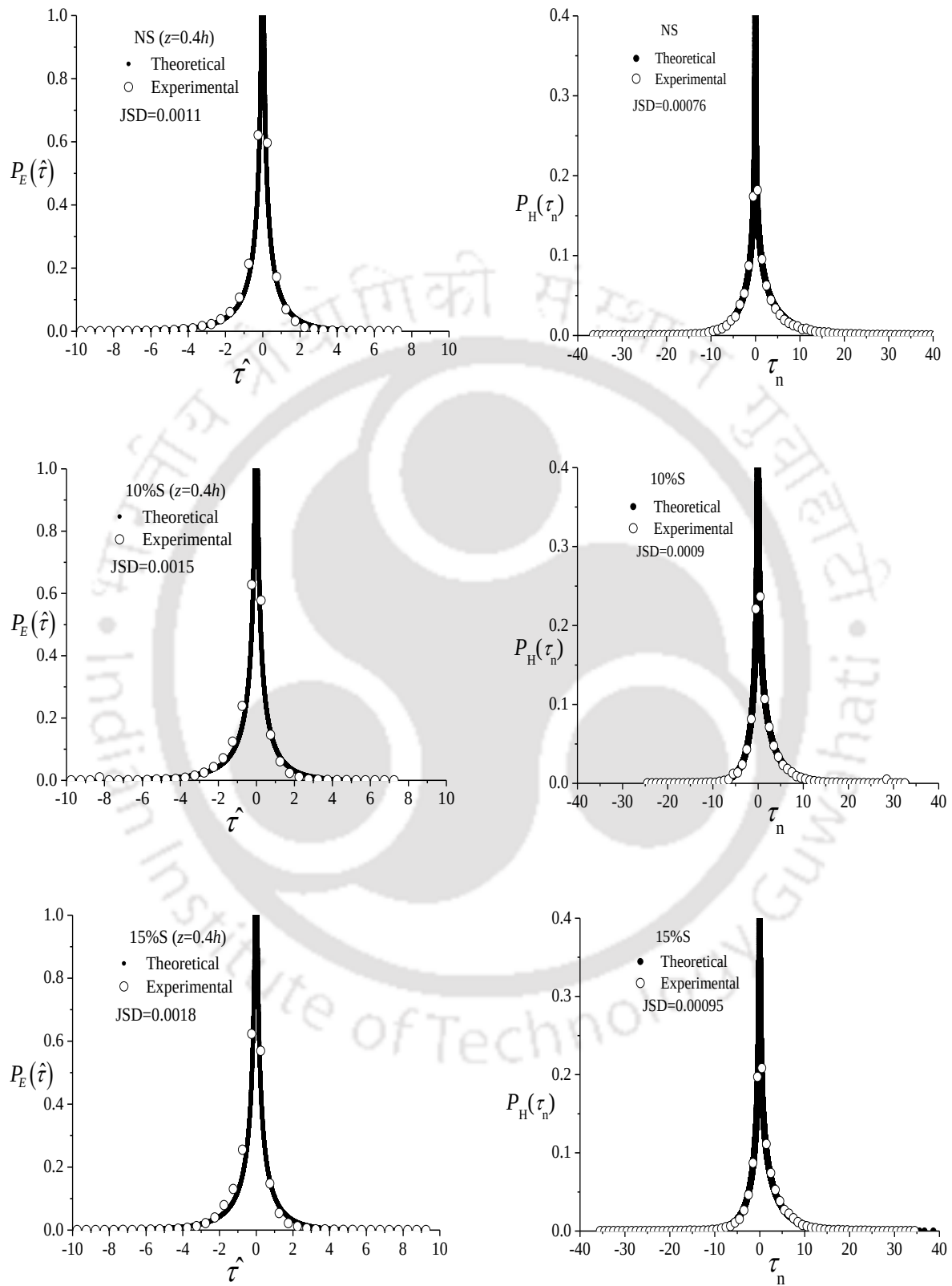


Figure 4: Plot of $P_E(\hat{\tau})$ and $P_H(\tau_n)$ against $\hat{\tau}$ and τ_n respectively for NS, 10%S and 15%S

4. Conclusions

The experimental study of probability distribution function of turbulent parameters is compared with theoretical PDFs based on GC series exponential distribution and Hermite polynomial over non-uniform bed material with and without seepage. The PDF distributions of Reynolds shear stress and joint PDFs of velocity fluctuations are calculated theoretically by GC series exponential distribution and Hermite polynomial considering the moment up to third order. The result shows that as the PDFs calculated from GC series based Hermite polynomial represents circular shape and exponential distribution shift the pattern to the square shape, whereas experimentally calculates joint PDFs show the extreme alteration of shape to elliptic inhabiting the second and fourth quadrants implying the downward momentum flux. The contour orientation has been altered cyclically from quadrant to quadrant based on the bed shear stress. The computed and experimental joint PDF distributions of velocity fluctuations for both no seepage and seepage case are in good agreement except that points of discontinuity is observed at each quadrant in the case of GC series exponential distribution which is disadvantage point of GC series exponential distribution in the calculation of joint PDFs of velocity fluctuation. The computed and experimental PDF distributions of Reynolds shear stress for both no seepage and seepage case are in good agreement in case of GC series Hermite polynomial while fairly agreement is observed in case of GC series exponential distribution because of higher value of JSD which represents the higher degree of divergence. Overall, the conclusion is that PDFs based on GC series Hermite polynomial is superior method over PDFs based on GC series exponential distribution. The presented work is theoretically applicable to probabilities of mobile bed sediments in seepage affected alluvial channels typically influenced by the turbulent parameters. Further, accurate predictions of PDFs of various turbulent parameters are a crucial element of many stochastic bed load models.

Appendix C

Structure Function of Flow in Seepage Affected Alluvial Channel

1. Introduction

Aggradation and degradation of sediment owing to flow turbulence is important to fluvial hydraulics. In recent years, sequences of research have been carried out to understanding the phenomena of fluvial hydraulics in alluvial rivers. The presentation of fluvial hydraulics is generally placed in the form of flow turbulence and bed material transport as the governing factors. Many researchers (**Qian, 1998; Davidson *et al.*, 2006; Huang *et al.*, 2010; Meneveau and Marusic, 2013; De Silva *et al.*, 2015**) studied the structure function of turbulence in different fields in order to understand the scaling behavior within the flow region of turbulent boundary layers. **Qian (1998)** investigated the scaling exponents of the second order structure function of the turbulence and used the Kolmogorov equation in order to compare the scaling of second order against third order structure function according to extended self-similarity method (ESS). It is found that experimental and numerical results of ESS scaling exponent favors the Kolmogorov $2/3$ law rather than irregular scaling. **Davidson *et al.* (2006)** experimentally investigated structure function in logarithmic region of turbulent boundary layer and observed that the logarithmic structure function law is also seen in the experimental data as similar as k^{-1} law. The difference in between logarithmic law and k^{-1} law arises mainly from the finite value of experimental Reynolds number. **Huang *et al.* (2010)** examined the second order structure function in flow turbulence and pointed out that structure function is highly affected by the presence of large energetic scales. They claim that the structure function is not a reliable process to extract the scaling exponents when the datasets hold large energetic scales. **Meneveau and Marusic (2013)** analyzed highly turbulence data in the logarithmic region in order to observe statistical moments of stream wise velocity fluctuations and generalized the logarithmic law for

high order moments in turbulent boundary layer. **De Silva et al. (2015)** used structure functions of the stream wise velocity fluctuation in order to examine the statistical characteristics of flow turbulence in the logarithmic region. They conclude that higher even order structure functions follows the nature of logarithmic law at sufficiently high Reynolds number.

Although there had been earlier research on structure function of flow, the flow structure function in the occurrence of sediment transport subjected to downward seepage in non-uniform sand bed channel are still unexplored. Therefore, the main aim of the present work is to investigate the structure function of flow over non uniform sand bed channel. Also, we will examine the influenced of downward seepage to the structure function of flow turbulence.

2. Results and Discussions

Here, results have been presented for no seepage discharge ($0.0488\text{m}^3/\text{s}$) at a bed slope 0.001 to avoid duplicity and redundancy in the findings. The important recognized feature of flow turbulence is the logarithmic law (**Prandtl, 1925; von Karman, 1930**) for the time averaged stream wise velocity as a function of distance to the wall (z) in the inertial region, for which there is substantial experiential confirmation (**Smits et al., 2011; Jimenez, 2012**).

$$\frac{\langle u \rangle}{u_*} = \langle u^+ \rangle = k^{-1} \ln \left(\frac{zu_*}{\nu} \right) + B \quad (1)$$

The shear velocity $u_* [=(\tau_0/\rho)^{0.5}]$ where τ_0 =boundary shear stress that are used to normalize the velocity has been evaluated by the linear projection of the RSS profile on the channel bed which is $\tau_0 = (\tau_{uw})_{z=0}$ as given by **Nezu (1977)** and ρ is fluid density, ν is the kinematic viscosity, k is the von Karman constant, and B is another constant. Definitely, there has been increasing proof in current years for a universal law of the form (**Marusic and Kunkel, 2003; Hultmark et al., 2012; Marusic et al., 2013**)

$$\langle (u^+)^2 \rangle = B_1 - A_1 \ln \left(\frac{z}{\delta} \right) \quad (2)$$

Where, $u'^+ = (u - \langle u \rangle) / u_*$ is the normalized stream wise velocity fluctuation and δ is the boundary-layer thickness. **Meneveau and Marusic (2013)** suggest that the p^{th} root of moments of velocity fluctuations follows a universal logarithmic law for high-order moments.

$$\left\langle (u'^+)^{2p} \right\rangle^{1/p} = B_p - A_p \ln \left(\frac{z}{\delta} \right) = D_p (\text{Re}_*) - A_p \ln z^+ \quad (3)$$

Where, $z^+ = \frac{zu_*}{\nu}$ is the distance to the wall and if Equation (3) is applied in viscous units, there is an expectation of logarithmic behavior with the same slope $-A_p$ but with a Reynolds-number-dependent Offset $D_p = B_p + A_p \ln \text{Re}_*$ where $\text{Re}_* = \frac{u_* \delta}{\nu}$. According to Gaussian statistics, A_p can be theoretically expressed as $A_p = A_1 [(2p-1)!!]^{1/p}$.

