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Influence of cohesion on the incipient motion condition of sediment mixtures

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[1] Previous investigations on initiation of motion and erosion of cohesive sediment mixtures have not included gravel within the mixtures. Experimental results on the incipient motion characteristics of cohesive sediment mixtures containing gravel are presented in this paper. The sediment used consisted of fine gravel mixed with clay in proportions varying from 10% to 50% by weight, and fine gravel with fine sand mixed in equal proportion, with clay proportions again varying from 10% to 50% by weight. The stages of the incipient condition of motion were visually identified. Three modes of initiation of motion of cohesive sediment mixtures were noticed, namely, pothole erosion, line erosion, and mass erosion. The modes of initiation of motion changed mainly with clay percentages in the mixture, its antecedent moisture characteristics, and the applied shear stress. Analysis of the experimental data from the present study and the literature revealed that depending upon the flow and sediment characteristics, the critical shear stress of the incipient motion condition in cohesive sediment mixtures can be up to 50 times larger than the critical shear stress of cohesionless sediments having similar arithmetic mean size as the cohesive sediments. On the basis of dimensional considerations, a relationship is proposed for the determination of critical shear stress of the incipient motion condition in cohesive sediment mixtures also containing gravel. The variables, namely, clay percentage, void ratio, and unconfined compressive strength of the sediment bed, are noticed to be the main parameters controlling the incipient motion condition of the cohesive sediments. In the absence of cohesion, the proposed relationship reduces to the critical shear stress for the incipient motion condition of the cohesionless sediment mixture having the same arithmetic mean size. The condition for the threshold of erosion of cohesive sediment mixtures by the flow is thus parameterized in the present study.

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1. Introduction

[2] The role of sediment detachment and its transport on the evolution of river morphology is well known. An understanding of the detachment and transport of cohesive sediments is required to solve various engineering problems such as: mitigation of soil erosion in catchments, reservoir sedimentation, stable channel design, river bed degradation and the effects of fine sediment deposition within riverbed material on aquatic life, etc. Some aspects of cohesive sediment detachment are also significant for engineering projects such as: stream bank erosion, embankment stability, scouring around bridge piers, navigation and water quality issues etc. The condition for initiation of motion and process of detachment and transport of sediment by stream flows in the form of bed load and suspended load, for uniform and nonuniform cohesionless sediments are reasonably well understood [*Garde and Ranga Raju*, 2000]. Similarly, several studies are available on erosion and transport of consolidated and unconsolidated cohesive sediments such as clays [*Raudkivi*, 1990].

[3] However in nature, the catchment surface and river bed material consist of mixtures of cohesive as well as cohesionless material. Soil in upland catchment areas is one of the examples of this type of sediment. For instance, fan delta bed slopes of Britannia beach, British Columbia dominantly consist of mixtures of clay and gravel [*Prior and Bornhold*, 1986]. The clay-gravel mixtures are commonly used as surface protective lining of earthen embankments that are subjected to erosive action of overland flow as well as rainfall [*Kumar and Wood*, 1999]. The clay-sand-gravel mixtures however naturally occur in several geologies [*van Rijn*, 2007]. The condition for initiation of motion and detachment of sediment mixtures consisting of coarser cohesionless sediment like gravel and cohesive material (clay) is a topic yet to be investigated by the researchers. The present investigation therefore proposes to examine the influence of the presence of cohesive material (i.e., clay minerals and biological matter which comprises natural mud) on the initiation of motion of cohesionless sediment consisting of fine gravel and sand size particles.

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[4] For cohesionless sediments the main resistance to erosion is provided by the submerged weight of sediment. However, in cohesive sediment beds the net attractive inter particle surface forces, frictional interlocking of grain aggregate and electrochemical forces control the resistance to erosion. These forces vary with the type and proportion of clay, antecedent moisture conditions, type of shear application and drainage conditions. As a result these forces are not yet understood completely. As reported in more recent literature the cohesive sediments consist principally of mineral particles and organic debris as well as microorganism and their secretions [*Ravisangar et al.*, 2005]. They are characterized by the tendency of the particles to stick together because of physicochemical forces [*Ansari et al.*, 2003]. The biological factors also play an important role in determining cohesion in natural sediments [*Black et al.*, 2002]. Cohesion also leads to particle aggregation (i.e., flocculation) when interparticle collision occurs [*McAnally and Mehta*, 2000]; Brownian motion, local velocity gradients, difference in settling velocities of different sized particles can result in particle collisions. As a matter of fact a wide range of aggregates of various sizes, strength and densities can be found both in suspension and in the deposits of cohesive sediments [*Ravisangar et al.*, 2005].

[5] *Righetti and Lucarelli* [2007] have attempted the parameterization of the threshold for natural cohesive-adhesive sediment motion. On the basis of a rational approach, a new formulation for the incipient motion condition of cohesive-adhesive sediments is proposed by them, which can be seen as an extension of the Shields's criterion for incipient motion condition of cohesionless sediments. Although cohesive sediment dynamics is important for many engineering and ecological applications, its general theory is still unavailable [*Black et al.*, 2002].

[6] The main mechanisms which cause sediment to be moved in flowing water are the velocity of flow, shear and normal stress resulting from turbulent flow [*Garde and Ranga Raju*, 2000]. When the hydrodynamic force in a turbulent flow exceeds the resisting force of a cohesive sediment bed, the sediment starts getting detached from the bed and the flowing water becomes turbid. This stage, which is characterized by the initiation of erosion, is described as the incipient motion condition of cohesive sediment. With further increases in shear stress more sediment is detached from the bed and transported by the flow. In the present study an attempt is made to parameterize the condition for threshold of motion by the flow of cohesive sediment mixtures that also contain gravel.

2. Concept Review

[7] The concept of critical bed shear stress has a vital position in the theory of sediment transport. Numerous investigations related to critical shear stress and transport rates of uniform and nonuniform cohesionless sediments are available in the literature. However, the methods are not available as yet for the determination of critical shear stress of nonuniform cohesive sediments consisting of particles of clay as well as gravel. A few investigators have however, studied through the use of laboratory based experimental data, the variation of critical shear stress of cohesive sediments formed by clay-sand mixtures that were having relatively uniform size distribution. These are reviewed here briefly.

[8] *Fortier and Scobey* [1926] were the earlier investigators who studied the incipient motion of cohesive sediments. For finer particles in the silt and clay size range, the critical velocity was noticed to increase as the mean particle size of the mixture decreased. *Dunn* [1959] proposed a method for estimating tractive resistance of cohesive channel beds. He found the vane shear strength, plasticity index and soil fines, to affect the tractive resistance of cohesive channel beds. *Smerdon and Beasley* [1959] correlated the critical shear stress for soil formed of silty loam and clay to the plasticity index, percent clay and other sediment characteristics. *Laflen and Beasley* [1960] studied the effect of sediment compaction on critical shear stress in cohesive sediments having clay percentages varying from 14% to 61.2%. They noticed a linear relationship between the void ratio and the critical shear stress. *Thomas and Enger* [1961] conducted experiments to determine critical shear resistance of cohesive sediment samples from the canals of western United States using gross characteristics of the sediment bed. *Flaxman* [1963] proposed a graphical relationship between the stream power and unconfined compressive strength of cohesive bed sediments. He found a sharp separation between stable and eroding sediments. *Lyle and Smerdon* [1965] extended the study of *Smerdon and Beasley* [1959] by using sediment from Texas soils and presented empirical relations for the determination of critical shear stress using void ratio and other sediment properties like plasticity index, dispersion ratio, vane shear strength and percent clay.

[9] *Kamphuis and Hall* [1983] also experimentally studied the effect of variations of clay content and consolidation pressure, on the critical shear stress required to initiate erosion of cohesive sediment under unidirectional flow. The samples were consolidated in a specially designed press (600 mm long, 150 mm wide and 100 mm thick). Various mixtures of clay-silt-sand were tested in which clay percentage was varied from 15% to 62%. The velocity 3 mm above the bed surface was taken as critical velocity when erosion became apparent. *Raudkivi* [1990] has presented a review on the work of various investigators on critical shear stress of cohesive sediments. He concluded that particle diameter, the proportion of clay, dispersion ratio, as well as plasticity index are the important sediment properties affecting the critical shear stress. *Mirstkhoulava* [1991] suggested an expression for permissible nonscouring mean velocity, for cohesive sediments in plane turbulent flows. *Mirstkhoulava* [1991] also proposed a relationship between scouring velocities of cohesive sediment and their physicochemical properties. *Ghebreiyessus et al.* [1994] studied the detachment by concentrated flow of silt loam (mixture of sand-silt-clay) compacted to two different bulk densities. They defined the critical shear stress of cohesive sediments as the shear stress corresponding to zero rate of detachment. The critical shear stress was noticed by *Ghebreiyessus et al.* [1994] to rise with an increase of the bulk density of sediment bed. *Samad et al.* [1995] conducted experiments to determine the critical shear stress for silts and clay samples collected from upper end of Elephant Butte reservoir in the USA. Flume test and rotating cylinder test were performed to determine critical shear stress and observa-

tions obtained from both the methods were found to be in agreement with *Smerdon and Beasley*'s [1959] relation. *Mitchner and Torfs* [1996] found the critical bed shear stress for erosion initiation of the sediment bed consisting of sand and cohesive mud, depends on grain size of sand and cohesive properties of the mud. The critical shear stress for erosion increased when mud was added to sand in varying percentages from 3% to 50%. They also reported that even the addition of up to 5% mud to sand can increase the critical erosion shear stress of the sand by a factor of two. *Panagiotopoulos et al.* [1997] presented experimental results on the influence of clay on the threshold for movement of fine sandy beds under unidirectional and oscillatory flows. The experiments were conducted on a mixture of Combwich mud and sand. The mud was added to sand in varying percentages ranging from 5% to 50% by weight. It was concluded that with the addition of clay, sediment bulk density is reduced and critical bed shear stress required to displace fine grained sand by unidirectional flows is increased by the order of up to 90%.

[10] The critical shear stress for cohesive sediments has also been considered as the function of erosion resistance and has been highlighted in several studies. *Hanson* [1990] measured erodibility of four types compacted soils in the channel under high range of flow shear stress. The soils in channel were compacted in layers of 8 cm or 13 cm by a sheep foot roller and edges in channel were hand tamped where roller was ineffective. *Hanson* [1990] found that the soil having highest sand content and lowest clay content has more erodibility compared to soil with highest clay content and lowest sand content.

[11] *Robinson and Hanson* [1995] performed experiments on headcut movement in a flume 1.8 m wide, 29.0 m in length, and 2.4 m high. The sediment was filled in layers of 152 mm or 203 mm thickness and compacted by a self-propelled vibratory pad-foot roller. The headcut advance rate was found to decrease with an increase in average dry density and unconfined compressive strength of sediment bed. *Jepsen et al.* [1997] and *Roberts et al.* [1998] found that the erosion resistance for homogeneous bed increases with an increase in the sediment bulk density. *Amos et al.* [1997] and *Torfs* [1997] however reported for sediment beds composed of larger size particles the erosion resistance is independent of its bulk density and is a function of the grain size only. Thus for sediments composed of mud-sand mixture the bed material also plays an important role in addition to the bulk density of sediment [*Mitchner and Torfs*, 1996; *Houwing*, 1999]. *Zreik et al.* [1998] conducted erosion tests on Boston blue clay beds deposited from suspension in distilled water in rotating annular flume. Age and structure of the clay were found to strongly affect the resistance to bed erosion. The resistance to erosion at a given depth and for a given bed structure increased with the increasing age of the bed sediment. *Krone* [1999] on the basis of analysis of the data of *Roberts et al.* [1998] and *Zreik et al.* [1998] illustrated the effects of change in the structure of fine sediments on their erosion rates. The resistance to erosion was noticed to increase as the sediment bed structure became denser due to an increase in density of the bed material. *Krone* [1999] also noticed that the resistance of a cohesive sediment bed to erosion increases linearly with an increase in the depth of sediment or the overburden.

[12] *Sanford and Maa* [2001] presented the criteria to define the critical shear stress as (1) it is the shear stress at which initiation of motion first occurs and (2) the shear stress corresponding to zero erosion rate obtained from a backward extrapolation of erosion rate versus shear stress. They noticed that the critical shear stress of sediment bed surface is weak and it increases with the depth of sediment bed. *Hasson et al.* [2004] observed that an increase in compactive effort and water content of the given cohesive sediment increases the erosion resistance of that sediment significantly. This observation points out the importance of soil properties in determining the detachment condition for cohesive embankment erosion. *Lick et al.* [2004] proposed a relationship to compute the critical shear stress of quartz particle that depends on bulk density and particle size. *Van Ledden et al.* [2004] studied the transition between non-cohesive and cohesive erosion behavior in sediments. The transition from cohesionless to cohesive behavior when clay content exceed 5–10% in natural sediment beds was reported by them based on experimental data.

[13] *Hanson and Hunt* [2006] conducted the jet erosion test to measure effect of compaction on soil erodibility. The soils were compacted in various lifts of 0.12 m using a self-propelled vibratory pad-foot roller. The erosion resistance in their experiments increased with increase in dry density but at a faster rate on the dry side of the optimum moisture content than on wet side of optimum moisture content. *Kothyari et al.* [2006] have proposed a relationship to compute the critical shear stress of cohesive sediments consisting of clay-sand mixtures, as a function of plasticity index, antecedent moisture content, moisture content at saturation and void ratio by utilizing the data of *Ansari* [1999] and *Laflen and Beasley* [1960]. *Julian and Torres* [2006] by using the data of *Dunn* [1959] have indicated the critical shear stress to be a function of silt-clay percentage. A third-order polynomial relation was proposed by them for critical shear stress based on the assumption that at 100% silt-clay, critical shear stress would reach a maximum value, and at 0% silt-clay, it takes a minimum value.

[14] The review presented above also indicated that no investigation is available as yet for quantification of the influence of cohesion on critical shear stress of nonuniform sediments having gravel and clay sizes present together in their mixture. The review also indicated that clay content in laboratory studies for critical shear stress of clay-silt-sand mixtures was varied by most of the researchers from 5% to about 50% or more. However, a significant increase in the value of the critical shear of cohesive sediment is also reported in literature, even while the clay percentage in them is small, i.e., less than 5%.

3. Experimental Program

[15] The experimental program was undertaken to study the influence of cohesion on initiation of motion and process of detachment of cohesive sediment mixtures also containing gravel. These experiments were conducted in the Hydraulic Engineering Laboratory of Department of Civil Engineering, Indian Institute of Technology Roorkee, India. Details are given below.

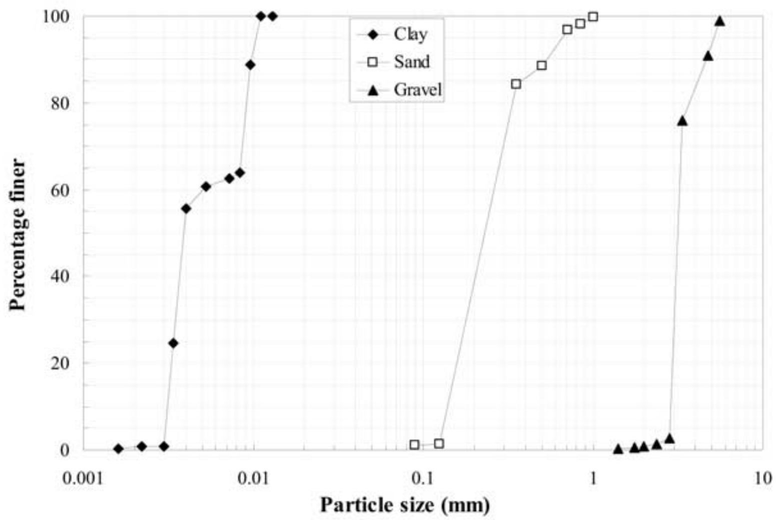

Figure 1. Grain size distribution of the sediment used in the present study.

3.1. Properties of the Cohesive Sediments

[16] Cohesionless sediments consisting of fine sand and fine gravel were used as the base sediment. Clay was added in various proportions to the base sediment to create the cohesive sediments. A wide range of field conditions were simulated by varying the clay content, antecedent moisture content and dry density of thus formed cohesive sediments. Laboratory tests were also conducted to determine various engineering properties of clay, sand and gravel and their mixtures as described in the following sections.

3.2. Properties of Clay-Sand and Gravel

[17] Locally available clay excavated from a depth of 2.0 m below the ground was used as cohesive material. Tests for determination of clay properties were conducted as per Indian standard (IS) code [*Bureau of Indian Standards (IS)*, 1970]. Laser particle size analyzer was used to obtain particle size distribution curve for clay. The clay material had a median size d_{50} equal to 0.0039 mm, geometric standard deviation σ_g equal to 1.7. The σ_g is determined as $\sqrt{d_{84}/d_{16}}$ [*Garde and Ranga Raju*, 2000]. Here d_{84} is the sediment size such that 84% of bed material is finer than this size by weight, and d_{16} is the sediment size such that 16% of bed material is finer than this size by weight. Sand had a median size d_{50} of 0.23 mm and σ_g of 1.53, while gravel had median size d_{50} of 3.1 mm and σ_g of 1.23. The relative density of sand and gravel was 2.65. Figure 1 shows the grain size distributions of clay, sand, and gravel used in the present study. The other engineering properties of clay material were: liquid limit W_L = 38%, plastic limit W_P = 24%, plasticity index PI = 14%, maximum dry density $(\gamma_d)_{\max}$ = 18.27 KN/m^3, optimum moisture content OMC = 16%, cohesion at OMC, C_u = 54 KN/m^2 and angle of internal friction at OMC, $\Phi_c = 21°$ and relative density = 2.6. As per IS-1498, the clay was classified as *CI*, i.e., clay with intermediate compressibility.

[18] The X-ray diffraction test was conducted to determine the composition of various minerals present in the clay. The slides of clay prepared were ordinary slide, heated slide and glycolated slide and these slides were tested under a specific set of testing conditions in X-ray diffraction apparatus. The percentage composition of various clay minerals were found as per *Klages and Hopper* [1985] was Kaolinite = 17.6%, Illite = 60.3%, Vermiculite = 15.3% and Chlorite = 6.8%.

3.3. Properties of Clay-Gravel and Clay-Sand-Gravel Mixtures

[19] A significant increase in the critical shear stress of the cohesive sediments even with small clay percentage (less than 5%) has been observed by the previous investigators. In engineering applications however, the cohesive properties are considered as important while the clay fraction in the sediment mixture is larger than about 5–10% [*van Ledden et al.*, 2004; *van Rijn*, 2007]. The cohesive sediments were therefore prepared by mixing clay materials in proportions varying from 10% to 50% with fine gravel and with fine sand–fine gravel mixtures (each in equal proportion by weight). This is also consistent with the previous experimental studies of cohesive sediment erosion by several researchers who varied the clay content from 5% to about 50% or more in clay-silt-sand mixtures [*Laflen and Beasley*, 1960; *Kamphuis and Hall*, 1983; *Hanson*, 1990; *Robinson and Hanson*, 1995; *Panagiotopoulos et al.*, 1997; *Dey and Westrich*, 2003; *Mazurek et al.*, 2003; *Wan and Fell*, 2004]. The amount of moisture content antecedently present in cohesive sediment has great influence on physical properties of the cohesive sediments [*Ansari et al.*, 2002]. Depending upon the moisture content present, the cohesive sediments change their stages, i.e., liquid, plastic and nonplastic (semisolid). In the present investigation, the tests were conducted under maximum possible range of antecedent moisture content so as to represent their different stages which occur in field conditions. Therefore cohesive sediments were tested for the determination of their incipient shear stress at several initial moisture consistencies ranging from very soft soil with negligible cohesion (viscous state), to hard soil with a high value of cohesion.

3.4. Experimental Flume

[20] The experiments were conducted in a tilting flume 16 m long, 0.75 m wide and 0.5 m deep. The channel had a test section of 6.0 m length, 0.75 m wide and 0.13 m depth

Figure 2. Specially fabricated roller to fit channel width used for compacting the bed material.

starting at a distance of 8.0 m from channel entrance. Observations were made at various slopes of flume ranging from 2.417×10^{-3} to 9.5×10^{-3}. The discharge in the flume was provided by a constant head overhead tank. The discharge flowing in to the channel was volumetrically measured with the help of another tank provided at the downstream end of the channel. The water supply into the flume was regulated with the help of a valve provided in the inlet pipe coming from the overhead tank. The sediment was filled in the test section up to the level of the general flume bed. A thin layer of sediment being filled in the working section was also pasted uniformly on flume bed on the upstream and downstream of the test section to simulate the roughness of the test section on the rest of the flume bed.

3.5. Preparation of Channel Bed

[21] The sun-dried powdered clay-sand and gravel were used for the preparation of the sediment. The moisture present at the stage of mixing in clay powder, sand and gravel was determined by measuring change in weight since drying of the sediments. Thus the amount of moisture needed to be mixed to obtain the desired consistency of cohesive sediment was ascertained. Accurately weighed clay powder, sand, gravel and the predetermined moisture (i.e., water) were mixed thoroughly. The mixed sediments were covered with polythene and left for 24 hours in order to achieve a uniform distribution of the moisture. The sediment was mixed thoroughly again before transporting it to the channel test section. Thus prepared cohesive sediments were filled in the channel test section and compacted either by a dynamic compaction method or a kneading method depending upon the moisture content of the cohesive sediment prior to compaction in similar technique as previously reported by *Hanson* [1990], *Robinson and Hanson* [1995], and *Hanson and Hunt* [2006].

[22] The dynamic compaction method was adopted for cohesive sediments of hard, semisolid and plastic consistencies. In dynamic compaction method the sediment was compacted in channel test section in three different layers each having thickness of 0.04 m. Each layer was compacted with a cylindrical roller of dimension 0.23 m diameter and 0.63 m length (see Figure 2) specially fabricated to fit into the channel width. The dead weight of roller was equal to 100 N. Depending upon the required amount of compaction, the arrangement was also made to increase the compaction weight up to 200 N by filling water inside the roller body.

[23] Depending upon the required value of dry density of sediments the number of passes of roller over the test section in each layer was decided. The sides of sediment bed were compacted by a 100 N weight hand rammer having 0.10 m base diameter. To ensure bonding among the different layers of the cohesive sediments the top surface of each layer was roughened before laying the next layer over it. The channel test section was slightly overfilled with sediments. The extra sediment was trimmed off by using a sharp edged large knife. The density of compacted sediment and antecedent moisture content was measured at three different locations of the channel test section to ensure the uniformity of compaction and placement. The observed value of densities and moisture content were similar at all locations of the channel test section.

[24] The kneading method of compaction was used for sediments of soft consistencies. Such sediments were dropped from a suitable height into the channel test section in the form of small heaps. The heaped sediments were first spread uniformly along the test section and then compacted by a wooden rammer having flat base to ensure the uniformity of compaction throughout the test section. The channel test section was overfilled with sediments in this case too and extra sediment was trimmed off as explained previously. Prior to experimentation the prepared bed was saturated for 24 hours. Figure 3 depicts a freshly laid cohesive sediment bed. The values of dry density and antecedent moisture content reported in the paper are measured at the completion stage of dynamic compaction or kneading.

3.6. Measurements

[25] The unconfined compressive strength (*UCS*) of the sediments was measured by using laboratory based unconfined compression test apparatus. The *UCS* is a quick test to obtain the shear strength parameter of cohesive sediments. Therefore it is used in all geotechnical designs as the strength parameter of cohesive sediment [*Terzaghi et al.*,

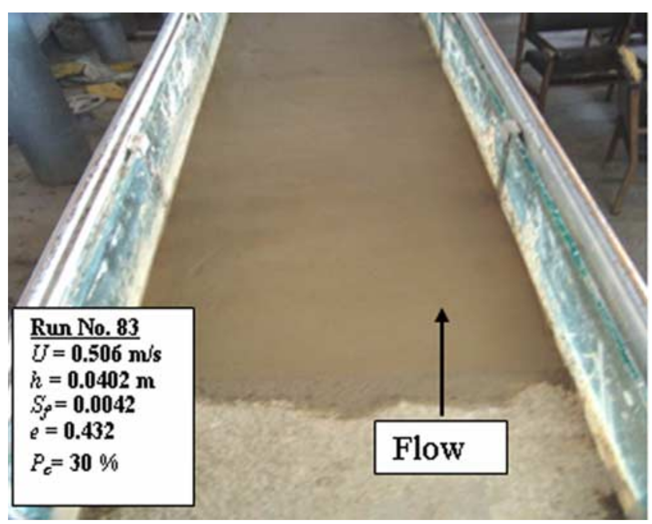

Figure 3. Sediment bed surface before conduct of the run for incipient motion condition.

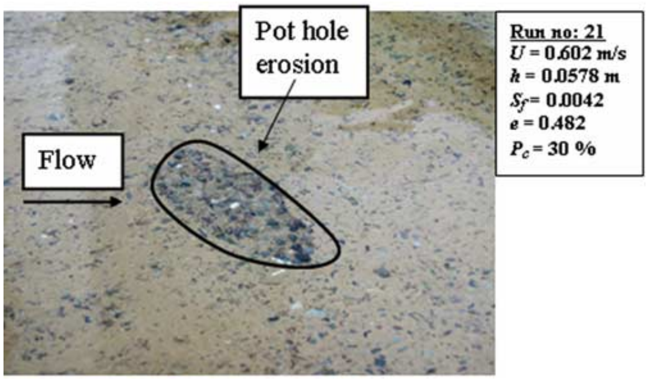

Figure 4. Sediment bed surface showing a pothole at the condition of incipient motion.

1996]. For the determination of *UCS*, cylindrical specimens were taken from the compacted bed and were tested as per IS-2720 Part X [*IS*, 1991] in unconfined compression apparatus. The bulk unit weight of sediment was determined as per IS-2720 Part XXIX [*IS*, 1975] by using standard core cutter method. The value of dry density was computed using the observed value of bulk density and antecedent moisture content. The void ratio of cohesive sediments was derived from the computed value of dry density.

3.7. Incipient Motion of Cohesive Sediments

[26] For each test, initially a small discharge was admitted into the flume and uniform flow was established by simultaneously operating the tail gate and the bed condition was carefully observed. The discharge was increased next in small increments by operating the inlet valve while uniform flow condition was maintained through simultaneous operation of the tail gate and monitoring of the bed condition was continued until the condition of incipient motion developed. Three types of conditions of incipient motion; each will be described as were observed during condition of incipient motion.

[27] 1. The first is pothole erosion. At the lower percentage of clay in bed material (up to 30%), the detachment generally initiates in the form of individual particles detached from bed leaving formation of potholes over the bed surface at different locations. These potholes were 4 to 5 cm of length and a few cm wide (see Figure 4) and their size increased as time progressed.

[28] 2. The second is line erosion. At the higher percentages of clay (greater than 30%) in bed material, the detachment begins by removing thin flakes from the sediment surface and that removal continues and spreads both longitudinally and laterally, but more in the longitudinal direction. These lines more often appeared on the upstream portion of the channel test section, though they were also noticed to occasionally occur in other part of the test section. The lines of erosion were 2 to 3 cm in width and much longer lengthwise, sometimes even 30 to 40 cm long (see Figure 5).

[29] 3. The third is mass erosion. This type of condition was frequently observed in clay-sand-gravel mixtures particularly for higher values of energy slope and for sediment bed having clay percentages of more than 30%. Here the detachment was noticed to occur in the form of chunks of varying sizes. These chunks were of sufficient thickness, 1 to 2 cm in length and few millimeters wide. The pieces of sediment appeared to be ripped or torn off from the bed surface. At the end of the run, a number of potholes were appearing on the bed surface as shown by Figure 6.

[30] Flow parameters corresponding to the condition of incipient motion in the sediment bed were noted. A total of 108 experimental runs were conducted on critical shear stress of cohesive sediment mixtures. Out of which more than half the runs (62) corresponded to incipient motion condition for cohesive sediments consisting of clay and gravel mixtures, while the remaining 46 runs were taken corresponding to incipient motion condition for cohesive sediments consisting of clay, sand and gravel mixtures. Tables 1 and 2 give the complete listing of the data collected in the present study. In addition, available data were also collected from literature from the publications of *Ansari* [1999], *Kamphuis and Hall* [1983], *Peirce et al.* [1970], *Lyle and Smerdon* [1965], *Smerdon and Beasley* [1961], and *Laflen and Beasley* [1960].

4. Data Analysis, Results, and Discussion

[31] The detachment of cohesive sediment is controlled not only by the macroscopic physical properties of sediment such as its size and density, but also by the interparticle bond strength and sediment fabric characteristics [*Mazurek et al.*, 2001]. Thus the erosion process of cohesive sediments depends both on the complex mechanical characteristics such as shear stress and shear strength, and the physicochemical properties of the cohesive sediments. Amount and type of clay, antecedent moisture content, bulk density, *UCS etc.* are therefore considered herein to be the easily measurable variables representing the factors controlling the erosion of cohesive sediments. The effect of these variables on incipient motion condition of cohesive sediments therefore, is studied with the help of experimental observations.

[32] For cohesionless sediments the critical shear stress can be determined sufficiently accurately by using one of the versions of Shields's function from knowledge of grain density, size and gradation and fluid properties [*Cao et al.*, 2006; *van Rijn*, 2007]. However, the critical shear stress of the graded cohesionless sediments is still a topic under active research [*Dong*, 2007; *de Linares and Belleudy*, 2007]. Nevertheless, the recently published literature on determination of the critical shear stress of cohesionless sediment mixtures considers the Shields's criterion as the base model for these studies [*Wilcock*, 1993; *Dong*, 2007; *de Linares and Belleudy*, 2007]. To quantify the behavior of cohesive sediments in comparison to cohesionless sediments of similar bulk characteristics, Figure 7 is prepared, in which the observed values of critical shear stress of cohesive sediments τ_{cc} are plotted along with the arithmetic mean size of the corresponding cohesive sediment mixtures. The Shields's curve is superimposed on Figure 7 to compare the τ_{cc} values with the critical shear stress values of cohesionless sediment having size equal to the arithmetic mean size of the cohesive sediment mixtures. Almost all observed τ_{cc} values fall much above than the Shields's line indicating that for the given value of particle size, the critical shear stress of cohesive sediments is much larger (up to 50 times) than that of the same sized cohesionless

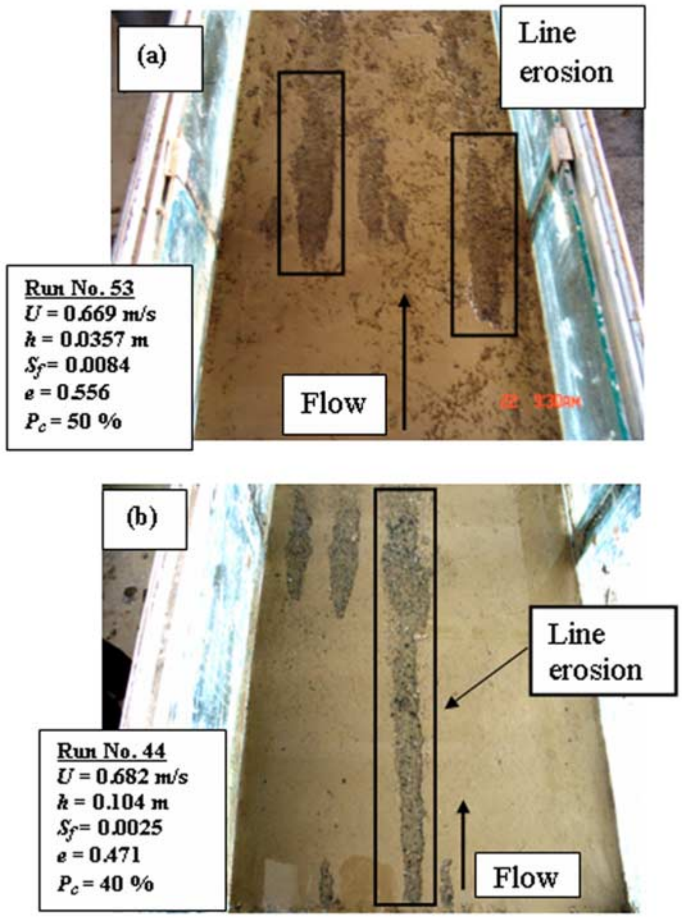

Figure 5. Bed surface showing initiation of detachment in the form of lines for (a) run 53 and (b) run 44.

sediment. Also increasing value of τ_{cc} with a reduction in particle size is followed for all the data including those from the present study. These findings are similar to those of *Raudkivi* [1990] and *Righetti and Lucarelli* [2007] for cohesive sediment mixtures without gravel. Some data acquired by *Ansari* [1999] fell below the Shields's line in Figure 7 and these had antecedent moisture content higher than the liquid limit.

5. Dimensional Consideration and Relationship for Incipient Motion of Cohesive Sediments

[33] A large number of interdependent variables influence the variation in shear stress for the initiation of motion conditions of cohesive sediment mixtures. It is thus difficult to investigate such variations by analytical methods, therefore dimensional analysis is used to study these variations. The critical shear stress for cohesive sediment mixtures is considered to be mainly affected by the following:

$$\tau_{cc} = f(P_c, e, d_a, \Delta\gamma_s, UCS, \tau_c). \tag{1}$$

Here P_c is the clay percentage, e is void ratio, $\Delta\gamma_s$ is the difference in the specific weight of sediment and fluids, UCS is the unconfined compressive strength of cohesive sediment bed, d_a is the arithmetic mean size of the sediment mixture and τ_c is the Shields's critical shear stress for the cohesionless sediment having size equal to the arithmetic mean size of the cohesive sediment mixture. The clay percentage of sediment bed is preferred as a parameter in equation (1) compared to the plasticity index used by the previous investigators like *Dunn* [1959] and *Kothyari et al.* [2006]. For the sediment mixtures studied presently, to

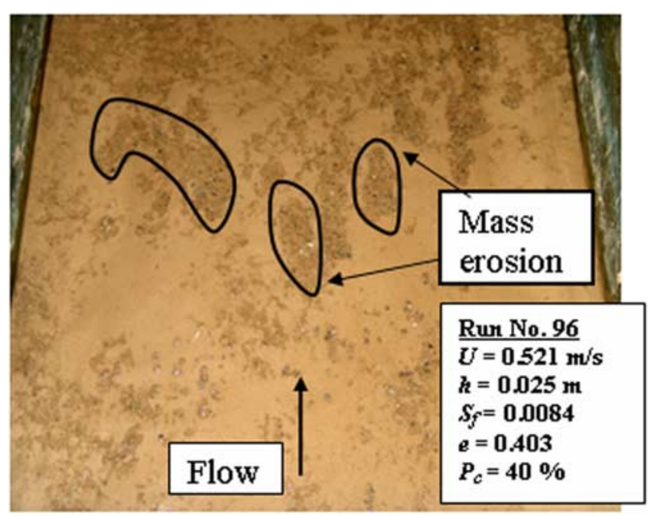

Figure 6. