

Design of plane sand-bed channels affected by seepage

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Received 2009-05-08

Abstract

The importance of seepage in the design of channels is discussed. Experimental investigations reveal that seepage, either in the downward direction (suction) or in the upward direction (injection), can significantly change the resistance as well as the mobility of the sand-bed particles. A resistance equation relating 'particle Reynolds number' and 'shear Reynolds number' under seepage conditions is developed for plane sediment beds. Finally, a detailed design procedure of the plane sediment beds affected by seepage is presented.

Keywords

Incipient motion · Resistance equation · Sand-bed channels · Seepage · Sediment transport · Shear stress

Acknowledgement

Authors gratefully acknowledge the anonymous reviewers for their precious comments and suggestions. Financial support received from the Ministry of Water Resources, Government of India through The Indian National Committee on Hydraulic Research (INCH) to the first author is gratefully acknowledged.

1 Introduction

A study of the effect of seepage flows on incipient motion (detachment of particles from the bed) is of great interest since this problem is related to the solution of important practical engineering problems. Channel seepage has been identified as a significant loss from the irrigation channels from both water quantity and environmental degradation perspectives. Seepage losses from alluvial channels have been estimated to range from 15 to 45 % of total inflow [1]. Recently, the Australian National Committee on Irrigation and Drainage [2] has indicated that a significant amount of water (10 to 30 %) is lost in the form of seepage from alluvial channel. Losses from on-farm channel systems to the ground water system have been variously estimated to contribute about 15 to 25 % of total ground water accessions [3]. Thus, it is important to study and analyze seepage phenomena undergoing in the alluvial channels [4]. Apart from the loss of water, seepage can significantly alter the hydrodynamic behavior of alluvial channels, which is considered in this paper.

Generally two types of seepage flow can occur in the field, injection (upward seepage: ground water contribution to the channel) and suction (downward seepage: contribution of water from the channel to the ground water). There are many contradicting reports in the published literature about the hydrodynamic effects of seepage on hydraulic resistance, stability and sediment transport characteristics of the alluvial channels. A complete discussion is given by Rao and Sitaram [5] and most recently by Lu et al. [6]. Stability of the channel, in this paper, refers to the change in the rate of sediment transport in the channel due to seepage. Some past research on the sediment transport due to seepage are discussed next.

Watters and Rao [7], Willetts and Drossos [8], Maclean [9], and Rao and Sitaram [5] reported that suction increases the bed material transport, where as Harrison [10], Burgi and Karaki [11], Oldenzel and Brink [12], and Nakagawa et al. [13] reported that suction decreases the mobility of bed material as compared to no-seepage. Similarly Burgi and Karaki [11], Oldenzel and Brink [12], Richardson et al. [14], Nakagawa et al. [13], Cheng [15], and Cheng and Chiew [16] reported that injection increases the transport rate or it is ineffective in pro-

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moting bed load transport [10] when compared to no-seepage transport rate. But Watters and Rao [7], Rao and Sitaram [5] reported that injection reduces the sediment transport rate and increases the stability of the particles or it does not aid in initiating their movement.

The point of discussion in the above referenced studies is to ascertain the effect of seepage on the hydrodynamics of the channel. The issue of whether seepage reduces or enhances sand-bed stability is still a matter of debate and considerable work is still needed to explore and better understand the phenomenon [6]. Hence, in order to clarify the contradictory findings 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 on a larger scale. Hence experiments are conducted on a higher scale tilted flume of an effective length 25 meters (length of seepage zone is 20 meters) and width 1.8 meters in the Hydraulics laboratory of Indian Institute of Science, Bangalore.

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. It is very interesting to note that the design methods which are available at present do not explicitly take into account the seepage effects. Hence an attempt is made here to develop a design procedure at threshold condition due to seepage.

Design of channel at incipient motion or in general requires a resistance equation. However, present existing resistance equations do not consider the seepage effect. Hence present work also tries to develop a resistance equation which will consider the seepage effect at incipient motion of the channel.

2 Seepage effects on sediment transport – a comparison

Many researchers have analyzed seepage effects on various hydraulic parameters like the rate of sediment transport, velocity profiles, bed shear stress, and turbulence. However, the following analysis compares the causes of different opinions on the rate of sediment transport due to seepage.

Waters and Rao [7] used plastic spheres of 3.75 inch (9.5 cm) diameter with a fluid 200 times more viscous than water in order to provide dynamic similarity and to study the hydrodynamic effect of seepage on bed particles. They measured the drag and lift forces on the plastic spheres and found that: (1) injection decreased the drag regardless of the position of sediment particle and (2) seepage increases or decreases the lift acting on a particle on a plane bed (like the natural sediments in rivers) depending on whether the seepage is upward or downward. Judging from the viewpoint of drag forces, injection inhibits the motion of a bed particle while suction enhances its motion. From the viewpoint of lift forces, injection inhibits the motion of a plane bed particle and the opposite result holds for the case of suction.

Willems and Drossos [8] used a narrow flume (76 mm wide) with a suction zone of size 76 mm by 125 mm and observed that the grains moved at a faster rate in the suction zone than elsewhere in the flume. They used sand sizes of 0.10 mm and 1 mm. The fine sand (0.10 mm) which was in ripple regime was totally immobilized in the suction zone and the medium sized sand (1.00 mm) moved at a faster rate (i.e., size effect of bed material is felt in seepage studies).

Maclean [9] analyzed the effect of suction on sediment transport in a tilted flume of 75 mm wide and 5 meters long having a suction length of 130 mm. He tested sand of a uniform size of 1 mm. His major findings are that the suction enhances the sediment transport rate and increased shear stress values are observed in the suction zone.

Rao and Sitaram [5] worked with 0.1575 m wide flume on the seepage effects on incipient motion of sand-bed particles. They found that seepage through a sand-bed in a downward direction (suction) 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 (injection) increases the stability of bed particles, it does not aid in initiating their movement.

Nakagawa et. al. [17] tried to explain the effect of suction and injection on the transport process with the help of 'pick-up rate' and 'step length' of bed particles. They verified their model by using a flume of 800 cm length (of which 30 cm was seepage length) and 33 cm width. Sand used are of uniform size ($d_{50} = 20$ mm, 16.4 mm 13.6 mm). They concluded that bed load transport is promoted by injection and suppressed by suction in the seepage length of only 30 cm. It may be considered as a localized phenomenon over a small seepage zone. Nakagawa et. al. [17] worked on particles of larger size. Generally the large particle sizes make the bed highly porous and are not representative of the sand-bed channels.

Harrison [10] concluded from his experiments that injection was ineffective in promoting bed load transport and suction slightly increases bottom roughness and decreases its erodibility due to the formation of mud seal. Due to this formation of mud seal, the seepage rate will certainly decrease and this may be the reason as to why the boundary layer may not be affected by the suction.

Burgi and Karaki [11] studied the stability of banks of alluvial channels subjected to seepage. At low velocities (less than 30 cm/s) erosion was primarily due to the seepage (injection) gradient and the main channel flow. The injection reduced the stability of channel banks; on the other hand suction has increased the stability of side slopes compared to no-seepage condition. This work is mainly on banks stability, but the present study pertains to the bed of the sand-bed channels.

The controversy is mainly due to the Oldenzial and Brink [12] results. Some of the comments given by them are: 1. Suction always decreases the rate of sand transport while blowing (injection) increases the transport rate. 2. The grain size of the

transported particles is greater than that of the bed particles. This effect is enhanced with suction and is less pronounced for blowing. 3. It will be clear that with blowing (injection) the horizontal velocity decreases near the sand-bed, while this velocity increases with suction. The comments given by them are confusing. The second comment says that the grain size of the transported particles is greater than that of the bed particles; this effect is enhanced with suction. On the one hand they claim that bed particles are stable against suction and on the other hand they argue that the near bed velocities increase with suction. The velocity profile given by Willets and Drossos [8] shows that the stability of the particles decreases as the near bed velocity increases. Generally it is expected that as the near bed velocity increases the particle reaches critical condition, which leads to destabilization. Any way Willets and Drossos [8] opposed the idea of [12] introducing the correction “ η ” by saying that it would be more appropriate if the change in the shape of stream wise velocity profile is considered after the application of seepage from section to section and the momentum flux changes are computed using the modified velocity profiles.

Richardson et. al. [14] worked on the effects of injection in a re-circulating flume 0.30 m wide and 9.45 m long. The inflow (injection) gallery is located for $1/3^{rd}$ length of the main channel. Sands tested are 0.62 and 1.76 mm (sand gradation coefficient $\sigma = 2.70$ and 5.00 for the sand sizes tested, respectively). They suggested that injection increases bed erosion, whereas suction tends to inhibit sediment motion. Actually they have worked on non-uniform sands and generally it is believed that for non uniform sands the interlocking may be high and it acts like a mud seal (as in the Harrison [10] case) and eventually the rate of seepage may be less and this could be the reason that they got the opposite results. It is strongly believed that the reason for the contradictory results is perhaps that the flow conditions of others might have not reached the pseudo incipient conditions. Once such conditions are not reached there is every possibility for making contradictory interpretations. Unfortunately, precise data is not available from the literature to prove this point.

From the various studies cited above and based on the majority of studies, one may conclude that during downward seepage (suction) the apparent weight of the particle increases whereas the effective velocity acting on the particle in the boundary layer also increases. Therefore, as the seepage intensity increases, the particle should reach a critical condition called pseudo incipient condition beyond which the particle moves. In the case of injection, it can be similarly argued that during injection the apparent particle weight decreases whereas the effective velocity also decreases. Therefore, it is expected that injection does not aid the incipient motion and hence inhibits the sediment movement. The opposite is true in case of suction in which downward seepage enhances the sediment transport and thus aids sediment movement. Experimental setup plays a key role in deciding the appropriateness and its applicability of the available equations in designing the canals. Many of the research findings are per-

taining to local conditions in the sense the application of seepage is not for the entire length of the flume or the size of the flume may be very small. In natural canals, seepage occurs over entire length without having any side wall effect, which is not reflected by the works presented by peers. For calculating shear velocity, the measurement of water surface slope is very critical. But due to the short length of seepage application in the experimental work, the accuracy of the water surface slope (S_w) is questionable. The slope of the S_w after and before application of the seepage will be affected, by the seepage. Suppose the S_w in the seepage zone is considered, will be influenced by the no-seepage zone before and after the seepage zone. Hence, S_w from longer seepage length channels may not be having the influence of the no-seepage zone and may be permitted in the usage of the designs. In many of the research findings the seepage length is very short when compared to the length of the flume or not been presented their experimental data or they might have not been worked on the threshold phenomenon with seepage. Experiments on larger flume with a seepage facility all over its length can explain the real field conditions. This will help in explaining the sediment particle motion affected by seepage at field conditions and in this way eliminates the above mentioned controversy created by peers.

3 Experimentation

The experiments are conducted in three laboratory flumes. Salient features of all the flumes are given in Table 1.

Tab. 1. Salient features of the laboratory flumes

Flume	Length	Width	Seepage Length	Sand-bed thickness	Flume type
	[m]	[m]	[m]	[m]	
Flume-1	25.00	1.80	20.00	0.30	Tilted
Flume-2	14.16	0.615	12.75	0.23	Horizontal
Flume-3	3.60	0.1575	2.40	0.05	Tilted

A sand-bed is laid on a perforated sheet at an elevated level from the channel bottom covered with a fine wire mesh (to prevent the sand falling through) to facilitate the seepage flow through the sand-bed. The space between the perforated sheet and the channel bottom act as a pressure chamber to allow seepage flow through the sand-bed either in a downward or an upward direction by creating a pressure lower or higher, respectively, than the channel flow. Photograph of Flume-1 is shown in Figs. 1.a and 1.b.

3.1 Sand sizes for experiments

Different sizes of sands (particle size, d), have been used for both seepage and no-seepage studies. No-seepage studies are conducted for generalizing the resistance equation which will cover both cases (seepage and no-seepage). Three sizes i.e., $d_{50} = 0.56, 0.65,$ and 1.00 mm are used as bed material in the three



Fig. 1.a. Downstream view of the experimental setup (1.80 m wide tilted flume at Indian Institute of Science, Bangalore, INDIA)

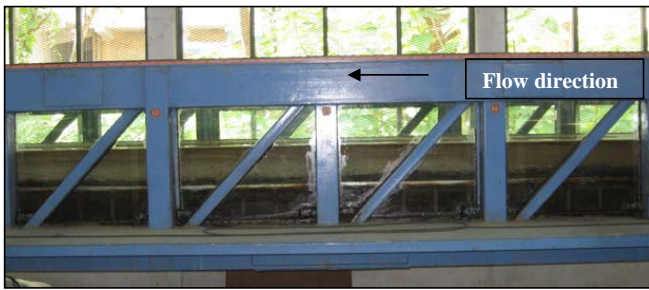


Fig. 1.b. Side view of the experimental setup

flumes for seepage studies and five sizes i.e., $d_{50} = 0.44, 0.65, 1.09, 1.77,$ and 8.00 mm are used for no-seepage studies. All sizes have fairly uniform material with gradation coefficient $\sigma = 0.5(d_{84}/d_{50} + d_{50}/d_{16})$ in the range of 1.08 to 1.3, where $d_{16}, d_{50},$ and d_{84} are the sizes pertaining to 16, 50, and 84 percent finer, respectively. In order to substantiate the results, various researchers' data [5, 18–21] have been taken from the literature and analyzed in this paper.

3.2 Procedure and measurements

Initially, the sand-bed is made plane for all the experiments with a required bed slope S_o . Then inflow discharge Q is allowed. A tailgate at the downstream end of the channel is used to adjust the flow depth. After reaching stable conditions, slow seepage flow q_s (suction or injection) is allowed to set the condition to pseudo incipient motion. Before and after the application of seepage, the water surface elevations are measured with an accuracy of ± 0.015 mm of water head at regular intervals along the channel by using a digital micro manometer in order to determine the water surface slope S_w . Flow depths along the central line of the channel are measured at regular intervals using a point gauge, and the average depth y is obtained. The amount of Q and q_s are measured either volumetrically or with calibrated orifice meters. Thus, the basic variables $S_o, Q, q_s, S_w,$

and y are obtained in every experimental run and given in Tables 2a and 2b. Pressure tapings are provided at some sections inside the sand-bed to measure the seepage gradients to verify the uniformity of seepage flow. Yalin's [22] criterion is used in this paper for setting the bed condition to be incipient. Experiments are conducted by maintaining two conditions: 1) Aspect ratio $B/y \geq 4$ (B is the width of the channel) and 2) Relative roughness height $y/d \geq 3$. It is generally difficult in practice to accurately measure the relative roughness heights y/d less than 3. These two constraints are self-imposed for a better understanding of the concepts rather than entering into misjudgments due to unavoidable experimental errors that are likely to creep into the analysis and the understanding of the concept.

4 Data analysis

4.1 Incipient motion

As mentioned above, incipient motion experiments are conducted with and without seepage. For no-seepage case, Shields' [23] criterion is being used to validate the experiments for incipient motion. According to this, there is a definite relationship between $\tau_{co}/[(\gamma_s - \gamma)d]$ and R_* at incipient motion (τ_{co} is the Shields' critical shear stress, γ_s and γ are the unit weight of the sand and water, $R_* = u_*d/\nu$ is Shear Reynolds number, u_* is shear velocity and ν is kinematic viscosity of water). Many researchers have given the explicit form of Shields' relationship [5, 24–27, 30, 31, 28, 29]. In the present work, the relationship given by Rao and Sreenivasulu [31] has been used. Fig. 2 shows the present experimental observations with the data taken from the literature on Shields' diagram. As it can be seen from Fig. 2, all the data points lie on the Shields' curve and this validates the present experimental runs of no-seepage incipient motion.

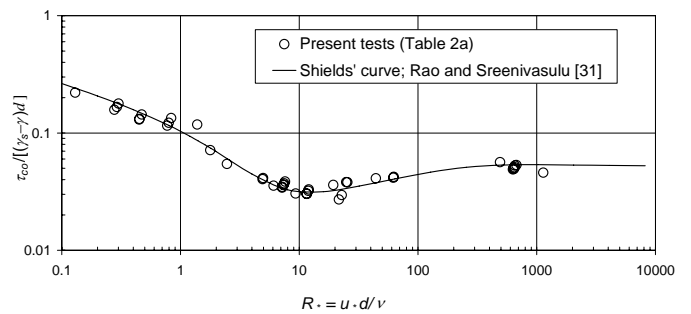


Fig. 2. Shields' diagram / Curve

4.2 Suction effects on inception of bed particles

Rao and Sitaram [5] have reported that Shields' criterion is not valid for incipient motion with seepage and proposed a relationship for threshold shear stress due to seepage by conducting experiments in a smaller laboratory flume of 0.15 m width as

$$\ln(\tau_{bo}/\tau_{co}) = -0.2525(\tau_{cs}/\tau_{co})^{-2.917} \text{ for } \tau_{bo}/\tau_{co} < 1 \quad (1)$$

here $\tau_{bo} = \gamma y S_{fo}$ is the bed shear stress without seepage; the subscript 'o' refers to no-seepage condition and $\tau_{cs} = \gamma y_s S_{fs}$ is

Tab. 2.a. Experimental data without seepage

Source	d_{50} [mm]	No. of tests —	γ_s/γ —	$10^2 y$ [m]	$10^4 Q$ [m ³ /s]	$10^4 S_f$ —	B [m]
INCIPIENT MOTION RUNS							
Present Tests	0.44	4	2.65	2.07 – 3.18	6.10 – 9.90	7.89 – 13.17	0.1575
	0.65	4	2.65	3.00 – 5.89	42.75 – 85.43	5.53 – 11.81	0.6150
	1.00	2	2.66	3.12 – 3.84	13.13 – 14.42	12.78 – 20.24	0.1575
	1.77	2	2.65	1.94 – 3.35	8.51 – 17.54	36.81 – 62.46	0.1575
	8.00	8	2.66	2.59 – 3.41	22.60 – 32.36	205.5 – 254.3	0.1575
Rao and Sitaram [5]	0.32	2	2.64	1.90 – 2.68	6.68 – 8.00	8.10 – 11.17	0.1575
	0.80	1	2.64	2.70	13.00	17.52	0.1575
	1.30	1	2.67	3.15	15.75	28.33	0.1575
Yalin and Karahan [18]	0.10	1	2.65	0.65	2.25	30.00	0.15
	0.14	1	2.65	0.55	1.69	30.00	0.15
	0.19	1	2.65	0.57	1.78	30.00	0.15
	0.40	1	2.65	0.47	1.50	50.00	0.15
	0.56	1	2.65	0.47	1.52	60.00	0.15
	1.00	1	2.50	0.63	2.72	100.00	0.15
Ashida and Bayazit [19]	6.40	1	2.656	2.40	24.00	250.00	0.20
	12.00	1	2.656	3.65	50.00	250.00	0.20
Mantz [20]	0.015	1	2.66	6.01	26.30	0.92	0.30
	0.030	3	2.66	2.27 – 5.86	9.93 – 32.30	1.52 – 3.46	0.30
	0.045	3	2.66	2.43 – 5.96	11.90 – 36.10	1.80 – 4.06	0.30
	0.066	3	2.66	2.46 – 6.07	13.50 – 42.80	2.42 – 5.14	0.30
Ippen and Verma [21]	2.00	5	1.28	6.82 – 16.20	—	7.00 – 29.00	0.6096
	3.17	5	1.28	6.28 – 12.66	—	7.00 – 29.00	0.6096
	3.17	5	2.38	15.05 – 40.64	—	7.00 – 29.00	0.6096
	4.00	5	2.38	16.20 – 40.64	—	7.00 – 29.00	0.6096

Tab. 2.b. Experimental data with seepage

Source	d_{50} [mm]	No. of tests —	γ_s/γ —	$10^2 y_s$ [m]	$10^4 Q$ [m ³ /s]	$10^4 q_s$ [m ³ /s]	$10^4 S_f$ —	B [m]
INCIPIENT MOTION WITH SUCTION RUNS								
Present Tests	0.56	4	2.67	7.56 – 10.50	290 – 520	110 – 260	0.88 – 4.08	1.80
	0.65	32	2.65	3.04 – 9.10	51.88 – 172.88	2.60 – 72.70	4.48 – 19.63	0.6150
	1.00	5	2.66	3.33 – 5.14	12.36 – 18.63	3.92 – 6.43	9.29 – 13.03	0.1575
Rao and Sitaram [5]	0.58	3	2.64	1.31 – 3.34	4.93 – 13.10	1.25 – 2.00	13.25 – 19.41	0.1575
	0.80	3	2.64	1.39 – 1.85	6.40 – 8.10	1.51 – 2.30	20.00 – 34.12	0.1575
	1.00	4	2.64	1.22 – 2.66	5.95 – 13.55	1.40 – 2.19	20.16 – 58.92	0.1575
	1.30	3	2.67	2.04 – 2.21	9.90 – 11.44	1.88 – 3.62	23.59 – 40.97	0.1575
	3.00	2	2.67	2.80 – 3.08	22.62 – 23.00	4.57 – 5.70	75.48 – 81.60	0.1575
	INCIPIENT MOTION WITH INJECTION RUNS							
Present Tests	0.65	1	2.65	6.36	82.14	– 25.54	5.86	0.6150
	0.58	2	2.64	2.81 – 4.39	11.02 – 16.78	– 2.62 to – 2.74	16.27 – 22.49	0.1575
Rao and Sitaram [5]	0.80	1	2.64	3.37	12.65	– 3.69	16.34	0.1575
	1.00	2	2.64	3.71 – 3.75	15.59 – 16.08	– 2.60 to – 4.4	21.47 – 24.58	0.1575
	1.30	1	2.67	4.37	23.75	– 3.60	13.39	0.1575

the critical bed shear stress under seepage condition; the subscript 's' refers to seepage condition. In the present analysis it is attempted to generalize their work by extending the experiments to the larger scaled tilted flume. The following plot between the dimensionless parameters τ_{cs}/τ_{co} and τ_{bo}/τ_{co} in Fig. 3 shows good correlation which strengthens Eq. (1).

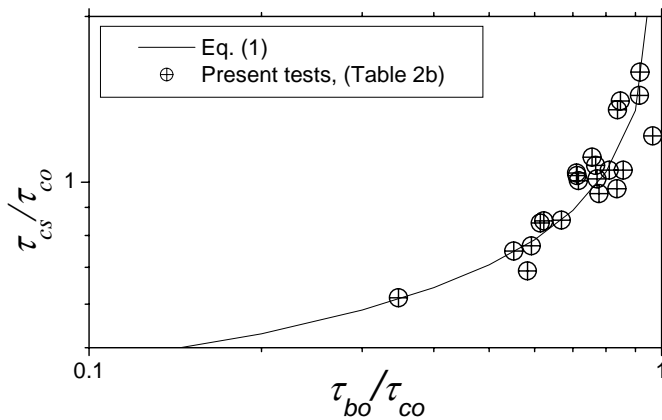


Fig. 3. Incipient motion sand-bed particles with suction

From Fig. 3, it is evidenced that suction reduces the stability of the bed particles and initiates their mobility. All the circles in Fig. 3 belong to the pseudo incipient motion and also to experiments with wider ranges of flume sizes, i.e., the width of flume ranges between 0.1575 m and 1.8 m. The statement, 'suction enhances the bed material transport' may be explained as follows. With the help of flow visualization technique, Willets and Drossos [8] have presented the velocity distribution in the suction zone on a flat bed as shown in Figure 4. Later, the experimental data collected by a hot-wire anemometer in a wind tunnel by Maclean [32] fits well with this profile. It can be observed in the Figure 4, that at the beginning of the suction zone the velocity is zero on the bed. But, in the suction zone and towards the flow direction, the velocity profile dipped inside the permeable bed and hence the bed particles are exposed to higher magnitude of velocity. Hence, in the suction zone the hydrodynamic forces are dominating the resistive forces.

Rao and Sitaram [5] investigated the behavior of turbulence characteristics (using a hot film anemometer) and velocity profiles for the conditions of no-seepage and seepage. They found that the velocities as well as the turbulence intensities increase with suction and decrease with injection when compared with the same under no-seepage conditions and they are in agreement with the findings of Schlichting [33]. Therefore, the bed particles are generally subjected to more hydrodynamic forces due to suction, whereas they are reduced due to injection. Hence, the stability of bed particles is generally dependent on the relative magnitudes of hydrodynamic forces acting on the particles and their resistive forces, and these two types of forces are interdependent. In the present experiments, suction decreases the stability of bed particles, which may be interpreted as the increase in hydrodynamic forces is more than the increase in resistive

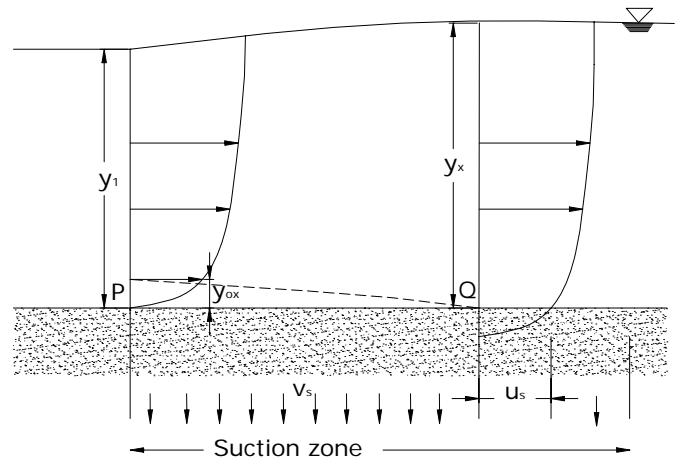


Fig. 4. Velocity profile over a permeable bed due to suction

forces.

Considering the views of the present experiment as well as those by Watters and Rao [7], Rao and Sitaram [5], the condition of upward seepage is not so important (as it does not aid the particle movement) in designing the alluvial channel, whereas the case of downward seepage is vulnerable (because suction enhances the motion of sediment particles). Hence the present work of design methodology considers only suction because downward seepage is more critical for the stability of the channel.

4.3 Resistance equation at incipient motion with and without seepage

The present theoretical and experimental investigations [13], [34], [35], [6] do not give an unequivocal answer to the question about the effect of seepage flow on the main flow. Chen and Chiew [34] showed that the customarily used law of the wall (logarithmic law) is also not applicable to open channel flow subjected to bed suction. Sitaram and Rao [35] have ascertained that Manning's n values are significantly affected from the seepage, thus there is a need to have a different kind of equation. Hence it has become necessary to develop a new resistance equation for plane sediment beds affected by bed suction.

Hence an attempt is made here to develop a resistance equation, based on the experimental data, for the average velocity in terms of shear Reynolds number and particle size. Yang [36] proposed a relation at incipient motion for the ratio of average velocity u and fall velocity ω as a function of shear Reynolds number R_* :

$$u/\omega = f_1(R_*) \quad (2)$$

Eq. (2) can be also expressed as:

$$\frac{ud/v}{\omega d/v} = f_1(R_*) \quad (3)$$

here $ud/v = R$ is the particle Reynolds number and $\omega d/v$ is the fall velocity Reynolds number.

According to Cheng [37], $\omega d/\nu$ is a function of the dimensionless particle size, d_* , which is a function of R_* based on the Shields' relationship. Thus the fall velocity Reynolds number can be written as:

$$\omega d/\nu = f_2(d_*) = f_3(R_*) \quad (4)$$

From the above analysis, the Eq. (2) can be written as:

$$ud/\nu = f_1(R_*) \cdot f_3(R_*) = f_4(R_*) \quad (5)$$

Function f_4 has to be determined through the experimental observations. The arguments given by Yang [36] and Cheng [37] are for incipient motion without the consideration of seepage. Here, it is hypothesized that as those arguments are valid for the incipient motion, they will also be true for incipient motion in the seepage case. Experimental observations of R and R_* (as shown in the Tables 2a and 2b) have been plotted in Figure 5 and it is found that they are uniquely related and fall on a single curve. The resulting equation representing the curve is given below:

$$u_s d/\nu = f_4(R_*) = R_* \left(13e^{-0.2\sqrt{R_*}} + 6.5 \right) \quad (6)$$

Eq. (6) is valid for incipient motion condition with and without seepage.

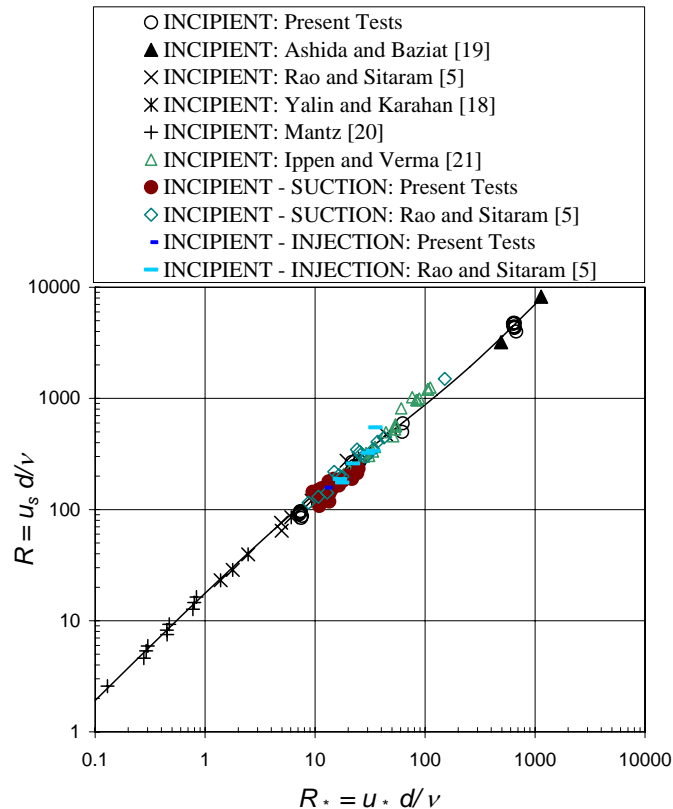


Fig. 5. Resistance equation for plane sand-beds at incipient motion with and without seepage

4.4 Methodology

Channel design involves the stabilization or realignment of an existing stream, or it may involve the creation of an entirely new

channel. As said earlier, this requires some governing equations through which a field engineer can design the channel. There are five design variables, namely, particle size d , flow depth y_s , water discharge Q , energy slope S_{fs} and seepage velocity V_s involved in the design of a channel. The resistance equation is one of the essential equations for the design of a channel, which has been developed in section 4.3. An incipient motion channel (with and without seepage) is that in which movement of the bed materials are negligible when the flow conditions are critical. Here, the channel is assumed to be stable if the design stress is below the critical stress. Thus another requirement is to have the governing equations of stresses which relate to the seepage phenomena for designing the incipient motion channel under seepage condition. The equation for stresses relating with seepage has been developed as follows:

According to Rao and Sitaram [5], the relation between the bed shear stresses with and without seepage and Shields' critical shear stress can be explicitly expressed as:

$$\tau_{bs}/\tau_{bo} = (\tau_{bo}/\tau_{co})^{\pm 2.2N} \pm N \quad \text{valid for } \tau_{bo}/\tau_{co} < 1 \quad (7)$$

here τ_{bs} is the bed shear stress with seepage (τ_{bs} is equal to τ_{cs} at incipient motion with seepage), $N = (2\rho u_s V_s)/\tau_{bo}$ is the seepage intensity parameter which signifies the relative intensity of seepage applied on the bed for given flow conditions, u_s is the average velocity under seepage condition and $V_s = q_s/(L*B)$ is the seepage velocity.

It can be seen from the Eqs. (1) and (7) that τ_{bo} is common in those two equations. So by eliminating τ_{bo} with some algebraic manipulations, one can express, $N = f(\tau_{cs}/\tau_{co})$ as

$$N = \frac{(\tau_{cs}/\tau_{co}) - \left[\exp\left(-0.2525 \left[\tau_{cs}/\tau_{co}\right]^{-2.917}\right) \right]^{2.2N+1}}{\exp\left(-0.2525 \left[\tau_{cs}/\tau_{co}\right]^{-2.917}\right)} \quad (8)$$

Eq. (8) plays a vital role in designing the plane sand-bed channels subjected to seepage. Generally it is known from the Shields' relationship that as soon as the τ_{bo} reaches near to τ_{co} , the bed particles attain incipient motion. From Eq. (8), for given τ_{bo} and d , the quantity of seepage can be estimated at threshold condition. Thus there are two equations available, i.e., Eqs. (6) and (8) to design the incipient motion channel with suction. All other equations, which are involved in the design, are related to these two equations either directly or indirectly.

5 Design procedure

A design procedure is developed based on Eqs. (6) and (8), which emerged from the data analysis. As mentioned above, there are five design variables (d , y_s , Q , S_{fs} and V_s) and out of these at least three must be known to solve for the remaining two variables. There are ten possible design problems of stable channels, as shown in Table 3, in which the known variables

are marked with a tick (\surd) and the unknown variables with a question mark (?). However, one should note that unit weights of both fluid γ and sediment material γ_s , and kinematic viscosity ν of the fluid (channel width B assumed to be wide) are assumed to be known in all types of problems. Such a design procedure is valid for plane sediment beds consisting of fairly uniform sized material of sizes ranging from very fine sands of 0.015 mm to gravel of 12 mm in wide rectangular channels under fairly uniform flow conditions. Problems 1, 2, 4, and 7 are straightforward and no iteration is required. The main problem of convergence comes with the Eq. (8) i.e., $N = f(\tau_{cs}/\tau_{co})$. With Eq. (8), the evaluation of N from known τ_{cs}/τ_{co} is easier than evaluating τ_{cs}/τ_{co} from the known value of N . Hence, at the time of refining the value of τ_{cs}/τ_{co} , the convergence comes in to picture. The problems 3, 5, 6, 8, 9, and 10 require the refinement of τ_{cs}/τ_{co} in their design procedure. Refinement of τ_{cs}/τ_{co} requires a number of iterations and thus takes more time in solving the unknown parameters. Here it is found that at each level of iterations, errors are decreasing and thus making the iteration process a stable one.

Design Type - 1

Known variables : y_s , d and S_{fs}

Unknown variables : Q and V_s

Compute τ_{co} with Shields' curve. Now compute $\tau_{cs} = \gamma y_s S_{fs}$. Compute τ_{bo} with Eq. (1). Compute N with Eq. (8). Compute u_s by the Eq. (6). Now compute $Q = u_s y_s B$. Finally find V_s by the definition of N .

Design Type - 2

Known variables : d , Q and S_{fs}

Unknown variables : y_s and V_s

Compute τ_{co} with Shields' curve. By assuming y_s , find $\tau_{cs} = \gamma y_s S_{fs}$. Compute u_s by the Eq. (6). Now refine y_s by using the relation $y_s = Q/(u_s B)$ and repeat the procedure until y_s stabilizes. Now the correct ratio of τ_{cs}/τ_{co} can be found. Find τ_{bo} with Eq. (1). Find N with Eq. (8). Finally find V_s by using the definition of N .

Design Type - 3

Known variables : d , S_{fs} and V_s

Unknown variables : y_s and Q

Compute τ_{co} with Shields' curve. By assuming y_s , find $\tau_{cs} = \gamma y_s S_{fs}$ and then the ratio of τ_{cs}/τ_{co} can be found. Compute u_s with Eq. (6). Find τ_{bo} with Eq. (1). Find N by using its definition. Now refine τ_{cs}/τ_{co} with Eq. (8). Compute again the u_s by the Eq. (6). Find the new value of $y_s = \tau_{cs}/(\gamma S_{fs})$. Repeat the procedure until the value of \bar{y}_s stabilizes. Finally find Q with $Q = u_s y_s B$.

Design Type - 4

Known variables : y_s , d and Q

Unknown variables : S_{fs} and V_s

Compute τ_{co} with Shields' curve. Find u_s using the relation $u_s = Q/(y_s B)$. Find shear velocity, u_{*s} by using the Eq. (6). Now τ_{cs} can be computed by using $\tau_{cs} = \rho u_{*s}^2$. Now find S_{fs} using the relation $S_{fs} = \tau_{cs}/(\gamma \bar{y}_s)$. Find τ_{bo} by using the Eq.

(1). Find N by using the Eq. (8). Finally find V_s by using the definition of N .

Design Type - 5

Known variables : d , y_s and V_s

Unknown variables : Q and S_{fs}

Compute τ_{co} with Shields' curve. By assuming the ratio of τ_{cs}/τ_{co} , find the value of τ_{cs} . Then compute u_s with Eq. (6). Now find τ_{bo} with Eq. (1). Find N by using its definition. Now refine τ_{cs}/τ_{co} with Eq. (8). Compute again the u_s by the Eq. (6). Repeat the procedure until the value of u_s stabilizes. Now compute Q with $Q = u_s y_s B$.

Design Type - 6

Known variables : d , Q and V_s

Unknown variables : y_s and S_{fs}

Compute τ_{co} with Shields' curve. By assuming τ_{cs}/τ_{co} , compute the value of τ_{cs} . Now compute u_s with Eq. (6). Compute τ_{bo} with Eq. (1). Find N by using its definition. Now refine τ_{cs}/τ_{co} with Eq. (8). Compute again the u_s by the Eq. (5). Now find a new y_s by using the relation $Q = u_s y_s B$. Repeat the procedure until the value of y_s stabilizes. Finally find S_{fs} by using $\tau_{cs} = \gamma y_s S_{fs}$.

Design Type - 7

Known variables : y_s , Q and S_{fs}

Unknown variables : d and V_s

Compute $u_s = Q/(y_s B)$. Compute $\tau_{cs} = \gamma y_s S_{fs}$. Compute $u_{*s} = \sqrt{\tau_{cs}/\rho}$. Now compute d with Eq. (5). Compute τ_{co} using Shields' curve. Find τ_{bo} with Eq. (1). Find N by using the Eq. (6). Find V_s by using the definition of N .

Design Type - 8

Known variables : y_s , S_{fs} and V_s

Unknown variables : d and Q

By assuming d , compute τ_{co} from Shields' curve. Now compute τ_{cs} by using $\tau_{cs} = \gamma y_s S_{fs}$. Compute τ_{bo} with the help of the Eq. (1). Compute u_s by the Eq. (6). Find N by using its definition. Now refine τ_{cs}/τ_{co} with Eq. (8). Now compute a refined value of d by Shields' curve. Repeat the procedure until the value of τ_{co} stabilizes. Finally find $Q = u_s y_s B$.

Design Type - 9

Known variables : Q , S_{fs} and V_s

Unknown variables : d and y_s

Assume \bar{y}_s and compute τ_{cs} with $\tau_{cs} = \gamma y_s S_{fs}$. By assuming τ_{cs}/τ_{co} , the value of τ_{co} can be estimated. Now compute d with Shield's curve. Then find u_s with Eq. (6). Now compute τ_{bo} with Eq. (1). Find N by using its definition. Compute τ_{cs}/τ_{co} with Eq. (8). A new value of y_s can be now found by the relation $y_s = Q/(u_s B)$. Repeat the procedure until the value of y_s stabilizes.

Design Type - 10

Known variables : y_s , Q and V_s

Unknown variables : d and S_{fs}

Find $u_s = Q/(y_s B)$. Then by assuming d , find u_{*s} by using the Eq. (6). Now compute τ_{cs} using $\tau_{cs} = \rho u_{*s}^2$. Using the

Tab. 3. The Various Problems of Stable Channel Design

Type of Design Problem	Design Variables					Design solution hint (Assume to start with)
	d Particle Size	S_{fs} Channel Slope	y_s Flow depth	Q Water Discharge	V_s Seepage velocity	
I	✓	✓	✓	?	?	–
II	✓	✓	?	✓	?	y_s
III	✓	✓	?	?	✓	y_s
IV	✓	?	✓	✓	?	–
V	✓	?	✓	?	✓	τ_{cs}/τ_{co}
VI	✓	?	?	✓	✓	τ_{cs}/τ_{co}
VII	?	✓	✓	✓	?	–
VIII	?	✓	✓	?	✓	d
IX	?	✓	?	✓	✓	τ_{cs}/τ_{co} and y_s
X	?	?	✓	✓	✓	d

Note: The properties like γ , γ_s and ν are assumed to be known, and channel is assumed to be wide ($B \sim 1\text{m}$) in all the problems

Shields' curve, compute τ_{co} . Find τ_{bo} with Eq. (1). Find N by using its definition. Now refine τ_{cs}/τ_{co} with Eq. (8). A new value of d can be computed by using Shields' curve. Now find a new value of u_{*s} by using the Eq. (6). Find a new value of τ_{cs} by using $\tau_{cs} = \rho u_{*s}^2$. Repeat the procedure until the value of τ_{co} stabilizes. Finally find S_{fs} by using $\tau_{cs} = \gamma y_s S_{fs}$.

5.1 Evaluation of design problems

The evaluated design variables from the above ten design types are plotted against the experimental values as shown in Figs. 6 to 15. It is noticed that the predictability is satisfactory.

5.2 Validity of the design

The results of the ten types of design problems are verified with the experimental data as shown in Figs. 6 to 16. There is a little scatter observed in the design diagrams, because it is highly difficult to have 100% control over the seepage in the laboratory. Therefore, the results are well justified for the incipient motion with suction. It is of interest to discuss the results obtained from the 7th type of problem, plotted on Fig. 12. One may notice that there is a large scatter particularly in the range of d from about 0.06 mm to 1.38 mm which suggests that the average velocity u_s is not very sensitive to d , that is to say that u_s does not vary significantly with the variation in d in that range. Hence it is advised to compute u_s with the known value of d (like in the other problems) rather than finding d with the known value of u_s . One can notice such a flat nature of the curve plotted between the dimensionless average velocity $u' = (u_s d / \nu) / d_* = u_s / [g \nu ((\gamma_s / \gamma) - 1)]^{1/3}$ and d_* in Fig. 16, in the range of about d_* from 1.5 to 35 (corresponding to the range of sand particles of size $d = 0.06$ to 1.38 mm).

6 Conclusions

The presence of seepage is altering the rate of sediment transport characteristics of the sand-bed channels and hence it is con-

cluded that seepage effects should be considered in the channel design. Careful attention is given on the contradictory findings in the published literature and hence for clarification the experiments are carried out in a bigger scaled flume, i.e., on 1.80 m wide and 25 m long (with a seepage length of 20 m) tilted flume at Indian Institute of Science, Bangalore. In this experimental study it has been observed 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, which strengthens the earlier conclusions of Watters and Rao, Willetts and Drossos, Maclean and Rao and Sitaram. A new resistance equation has been developed which would be suitable for seepage affected sand-bed channels. A thorough design procedure for plane sediment bed subjected to seepage has been discussed and also its predictability has been demonstrated under different conditions.

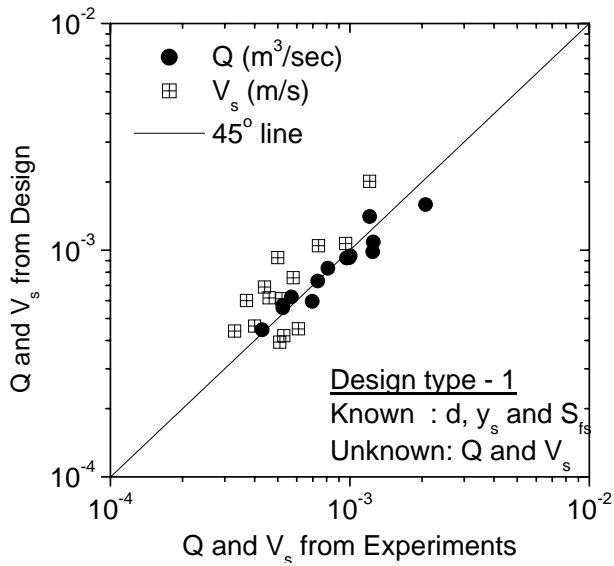


Fig. 6. Design type - 1

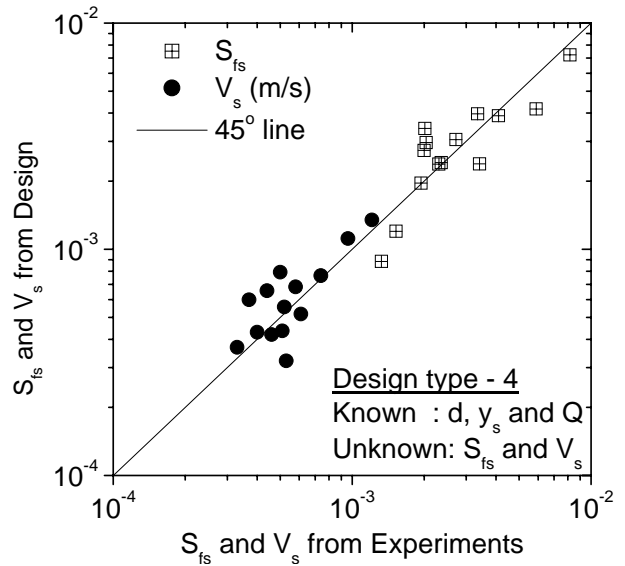


Fig. 9. Design type - 4

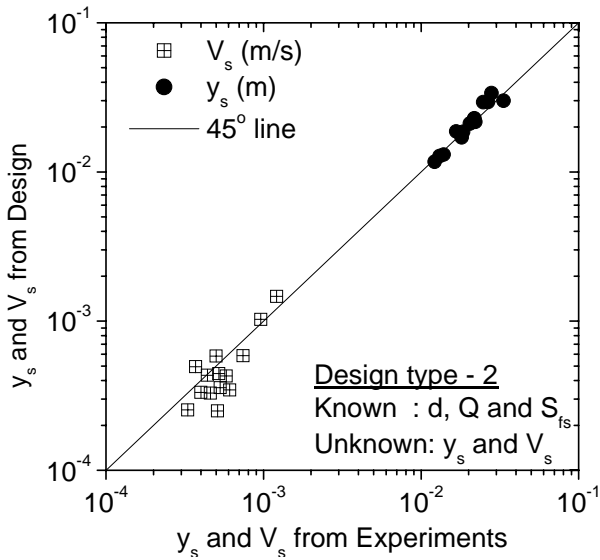


Fig. 7. Design type - 2

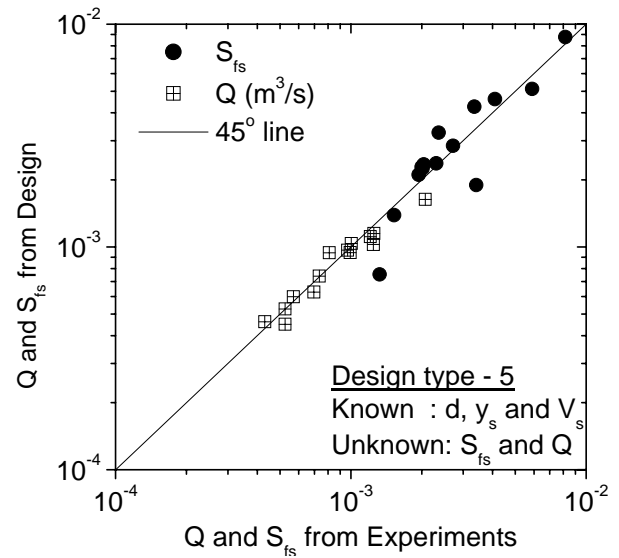


Fig. 10. Design type - 5

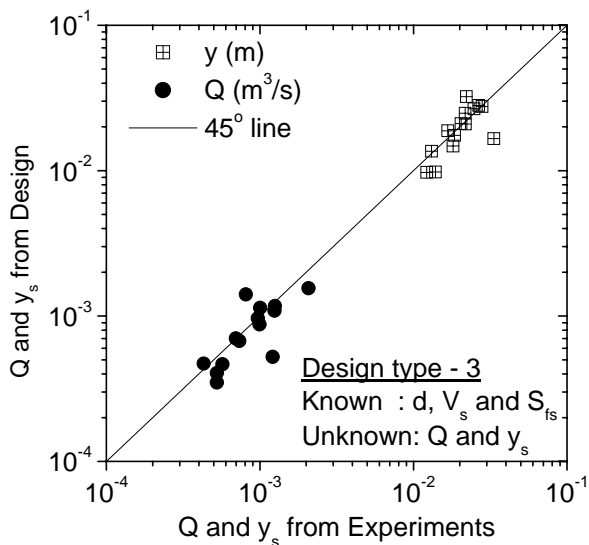


Fig. 8. Design type - 3

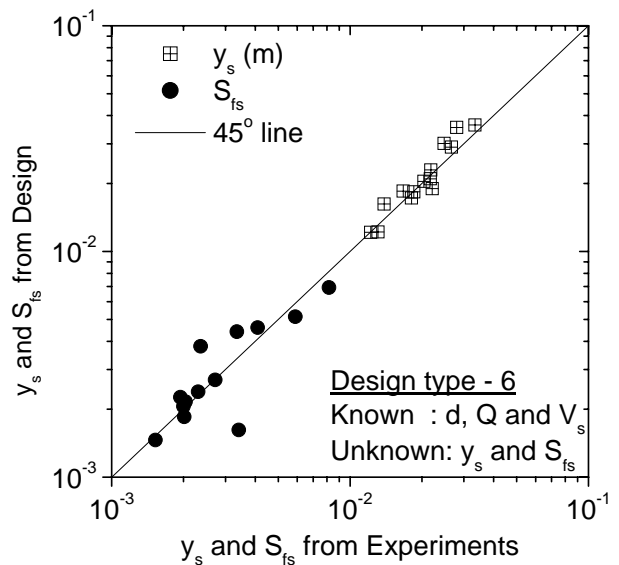


Fig. 11. Design type - 6

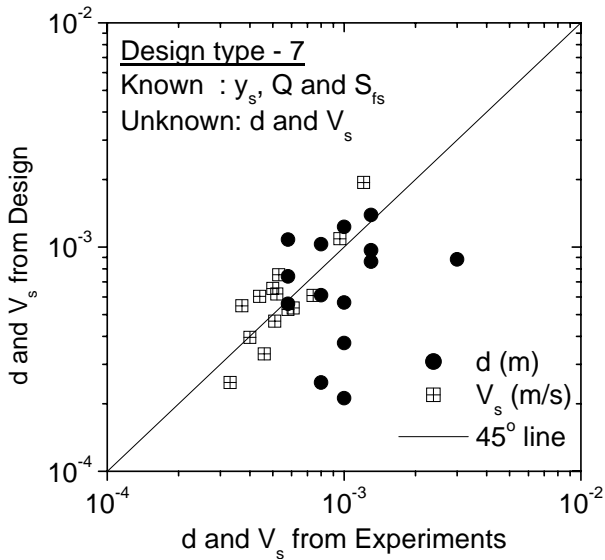


Fig. 12. Design type - 7

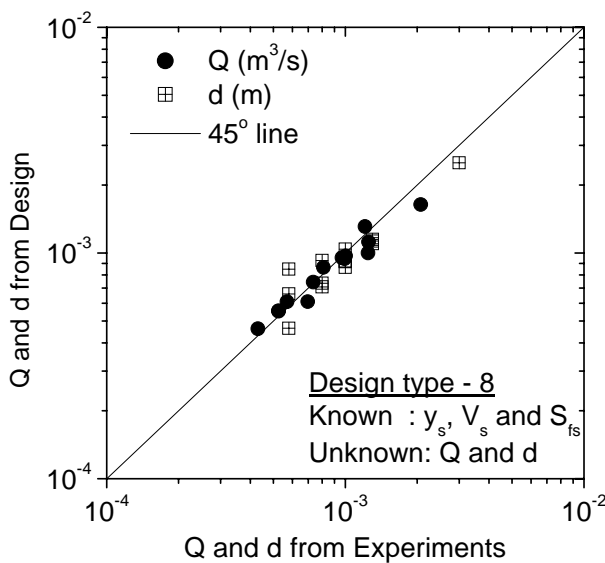


Fig. 13. Design type - 8

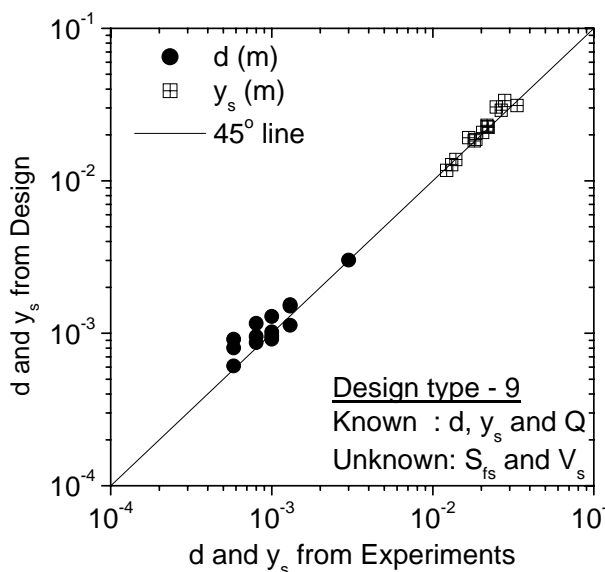


Fig. 14. Design type - 9

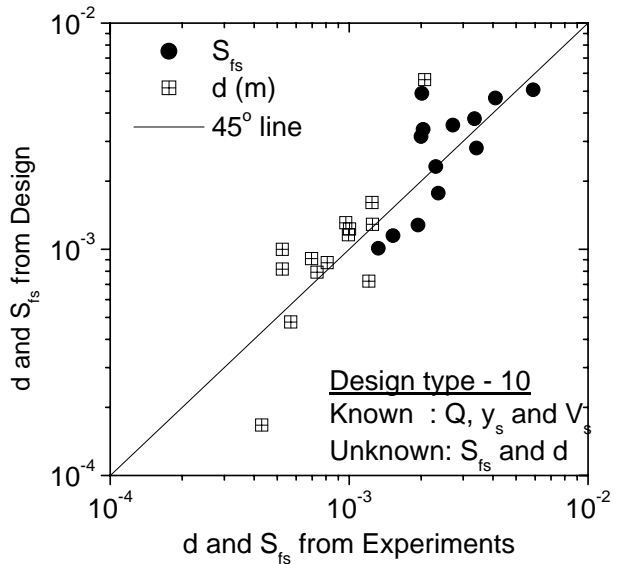


Fig. 15. Design type - 10

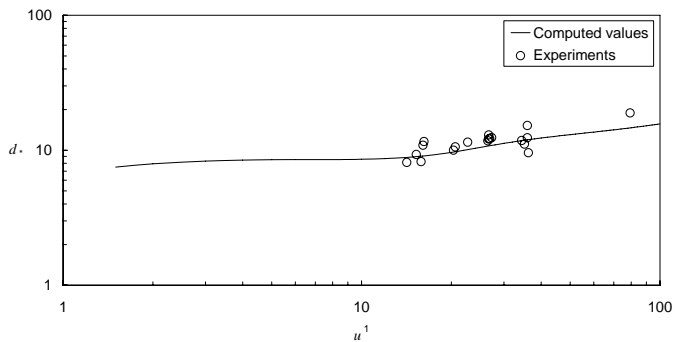


Fig. 16. Sensitivity of average velocity with particle size [solid line represents the values of d and generated from the Shields' curve and Eq. (8)]

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Notations

B	= width of the channel, (m)
d	= sediment particle size, (m)
d_{16}, d_{50}, d_{84}	= diameter of particles at 16%, 50% and 84% finer by weight, (m)
d_*	= dimensionless particle size $[gd^3(\gamma_s/\gamma - 1)/\nu^2]^{1/3}$
g	= 9.80665 m/s ² , acceleration due to gravity
n	= Manning's roughness coefficient, (s/m ^{1/3})
N	= Seepage intensity parameter
Q	= main discharge in the channel, (m ³ /s)
q_s	= seepage discharge over the length, (m ³ /s)
R	= $\bar{u}d/\nu$, particle Reynolds number
R_*	= u_*d/ν , shear Reynolds number
S_o	= bed slope of the channel
S_f, S_{fo}, S_{fs}	= energy/friction slope of the channel with and without seepage
S_w, S_{wo}, S_{ws}	= water surface slope of the channel with and without seepage
u, u_o, u_s	= average velocity in the channel with and without seepage, (m/s)
u_*	= critical shear velocity, (m/s)
u'	= dimensionless average velocity = $(\bar{u}d/\nu)/d_*$
V_s	= Seepage velocity in the channel, (m/s)
y	= average flow depth, (m)
y_o, y_s	= average flow depth with and without seepage, (m)
ud/ν	= Particle Reynolds number
f_1, f_2, f_3, f_4	= functions of R_* as given in Eqs. (2) to (7)
γ	= unit weight of fluid, (N/m ³)
γ_s	= unit weight of the sediment material, (N/m ³)
ν	= kinematic viscosity of fluid, (m ² /s)
ρ	= mass density of the fluid, (Kg/m ³)
σ	= gradation coefficient = $0.5(d_{84}/d_{50} + d_{50}/d_{16})$
τ_{bo}, τ_{bs}	= bed shear stress with or without seepage, (N/m ²)
τ_{co}	= critical shear stress, (N/m ²)
τ_{cs}	= critical shear stress under incipient motion with suction or injection, (N/m ²)
ω	= fall velocity, (m/s)
η	= Correction proposed by Oldenzil and Brink [12]