Figure 1 produces results for the stream wise velocity variance as a function of flow depth for no seepage and seepage flow at higher Reynolds number. Initially, the profiles of stream wise velocity variance is slightly scattered in general and increased with seepage which signifies the greater level of turbulence in the boundary layer for seepage flow. At higher Reynolds number, the distribution of stream wise velocity variance undergoes a damping due to a decreasing level of turbulence fluctuation within the wall shear layer. The logarithmic scaling of the variance is used to determine the turbulent intensity (indicates the level of velocity fluctuation) of flow turbulence within the boundary layer at higher Reynolds numbers. It is observed that a possible significance of the log law for mean and variance is that turbulence intensity inclines to zero at asymptotically large distance to the bed for both no seepage and seepage flow. The usual logarithmic region can be determined, and trend data sets for $z^+ > 200$ satisfied the logarithmic law (**Hultmark et al., 2012; Marusic et al., 2013**) $\left\langle (u'^+)^2 \right\rangle = B_1 - A_1 \ln(z/\delta)$ for each cases. For

the no seepage and seepage runs, the range $z^+ > 200$ is taken to execute the performance. For the present study, we have chosen to use the elementary and shorter range with a fixed $z^+ > 200$. The resulting fits would be changed in the present range of no seepage and seepage discharge. For no seepage case, the fitted constants are $B_1=8.47$ and $A_1=0.0034$. For seepage runs of 10% and 15%, the fitted values are $(A_1, B_1) = (0.0037, 8.44)$, and $(0.004, 8.4)$ respectively. We thus confirm that A_1 and B_1 appear to be dependent on the flow condition. The downward seepage influenced the

bed particles start to move rapidly and value of A and B drastically increased and decreased respectively with seepage.

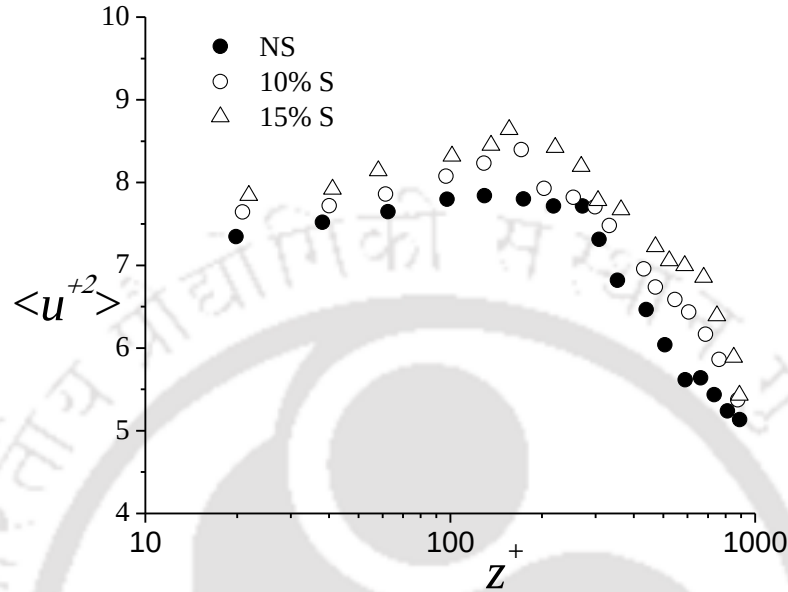


Figure 1: Variance of stream wise velocity in turbulent boundary layers

In this section, high order moments of the velocity fluctuations are evaluated with respect to the flow depth. Before analysis of high order moments of the velocity fluctuations, we have to carefully verify the convergence of high-order moments. For that reason, we have calculated the pre-multiplied velocity fluctuation probability density function (PDF) in order to evaluate whether the moments can be treated statistically converged. Because of inhomogeneity and anisotropic nature of turbulent flows, it is difficult to model turbulent flows in near the core of the logarithmic region. But the effect of seepage is prevalent in the logarithmic region as mentioned in the literature, a measurement location near the core of the logarithmic region ($z=0.07h$) is selected for the study of pre multiplied PDFs. Figure 2 display pre multiplied PDFs for $2p= 2, 4$ and 6 . Close examination of Figure 2 indicate that there is satisfactory conclusion of the pre multiplied PDF in the logic that the area under the curve surrounded by the moment, is well secured by the number of data available, at least up to the highest order $2p=6$ considered here. Also, we have observed that pre-multiplied velocity fluctuation PDFs increases with seepage for $2p= 2, 4$ and 6 which enough indicate that bed material transport is increased with seepage flow.

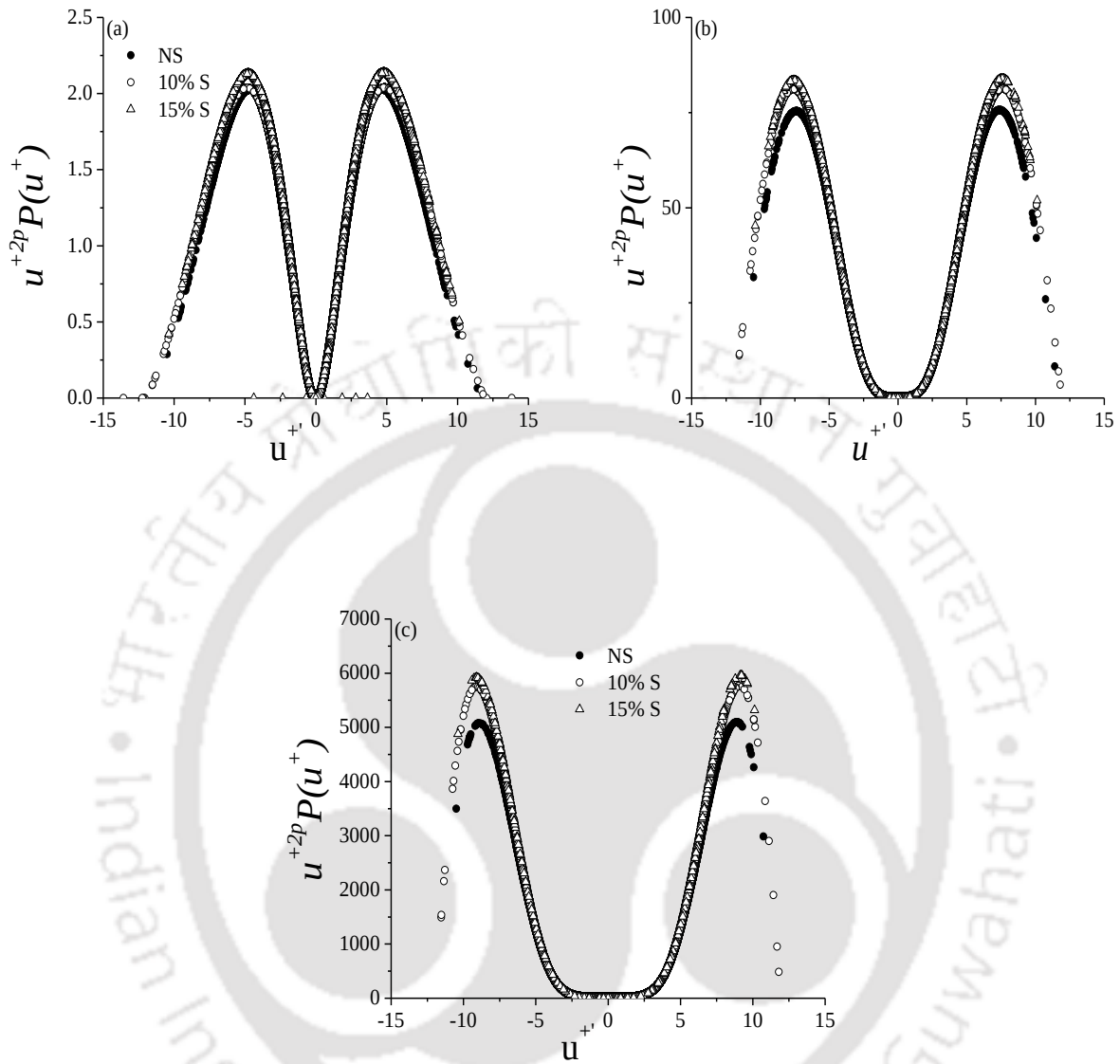


Figure 2: Premultiplied PDF of normalized velocity fluctuations $u^{+2p} P(u^+)$ at a height of $z/h=0.07$. Different moments are represented as (a) $2p=2$, (b) $2p=4$ and (c) $2p=6$.

Higher-order moments for $2p= 4, 6, 8$ and 10 are calculated with respect to the distance from the bed for no seepage and seepage data sets. Figure 3 display the result of high order moments for $2p=6$. Even for sixth order moments, the obtained results are satisfied with a universal logarithmic law in the turbulent boundary layer, in the same boundary layer in which the variance shown logarithmic law. Figure 4 shows the results for the no seepage and seepage datasets considered, for all the even-order moments up to $2p=10$. In the similar manner, the results for all the moments considered here, are reliable with a universal logarithmic law within

the boundary layer. The profiles of higher order moments are similar for both no seepage and seepage runs but the magnitude is higher for seepage runs. The result suggests that strong exchange of momentum occurs from the flow to the bed particle which is enough for the bed particles to move rapidly with seepage flow.

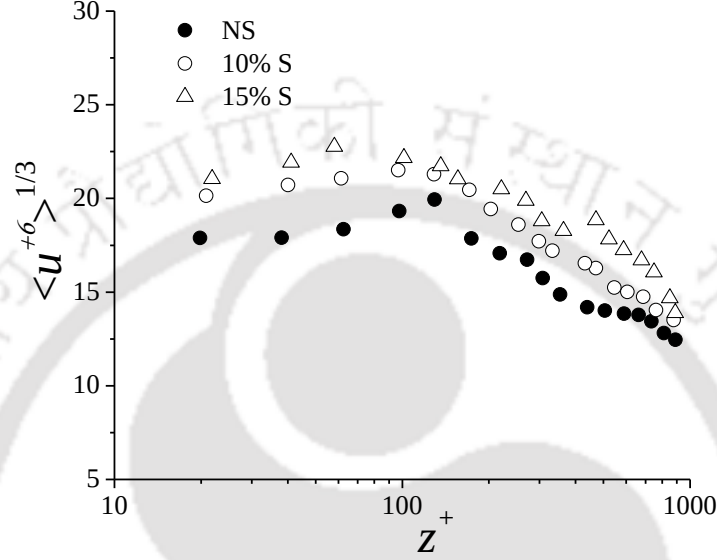


Figure 3: Moments of order $2p=6$ of stream wise velocity as a function of wall-normal distance, for no seepage and seepage runs.

In the next section, we investigate whether for a given order, there is a continuing dependence on seepage discharge even at high seepage percentage or whether the slope A_p asymptotes to a definite value that becomes independent of seepage discharge at large seepage percentage. For that reason, we execute performance of Equation (3) to the number of data available in the core of logarithmic regions mentioned above, mainly $z^+ > 200$. Thus we determine the slopes A_p for both no seepage and seepage runs for the experimental data. Figure 5 shown the results for slopes A_p represented by symbols. The dashed line displays the predictive values of slopes A_p based on Gaussian statistics $A_p = A_1 [(2p-1)!!]^{1/p}$ and shows almost linear owing to the asymptotic behavior of the double factorial function ($n!! \sim n^{(n+1)/2} e^{-n/2}$ and so $(n!!)^{2/n} \sim n$). It shows that A_p slightly deviate from Gaussian values or we can say that it follows sub-Gaussian statistics for no seepage flow while A_p inclines to values that rise above the Gaussian values for

seepage runs. It thus concludes that the sub-Gaussian scaling of moments does not become more Gaussian at seepage flow.

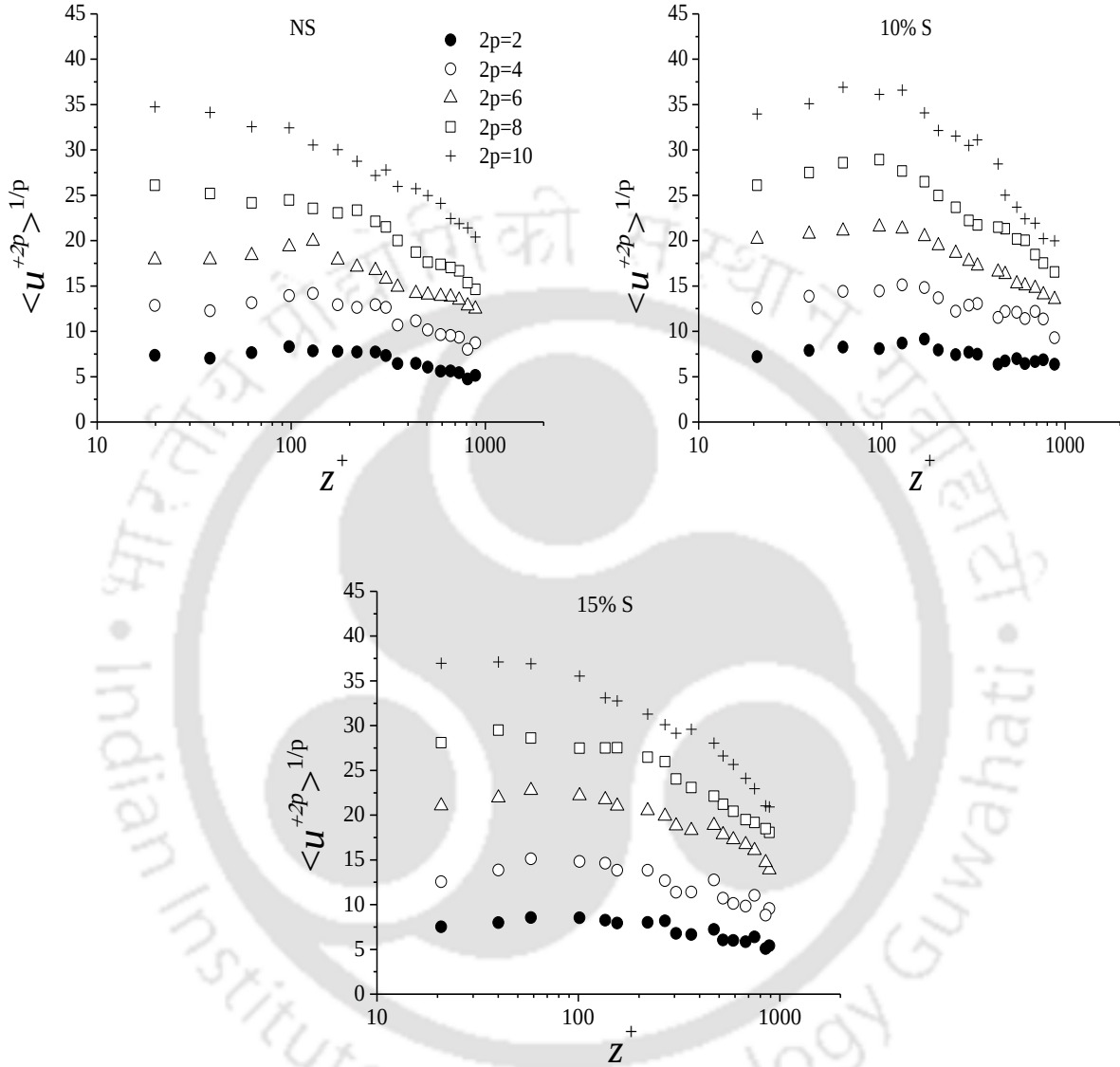


Figure 4: Moments of different orders of stream wise velocity fluctuation as a function of wall-normal distance.

In order to further investigate the seepage discharge assurance of sub-Gaussian statistics, we have plotted the flatness $F_4(z^+)$ in outer units as shown in Figure 6. The obtained results for flatness in the present study are in good agreement with previously stated flatness results in zero-pressure-gradient boundary layers, as reviewed by previous studies (**Fernholz and Finley, 1996; Meneveau and Marusic, 2013**).

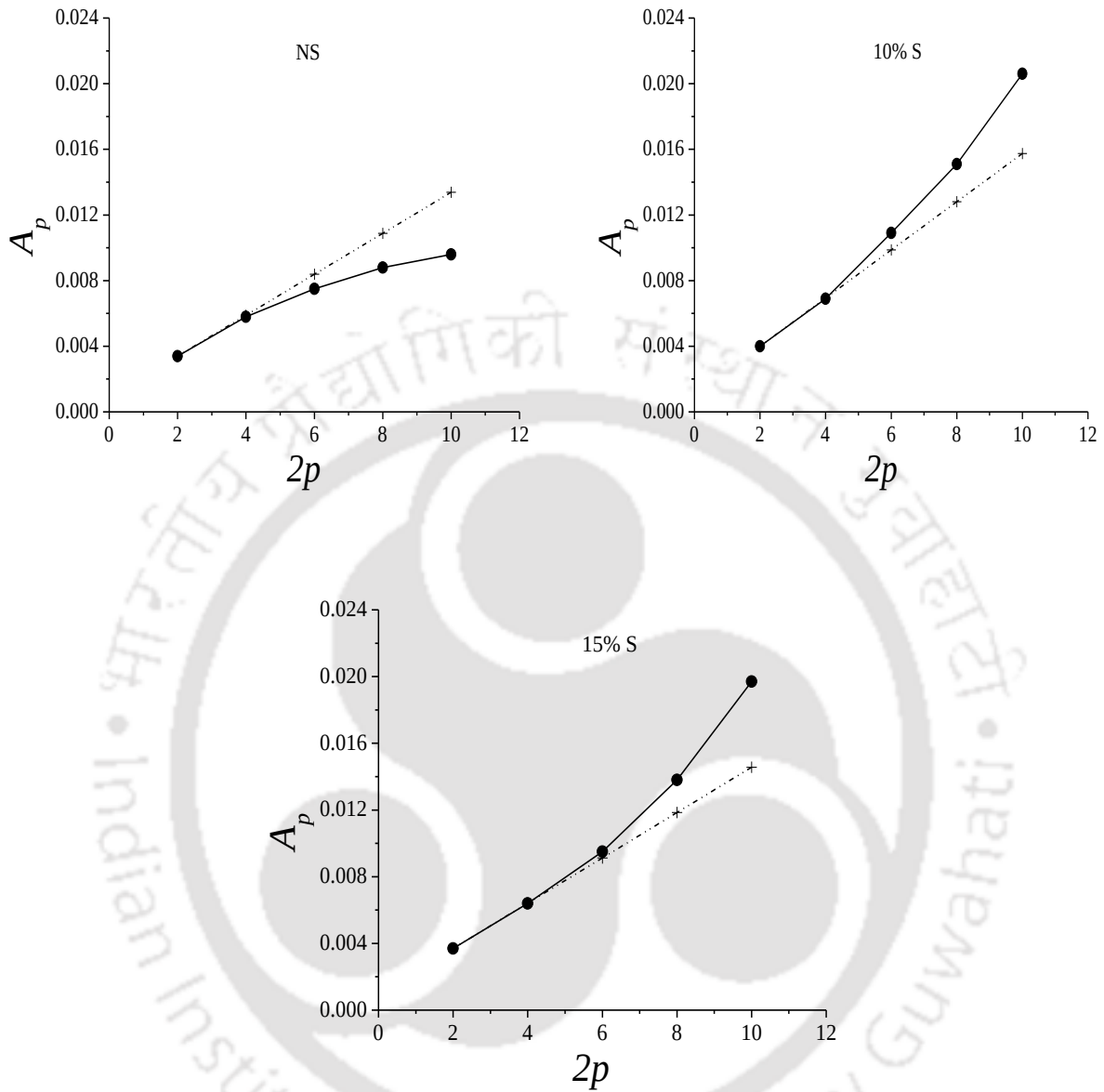


Figure 5: Coefficients A_p in logarithmic law for moments as a function of moment order $2p$

Flatness in the inertial zone is observed to be below the Gaussian value of $F_4=3$, and thus does not seem to be a noticeable seepage discharge dependence in the form of sub-Gaussianity. It is observed that flatness becomes minimum at $z^+ \approx 150$ after which the value is increased in both inner and outer layer of turbulent flow. It suggests that turbulence is highly intermittent in both layers which further suggest that sweep and ejection provides strong intermittent contribution to the turbulence production. Noticeably, $F_4 > 3$ in flow layer refers to a distribution with a peaky signal characteristic of intermittent turbulent events while $F_4 < 3$ refers to a flat characteristic.

However the position of occurrence of minimum flatness is increased with the percentage of seepage which is $z^+ \approx 175$ for 10%S and $z^+ \approx 185$ for 15%S. Further, flatness is increased in the near bed surface with seepage which represents the higher level of turbulence intermittency is observed with seepage as compared to that with no seepage.

3. Conclusions

The results shown in this work climax the universality of the structure function as a means to understand the scaling phenomena inside the core of logarithmic region of flow turbulence. The implication of both spatial and temporal datasets permitted us to calculate structure functions in alluvial channel with and without downward seepage. The experimental datasets tends to a series of probably non-universal coefficients A_p , at which A_1 is the Townsend–Perry constant. Higher-order moments are reliable with sub-Gaussian data for no seepage flow and super-Gaussian data for seepage flow, which suggest that A_p cannot be insignificantly connected to A_1 and a valid theoretical expression for A_p is yet to be develop. The uniformity in scaling law noticed in the structure functions from the present work complements the results of Meneveau and Marusic (2013) on the higher order moments of the stream wise velocity fluctuations.

Appendix D

Modified Singular Spectrum Analysis for Despiking Acoustic Doppler Velocimeter (ADV) Data

1. Overview

Acoustic Doppler Velocimeter (ADV) has been widely used in experimental flume and in fields as well. The main advantage of using ADV is that it can measure three dimensional instantaneous velocity measurements, turbulent characteristics and power spectral density (PSD). Major concern in ADV measurements is the presence of contaminants, which generally occur as spikes in the velocity data series. These spikes occur due to the presence of high turbulence intensities or due to the presence of earlier contaminants that returned from the boundary wall (**Voulgaris and Trowbridge, 1998; Nikora and Goring, 1998**). The contaminants are generally initiated by presence of free shear layer, boundary interferences, signal aliasing and Doppler noising (**Yafei, 2015; Islam and Zhu, 2013**). Because of the presence of these spikes, the turbulent characteristics and power spectral of such time series may not reflect the actuality. Therefore, turbulence and power spectral density of data sets should be analysed with distinct cares otherwise the trends can affect from significant inaccuracy (**Wahl, 2000; Lane et al., 1998; Cea et al., 2007; García et al., 2005**). The despiking ADV data can be used to predict accurately the turbulent parameters in turbulent flows (**Lohrmann et al., 1994; Hurther and Lemmin, 2008; McLelland and Nicholas, 2000**). Various techniques have been developed to eliminate spikes from the velocity temporal data sets. **Nikora and Goring (1998)** have used various thresholding methods to discriminate spurious data from the acceptable ones. **Goring and Nikora (2002)** proposed an acceleration threshold method (**Donoho and Johnstone, 1994**) and phase-space threshold method to despik ADV data. The phase-space threshold (PST) method was later modified by **Wahl (2003)**, and the modified version was included in postprocessing software called WinADV (**Wahl 2012**). **Mori et al. (2007)** also used PST

technique to eliminate the spikes from velocity time series data in bubbly flows. **Cea *et al.* (2007)** considered PST approach to find intermittent spiked time series data and remodelled the ADV data to evaluate the turbulent characteristics as well as velocity spectra. **Parsheh *et al.* (2010)** applied a preconditioning to the PST method (**Goring and Nikora, 2002**) which uses a robust data to calculate the dimensions of cluster. **Razaz and Kawanisi (2011)** developed a new technique for detecting and replacing spikes by converting data into wavelet packets and thresholding the wavelet source by a constant threshold. Recently, **Islam and Zhu (2013)** applied above mentioned methods to investigate the experimental data from a wall jet test which contained a lot of contaminants in the time series and they found that the existing methods are ineffective at many places with 40% of contaminated data. In order to improve despiking in such cases, **Islam and Zhu (2013)** used the Gaussian kernel (**Duong and Hazelton, 2003**) to develop a kernel density based density function to remove the spikes from the cluster. Regardless of considerable progress, the estimation of turbulent parameter such as mean velocity and velocity fluctuation for highly and severely contaminated time series remain a subjective problem. The more interesting difficulty is the exact calculation of the velocity power spectra, which gives crucial knowledge about the turbulent coherent structure.

The literatures suggest that the previous despiking approaches were not completely effective in the fields where the time series was significantly contaminated by spikes. The inspiration of the present work is to propose a new technique applicable for fields where the time series was significantly contaminated by spikes. Thus, the present work proposes a singular spectrum analysis (SSA) technique with modified grouping criteria to remove the spikes from contaminated ADV data sets and compare the results from the existing spike removal techniques. Power spectral density (PSD) is expressively subjected to the occurrence of spikes, which is reflected in the power spectra of the contaminated ADV data series. Here, we will validate the correctness of our restructured technique by detecting a $-5/3$ slope in the region of inertial sub range of the power spectra of the restructured data series. The inertial sub range, where the assumption of isotropy and Kolmogorov $-5/3$ law is applied, gives the valuable information about the turbulent properties of the flow. To validate the proposed method, the focus is to analyse the resulting PSD of the corrected velocity time series. Comparing the PSD of restructured data to that of the contaminated data set agrees the validation of the proposed method for the complete sets of spikes exist in the velocity time series.

2. Mathematical Background of SSA Technique

Subspace based techniques are used to enhance the noise corrupted time series signals. Singular spectrum analysis (SSA) (**Golyandina et al., 2001; Ghil et al., 2002**) is a subspace built method that decomposes the time series data into trend, oscillating and noise components. It is widely used for analysing climatic and biomedical signals. The main difference between the traditional SSA method discussed in **Golyandina et al., 2001, Ghil et al., 2002** and our method is lies in the way of identifying the desired signal (velocity signal) subspace ; in other words the grouping step in SSA. In traditional SSA, the desired signal subspace is estimated by based on the magnitude of the eigenvalues. However, in our technique, the desired signal subspace is estimated based on the local mobility of the eigenvectors. SSA includes four basic steps: embedding, decomposition, grouping and reconstruction.

2.1 Embedding

Consider M sampled signal vector $\mathbf{f} = [f(1), f(2), \dots, f(M)]$, which is a mixture of two signals $\mathbf{f} = \mathbf{a} + \mathbf{b}$. The signal vectors $\mathbf{a}$ and $\mathbf{b}$ represents the desired and the noise signals respectively. The embedding step in SSA maps the given univariate signal vector $\mathbf{f}$ into multivariate data matrix $\mathbf{F}$, which is

$$\mathbf{F} = \begin{pmatrix} f(1) & f(2) \cdots & \cdots f(L) \\ f(2) & f(3) \cdots & \cdots f(L+1) \\ \cdots & \cdots & \cdots \\ \cdots & \cdots & \cdots \\ f(N) & f(N+1) & \cdots f(M) \end{pmatrix} \quad (1)$$

Where, N is the window length and $L = M - N + 1$. The trajectory matrix $\mathbf{F}$ is called hankel matrix as its anti-diagonal elements are equal. The window length is $N > f_s/f$, where f_s and f respectively represent the sampling frequency and maximum frequency of the desired signal (**Teixeira et al., 2006**). As the signal vector $\mathbf{f}$ is mixture of two signals, then the matrix $\mathbf{F}$ can be represented as

$$\mathbf{F} = \mathbf{A} + \mathbf{B} \quad (2)$$

Where, $\mathbf{A}$ and $\mathbf{B}$ are the trajectory matrices corresponding to the desired and noise signals respectively. Here, the task is how to estimate the trajectory matrix $\mathbf{A}$ from $\mathbf{F}$.

2.2 Decomposition

After embedding, the trajectory matrix in Equation (1) is decomposed into N lower rank matrices. In order to perform such decomposition, singular value decomposition (SVD) or eigen decomposition (ED) of $\mathbf{F}$ has to be performed. However, as the computation of SVD of matrix of size $N \times L$ involves infeasible computations, the decomposition of $\mathbf{F}$ is performed using ED. Let $\lambda_1, \lambda_2, \dots, \lambda_N$ and $\mathbf{u}_1, \mathbf{u}_2, \dots, \mathbf{u}_N$ are the eigenvalues and eigenvectors of a covariance matrix $\mathbf{C} = \mathbf{F}\mathbf{F}^T$, where T is the transpose operator. It is noticed that the eigenvalues are organized in descending order based on their magnitudes and the corresponding eigenvectors. In general, the SVD of trajectory matrix is given by

$$\mathbf{F} = \mathbf{U}\mathbf{\Sigma}\mathbf{V}^T \quad (3)$$

where, $\mathbf{U}$ and $\mathbf{V}$ are the left and right singular matrices of size $N \times N$ and $L \times L$ respectively and $\mathbf{\Sigma}$ is diagonal matrix of size $N \times L$, whose elements are singular values. The columns of $\mathbf{U}$ and $\mathbf{V}$ are the eigenvectors of the covariance matrix $\mathbf{C}_u = \mathbf{F}\mathbf{F}^T$ and $\mathbf{C}_v = \mathbf{F}^T\mathbf{F}$ respectively. Let k^{th} eigenvector of a right singular matrix $\mathbf{V}$ is given by

$$\mathbf{v}_k = \mathbf{F}^T \mathbf{u}_k / \sqrt{\lambda_k} \quad (4)$$

Then, k^{th} trajectory matrix $\mathbf{F}_k$ is given by

$$\mathbf{F}_k = \sqrt{\lambda_k} \mathbf{u}_k \mathbf{v}_k^T \mathbf{F} \quad (5)$$

After substituting the term (6.4) in Equation (6.5) and after simplification $\mathbf{F}_k$ can be written as

$$\mathbf{F}_k = \mathbf{u}_k \mathbf{u}_k^T \mathbf{F} \quad (6)$$

Where, the term $\mathbf{u}_k \mathbf{u}_k^T$ forms a subspace for k^{th} component present in $\mathbf{f}$ signal and $k = 1, 2, \dots, N$. Projecting $\mathbf{F}$ onto the subspace results a trajectory matrix belongs to the desired signal. It is clear from Equation (6) that the trajectory matrix $\mathbf{F}$ is decomposed into N low rank matrices using the eigenvectors of the covariance matrix $\mathbf{C} = \mathbf{F}\mathbf{F}^T$.

2.3 Grouping

The crucial step in SSA is to identify the basis vectors (eigenvectors), which are needed for the faithful reconstruction of trajectory matrix $\mathbf{A}$. In the grouping step of basic SSA, the N decomposed trajectory matrices using Equation (6) are splitting into d groups. Let $I_j = \{l_1, l_2, \dots, l_p\}$ indicates the indices of p largest eigenvalues of j^{th} group. The trajectory matrix for the j^{th} group is given by

$$\mathbf{F}_{I_j} = \sum_{i=1}^p \mathbf{F}_{l_i} \quad (7)$$

The trajectory matrix $\mathbf{F}$ can be obtained by

$$\mathbf{F} = \sum_{j=1}^d \mathbf{F}_{I_j} \quad (8)$$

In traditional SSA, the grouping step is performed based on the magnitude of the eigenvalues. However, such grouping technique may work well when the signal to noise ratio (SNR) is high.

2.4 Reconstruction

Once after grouping the N decomposed trajectory matrices into d groups during reconstruction step of SSA, the resulted trajectory matrix is mapped into univariate signal. For example, $\hat{a}_{i,j}$ be the elements of the estimated trajectory matrix $\hat{\mathbf{A}}$ of the desired signal, and then the univariate signal can be obtained using the following equations

$$\hat{a}(t) = \begin{cases} \frac{1}{t} \sum_{k=1}^t \hat{a}_{k,t-k+1} & \text{for } 1 \leq t < N \\ \frac{1}{N} \sum_{k=1}^N \hat{a}_{k,t-k+1} & \text{for } N \leq t \leq N \\ \frac{1}{M-t+1} \sum_{k=t-L+1}^{M-L+1} \hat{a}_{k,t-k+1} & \text{for } L < t \leq M \end{cases} \quad (9)$$

The Equation (9) performs diagonal averaging to reconstruct the univariate signal. Here diagonal averaging means all the anti-diagonal elements of $\hat{\mathbf{A}}$ are replaced with their average. The critical step in SSA is grouping the desired trajectory matrices, in another word identifying the desired signal subspace. Several grouping techniques have been discussed in (Teixeira *et al.*, 2006;

Ghaderi et al., 2011) to identify the desired signal subspace. In all these techniques, magnitude of the eigenvalues was considered to identify the signal subspace. However, such criteria might work well when the SNR is high, which means that the variance of desired signal is higher than the noise component. In general, as we do not know the contribution level of the noise in the mixed signal, hence such criteria may not be feasible. Recently, the local mobility of eigenvectors is considered in identifying the signal subspace and is applied to remove several artefacts from the biomedical signals (**Maddirala and Shaik, 2016a; Maddirala and Shaik, 2016b**). The local mobility is a signal complexity measure often used in the analysis of biomedical signals (**Nagarian and Splinter, 2005**). The key idea of this technique is that the local mobility of eigenvectors associated to the desired signal is low and is high for the eigenvectors associated to the noise component.

Consider a signal vector $\mathbf{u} = [u(1), u(2), \dots, u(N)]$, then the local mobility of the signal vector is given by (**Nagarian and Splinter, 2005**):

$$m_u = \frac{m_1}{m_0} \quad (10)$$

where, $m_0 = \sqrt{\frac{\sum_{n=1}^N u^2(n)}{N}}$ and $m_1 = \sqrt{\frac{\sum_{n=1}^{N-1} s^2(n)}{N-1}}$. Here, $s(n)$ represents the first order variations of the signal *i.e.* $s(n) = s(n+1) - s(n)$, $n = \{1, 2, \dots, N-1\}$. As the noise components are high frequency signals, the local mobility of eigenvectors associated to these components are high. Thus this criterion shows distinct local mobility of eigenvectors associated to the desired and the noise components.

3. Algorithm Description

For efficient noise cancellation from ADV time series data, the free parameters of the proposed algorithm such as window length N and the threshold T_{th} has to be selected carefully. Here, the window length N is set based on the following criteria *i.e.* $N > f_s/f$, where f_s is the sampling frequency and f is the maximum frequency of the desired signal (**Teixeira et al., 2006**). As the useful information in the ADV time series lies below 1Hz, the window length N is set to 201. Note that the ADV time series is recorded at a sampling frequency $f_s = 200$ Hz. In order to identify the eigenvectors associated to the noise free ADV time series, the threshold T_{th} is set to 0.031.

To set T_{th} , first 1 Hz sinusoidal signal is generated. Later, the local mobility of that signal is computed and used as threshold T_{th} . As the temporal morphology of the eigenvectors derived from the covariance matrix is oscillating in nature, here, sinusoidal signal was chosen to compute the threshold T_{th} . The key steps of proposed SSA technique were discussed below:

- (i) Set the window length N and threshold T_{th} based on the desired signal.
- (ii) Map univariate signal vector $\mathbf{f}$ into multivariate data matrix $\mathbf{F}$ using embedding step.
- (iii) Compute the covariance matrix $\mathbf{C}=\mathbf{F}\mathbf{F}^T$ and perform Eigen decomposition of $\mathbf{C}$.
- (iv) Evaluate the local mobility of each eigenvector and identify the arguments of the basis vectors (eigenvector) associated to the desired signal using threshold T_{th} .
- (v) Compute Equation (6) for each such eigenvector whose local mobility is less than the predefined threshold and add all the resulted trajectory matrices.
- (vi) Perform the diagonal averaging (Equation 9) step on the trajectory matrix obtained in step (v) to reconstruct univariate despiky signal.

4. Data collection and Results

The time series velocity data of vegetated sand bed channel were taken from the previous literature (**Devi and Kumar, 2016**) that is highly contaminated ADV time series data. **Devi and Kumar (2016)** have done the experiments in a rectangular flume of 20 m in length, 1 m in width, and 0.72 m deep. The flow depth ($h=12$ cm) and incipient discharge ($Q_0=0.0326$ m³/s) were applied for experiment with vegetation. More details about data collection and experimental setup can be observed from previous literature (**Devi and Kumar, 2016**). The benefit of using experimentally contaminated data sets is that by comparing filtered and contaminated data sets one can validate the proof of spike removal and signal restructured techniques. The ADV having sampling frequency of 200 Hz was used to get the velocity time data series contaminated with spikes. The ADV sample volume (measurement data) was collected at a position of 4.6 cm from the bed which is within the vegetation height. We have collected one sample with 24000 instantaneous velocities at the centre line of the flume to assess the capability of previous despiking approach and the proposed modify SSA technique. It should be noted that a velocity time series data taken by ADV is generally highly contaminated because of presence of high

shear, high turbulent fluctuation, and wall boundary. Therefore, in the present study, we have compared the performance of all despiking techniques for velocity time series obtained by ADV. The vegetated sand bed containing highly contaminated data are discussed in order to validate the proposed modified SSA technique for despiking ADV.

The uncertainty analysis is carried out to verify the validity of the measurements. An uncertainty may originate from causes such as the lack of accuracy in measurement equipment, random variation in the measures and approximations in data reduction relations **(Coleman and Steele, 2009; Çarpınlioğlu and Özahi, 2013; ANSI/ASME, 1985)**. In the present study, the uncertainty of Vectrino data was determined by collecting 10 pulses samples at the centre of the test section at a location of 10 mm over the beds. The term standard uncertainty of the mean describes the standard deviation of the mean for a set of several repeated pulses of instantaneous velocities. Standard uncertainty is calculated as:

$$S_u = \frac{S_d}{\sqrt{n}} \quad (11)$$

Where, S_d is the standard deviation and n is the number of measurements in the set. The value of S_d , S_u and the percentage uncertainty of time averaged velocity are calculated as 4.31×10^{-3} , 1.36×10^{-3} and 0.33% respectively, which is within $\pm 5\%$ error for the time averaged velocity **(Dey et al., 2012)**.

We have applied proposed technique on the ADV time series data with window length $N=201$ and $T_{th}=0.031$. Figure 1 shows the local mobilities of N eigenvectors. As the local mobility of eigenvector $\mathbf{u}_1$ is less than the specified threshold T_{th} , it is the basis vectors to reconstruct despiked ADV time series signal. The filtered ADV time series signal obtained by existing methods **(Wahl, 2003; Goring and Nikora, 2002; Parsheh et al., 2010; Islam and Zhu, 2013)** and the proposed techniques were shown in Figure 2. The filtered ADV time series data obtained using existing methods is not fully reconstructed, moreover the existing techniques modifying the lower frequency components which is seen from the zoomed portion of Figure 2. The proposed SSA technique preserves the trend in the time series signal and also removed noise components.

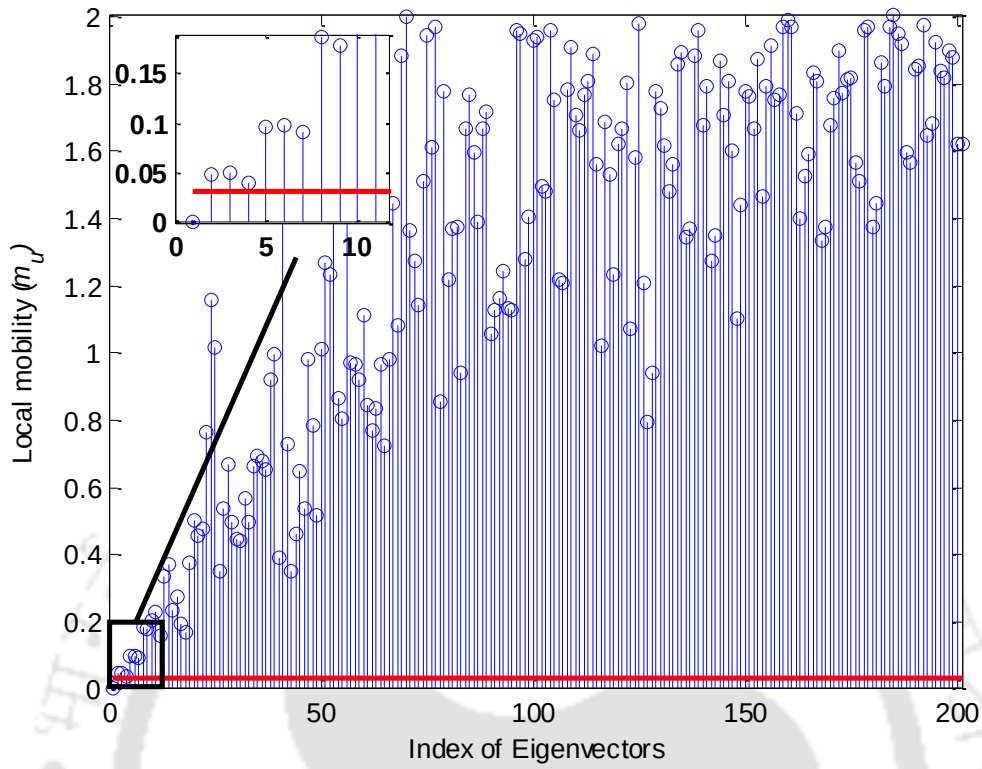


Figure 1: Local mobility of N eigenvectors and threshold $T_{th}=0.031$.

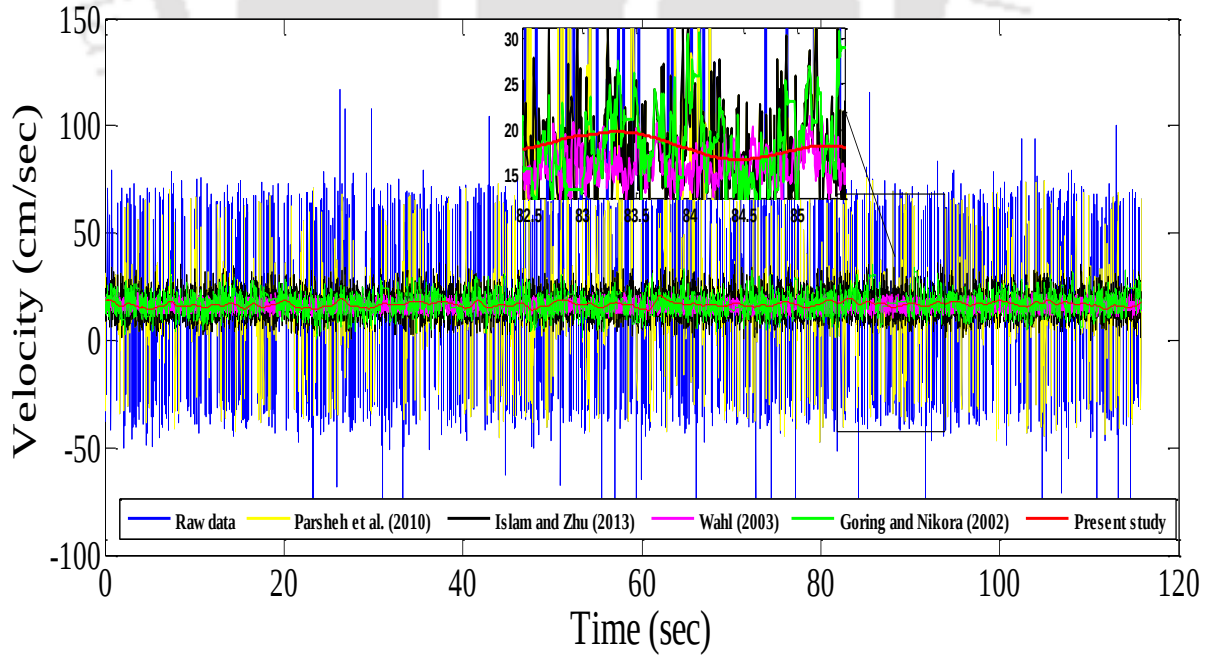


Figure 2: Contaminated and filtered ADV time series data for vegetated sand bed channel

To validate the above conclusions, we compute the power spectral density (PSD) of contaminated and filtered ADV time series data. The PSDs of contaminated and filtered ADV time series data using existing methods and proposed technique were also shown in Figure 3. When comparing the PSD of ADV data using existing technique with the PSD of contaminated data, it was observed that the noise components are present in the existing methods. However, proposed technique attenuates the noise components efficiently. It is clear from Figure 3 that PSDs satisfies Kolmogorov -5/3 law in the inertial sub range.

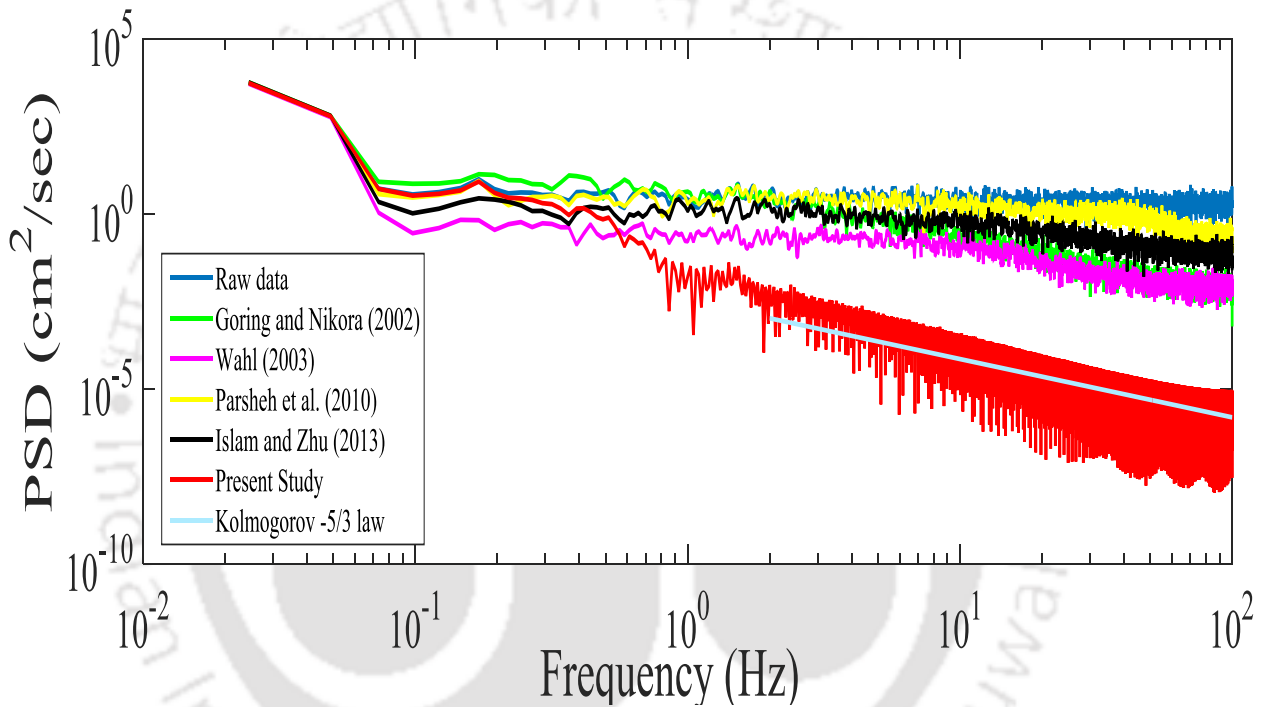


Figure 3: Power Spectral Density (PSD) of u-velocity component of contaminated data and restructure data subjected to vegetated sand bed channel.

In present section, we have applied the SSA method to a signal with no spikes to demonstrate that the method does not destroy the structures within the turbulence (ejections and sweeps) because this is the whole point of analysing turbulence data. Figure 4 shows the joint distributions of u' and w' at the propose flow depth using SSA method. It can be seen that, ejection and sweep are the main contributors to the Reynolds stress. In Figure 4, ejection and sweep events contribute about 110.6% and 118.4% respectively towards the production of Reynolds stress while outward interaction and inward interaction events contribute minimal as 65.8% and 63.2% respectively. We have also analysed bursting events using **Islam and Zhu**

(2013) algorithm at the same flow depth and we have observed that the contribution from ejection and sweep events is around 80.7% and 90.6% respectively while the contribution from outward and inward interaction is around 36.9% and 34.4% respectively. Thus in both SSA and **Islam and Zhu (2013)** method, sweep and ejection are the main events to the production of Reynolds stress and it is clear that the present SSA method does not affect the structure of turbulence.

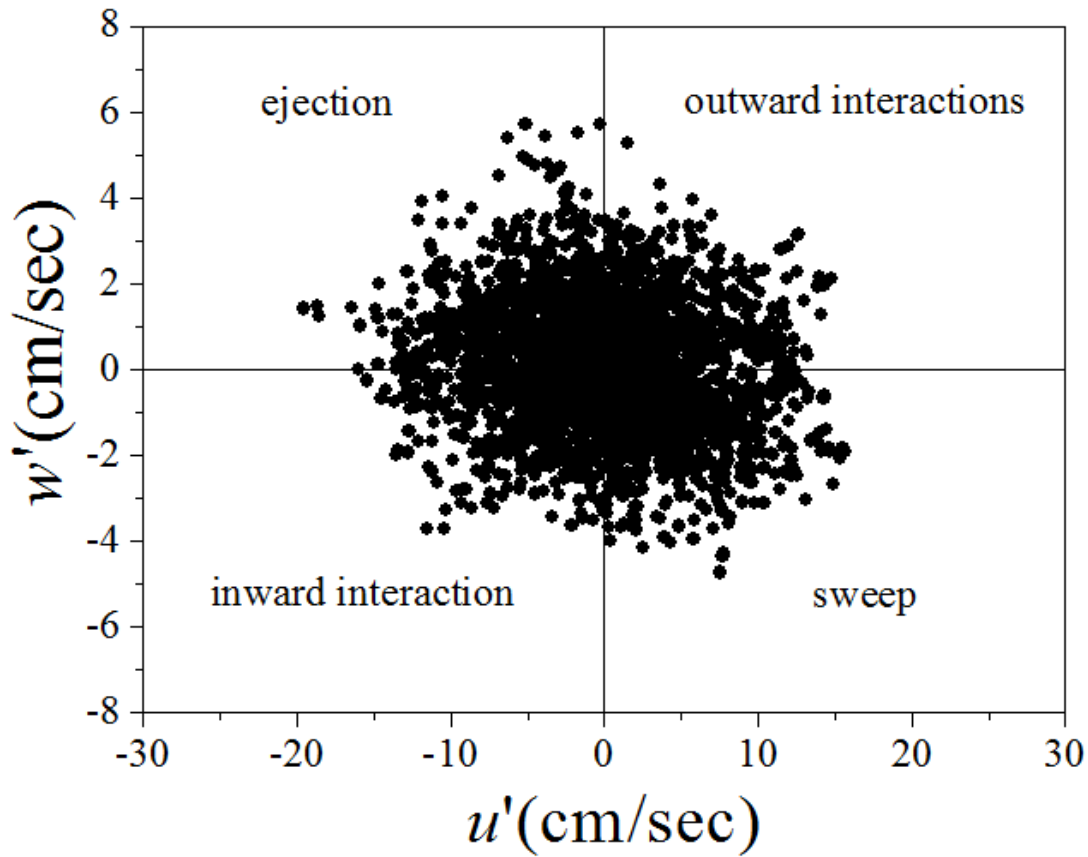
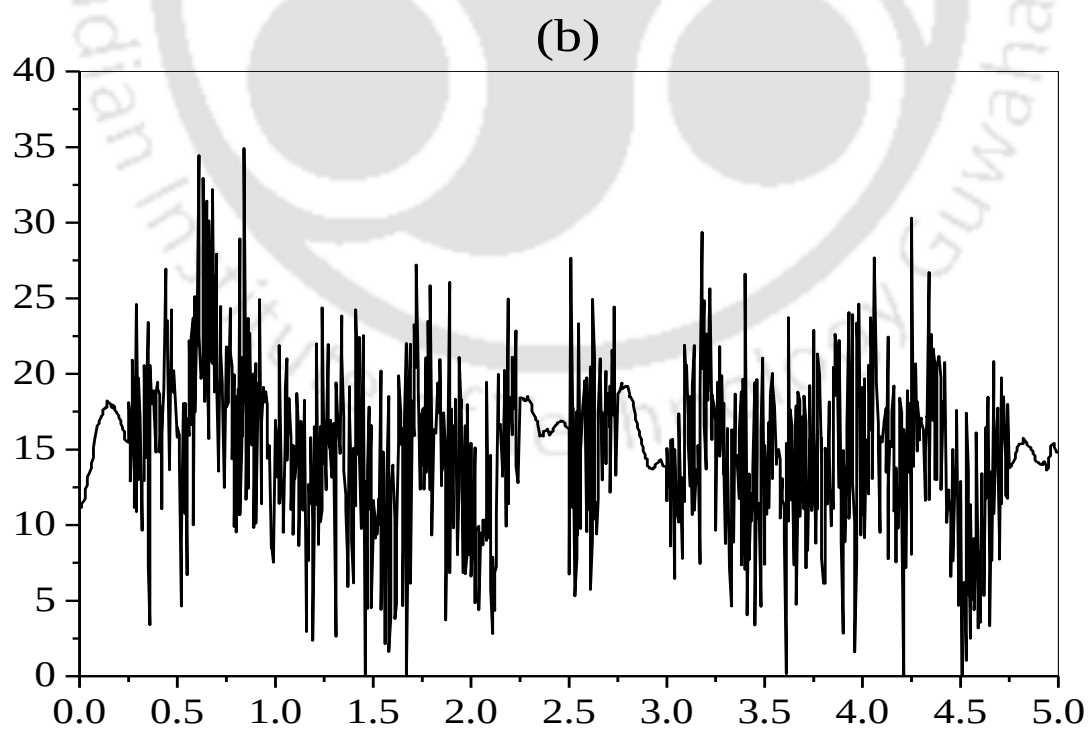
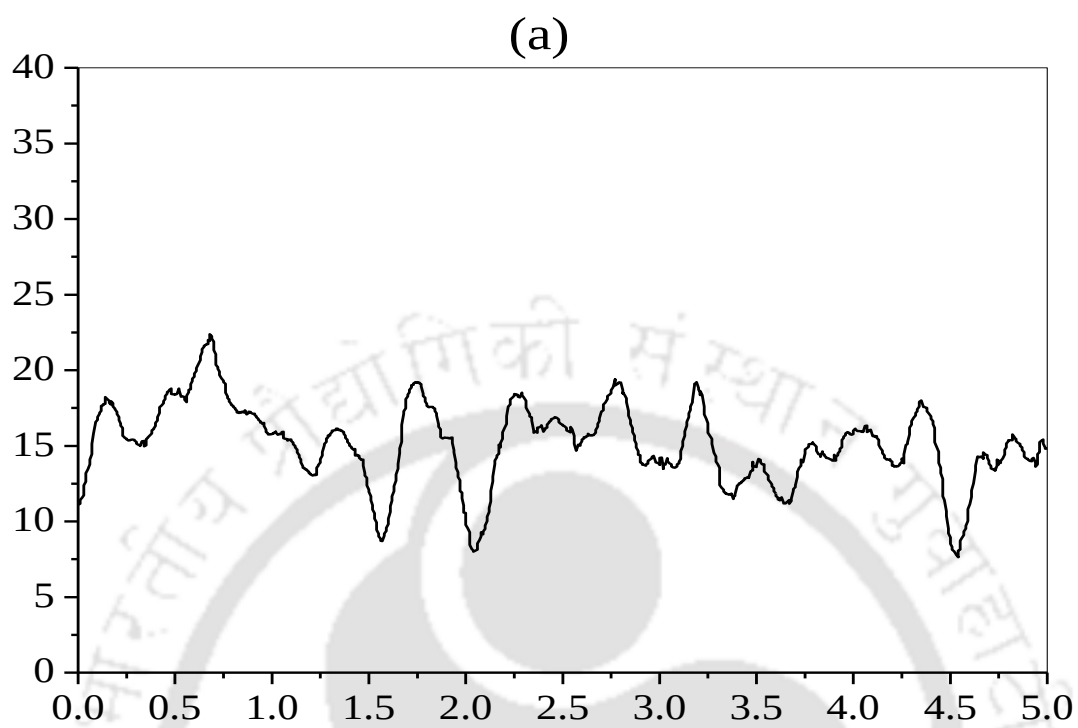


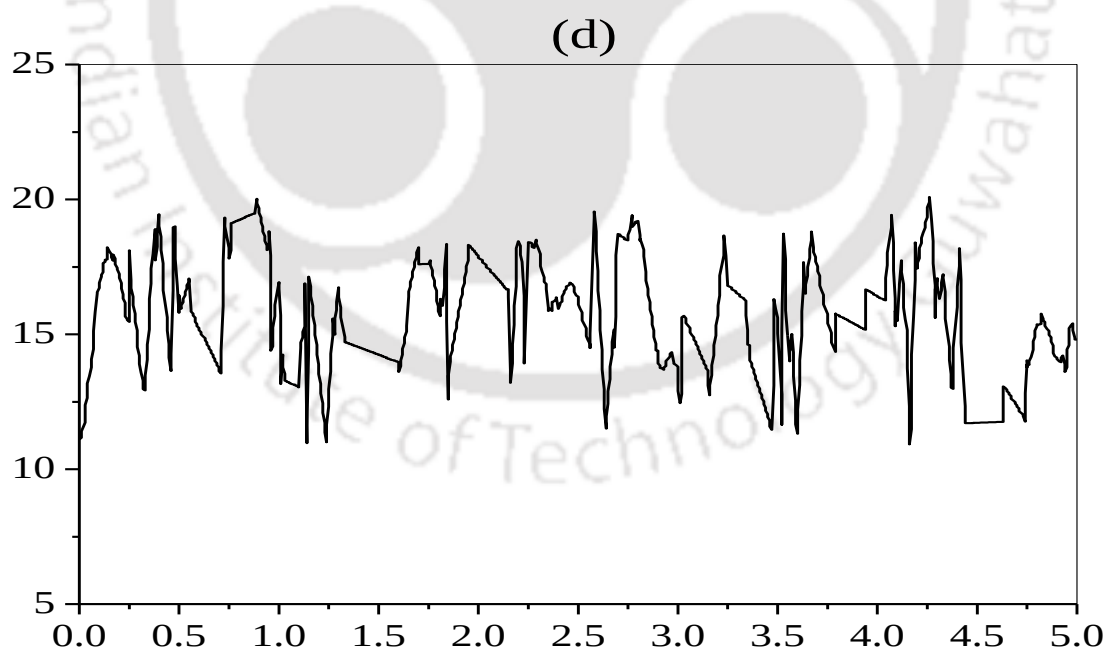
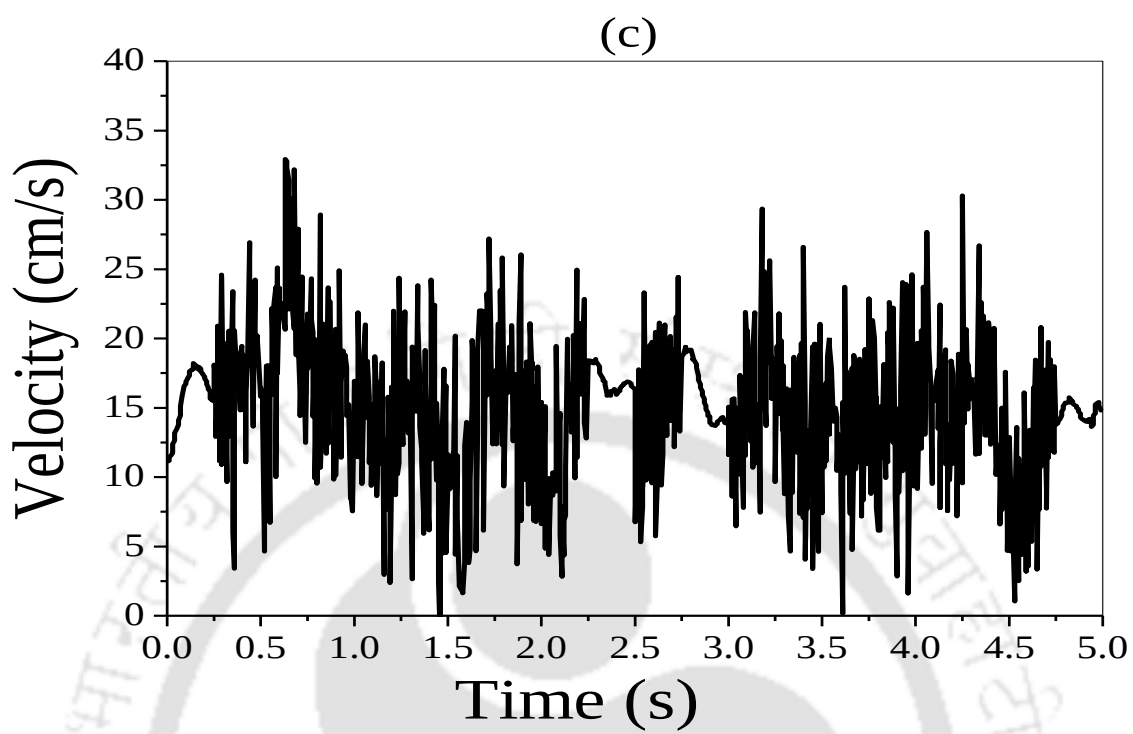
Figure 4: Joint distribution of u' and w'

In order to evaluate the performance of despiking algorithms, we have added Gaussian based random spikes to a clean time series data taken at a propose flow depth which is away from the bed, and the contaminated data were cleaned by the despiking methods. **Islam and Zhu (2013)** states that **Wahl (2003)** and **Goring and Nikora (2002)** algorithms are satisfied well respectively when we have replaced 33% and 5% of the data by spikes. The same result was also observed in the present study. **Parsheh et al. (2010)** developed a method that applies a

preconditioning to the **Goring and Nikora (2002)** phase-space threshold method and applies robust statistics to compute the size of the ellipsoid. Following the removal of spikes, the missing data are replaced by the last valid data point. The method also requires iterations. **Parsheh et al. (2010)** reported that their method significantly improves the power spectra. But, when we have applied **Parsheh et al. (2010)** algorithm in the present contaminated data, it is observed that their algorithm is well fitted when 40% of the data was replaced by spikes. **Islam and Zhu (2013)** developed a despiking method that can determine the cutoff point from the morphology of the data. To achieve this, a bivariate kernel density function was applied to produce a density map of a data cluster and spike clusters, which in turn helped to isolate the data cluster from the surrounding spike clusters. A limitation of **Islam and Zhu (2013)** method is that the peak of the kernel-density function should be located in the data cluster. If the peak is located in the spike cluster, the method will break down. **Islam and Zhu (2013)** purposed kernel density based algorithm worked well when we have replaced 70% of the data by spikes. We have checked the modified singular spectrum method by replacing more than 70% of the data by spikes which is not available in the existing literatures and found that the propose method worked well when 80% of the data was replaced by spikes.

Figure 5(a) shows a portion of a clean time series of contaminated data as shown in Figure 2, and Figure 5(b) shows the addition of 80% spikes to a time series. Figures 5(c, d and e) display the filtered data by **Parsheh et al. (2010)** algorithm, **Islam and Zhu (2013)** algorithm and modified singular spectrum algorithm. We have found that **Parsheh et al. (2010)** method retained all spikes at 80% contamination [Figure 5(c)], while the algorithm proposed by **Islam and Zhu (2013)** partially cleaned the data series [Figure 5(d)] because their method is successfully cleaned the time series when 70% of the data was replaced by spikes. In Figure 5(e), the removed data were replaced by preserving the trend (low frequency components) in the time series signal due to which restructured signal slightly deviates from its original signal. The time averaged of the original and artificially spiked data series was obtained as 0.1512 m/s and 0.1502 m/s respectively but when we have used SSA method for despiking, the time averaged drops to 0.1505m/s, which produced a 0.07% deviation from the time averaged of the original data series.





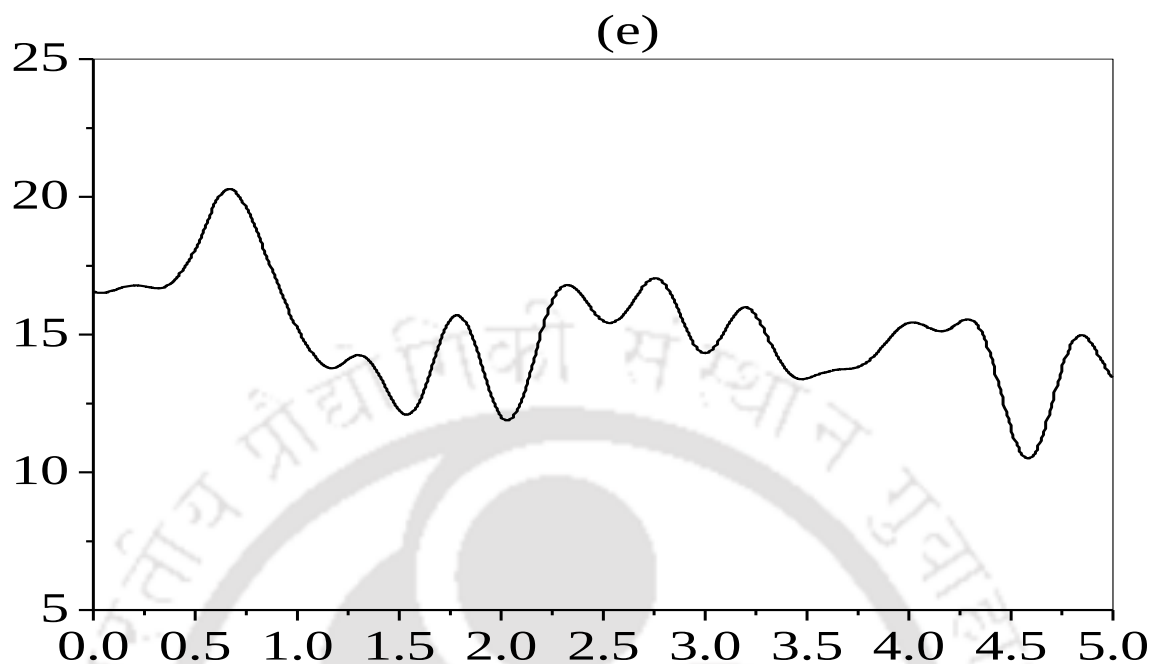


Figure 5: Portion of time series of (a) original signal (b) artificially contaminated signal; (c) despiking effort by Parsheh *et al.* (2010) algorithm; (d) despiking effort by Islam and Zhu (2013) algorithm (e) despiking using method developed in the present study

The developed algorithm has been further tested with experimental observations on curved cross-section sand bed channel by **Deshpande and Kumar (2016a)**. Near bed observation (3.6 cm from the bed) has been taken specifically as near bed observation is highly contaminated owing to high turbulent fluctuation and wall boundary. The observation corresponds to the incipient discharge ($Q=0.025 \text{ m}^3/\text{s}$) with flow depth ($h=13.5 \text{ cm}$). In Figure 6(a), the proposed SSA technique preserves the trend in the time series signal and also removed noise components (spikes). The time averaged of the contaminated and **Deshpande and Kumar (2016a)** filtered data series was obtained as 0.2681 m/s and 0.2695 m/s respectively but when we have used SSA method for despiking, the time averaged increase to 0.2682 m/s, which produced a 0.01% deviation from the time averaged of the contaminated data series. The PSDs of contaminated and filtered ADV time series data using existing methods and proposed technique were also shown in Figure 6(b). When comparing the PSD of ADV data using existing technique with the PSD of contaminated data, it was observed that the noise components are present in the existing

methods. However, proposed technique attenuates the noise components efficiently. It is clear from Figure 6(b) that PSDs satisfies Kolmogorov -5/3 law in the inertial sub range.

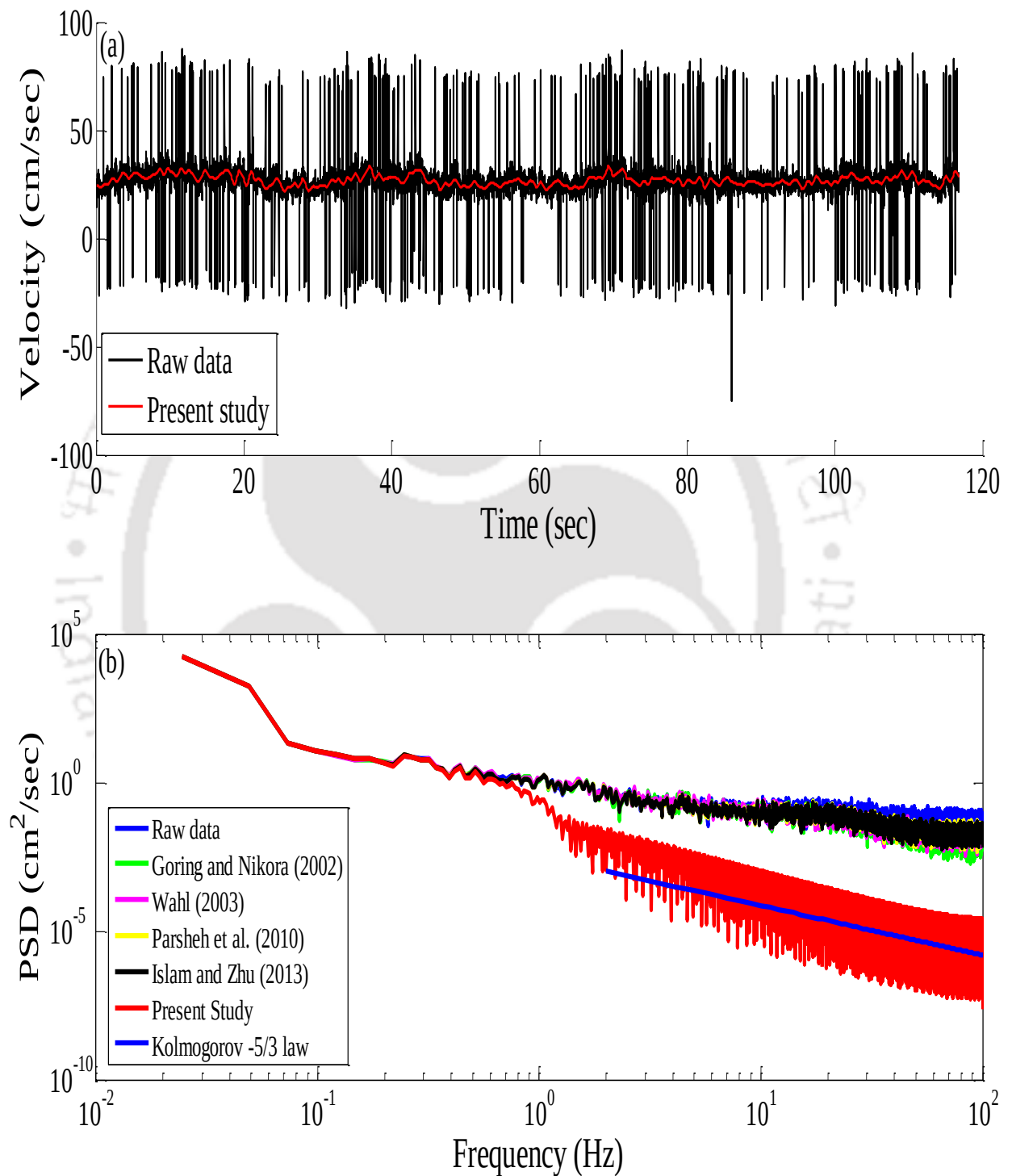


Figure 6: (a) velocity time series data and (b) Velocity power spectra subjected to curved cross section sand bed channel

5. Discussions

We take the results of instantaneous velocity time series in the vegetated sand bed channel whose time series and spectrum were presented in Figure 2 and 3 respectively. The time series data are taken using an ADV having sampling frequency of 100 Hz. We have considered the measurement location by high turbulence intensity creating the job of estimating the accuracy of the SSA algorithm especially interesting. A vertical data point was taken at the vegetated sand bed channel, where the turbulence is highly inhomogeneous and anisotropic and the time series data was significantly contaminated by spikes. For the iteration-based technique of **Goring and Nikora (2002)**, replacement is compulsory to get the next iteration, as the slope of velocity requires to be calculated. Because the technique propose in the present study is iteration free, replacement may be left as an elective procedure. However, to get the power spectra, a full-length velocity time series is mandatory, and therefore the removed data require to be substituted by the interpolated values. Figure 2 shows the comparison among the velocity profiles of contaminated data set, despiked dataset using the proposed method and the method given by existing literatures. The profiles found by developed technique are much cleaner than those of existing methods showing that the steps developed in the present study can recover profiles of ADV time series data in vegetated sand bed channel. In Figure 2, the velocity time series were restructured by replacing spikes with the selective algorithm procedure. It was found that existing method does not retained all spikes at approximately 80% contamination while the algorithm proposed in the present work positively cleaned the velocity time series. In order to show the difference between reconstructed and contaminated time series spectra, power spectra can be plotted in the form of log log scale. Such a plot is beneficial as it basically presents the comparative energy content in variable frequency bands (**Warhaft, 1980**). It is clear from Figure 3 that power spectra of despiked time series data satisfy Kolmogorov's $-5/3$ slope in the inertial sub range. It is remarkable that the power spectra of the contaminated time series may not be precisely the same. The advantage of present technique is that initially high energy power spectra line of despiking ADV data is remains matching with the power spectra for contaminated ADV time series data in the same energy range and same observation is also possible with the existing algorithm (**Wahl, 2003, Parsheh et al., 2010, Islam and Zhu, 2013**). The restructured data series satisfy well with the contaminated data for the high energy range of the spectrum is in good agreement with other restructured methods. Therefore, for flows subject to large scale

coherent vortices at which most turbulence energy is developed by large scale, low frequency fluctuations, the proposed SSA technique should be acceptable for catching the maximum energetic scales of motion. In general, the measured velocity signal (low frequency component) is contaminated by spikes, which is high frequency component. As the multivariate data is one sampled delayed version of original signal, there will be significant difference in the variance of the data and is exploited for efficient decomposition. Therefore, the proposed SSA technique exhibits superior performance than the existing techniques. Moreover, current method work as a data driven based low pass filter, as the filter basis vectors (eigenvectors) are estimated from the data itself. In addition to that, the following are the reasons behind the advantage of current method over existing algorithm

(a) The profiles found by developed technique are much cleaner than those of existing methods showing that the steps developed in the present study can recover profiles of ADV time series data in vegetated sand bed channel.

(b) It was found that existing method does not retained all spikes at approximately 80% contamination while the algorithm proposed in the present work positively cleaned the velocity time series.

6. Conclusions

ADV velocity time series data is generally contaminated with spikes. Therefore, in this chapter, we proposed modified SSA technique to remove spikes from the ADV velocity time series data and compared with existing despiking methods in the literature. The filtered ADV time series data so obtained using present SSA method is fully reconstructed and preserves the trend in the time series signal and also removed noise components (spikes). When the modified SSA method was applied to the contaminated ADV data of vegetated sand bed channel, it was found that the spectral density achieved from the despiked data showed a $-5/3$ slope in the inertial sub range. The analysis of bursting events shows that the modified SSA method removes substantial amount of spikes and preserve the information associated to the structure of turbulence. The probable reason for efficient removal of spikes is that SSA exploits the significant differences in the variance of the data. Therefore, the proposed SSA technique exhibits superior performance than the existing techniques. Moreover, current method work as a data driven based low pass filter, as the filter basis vectors (eigenvectors) are estimated from the data itself. The present

method can be able to clean the artificially high contaminated data when 80% of the data was replaced by spikes. In ADV time series data studied in the present work, high energy components of the power spectra can be able to recover and also it improves the data quality by filtering substantial amount of spikes.



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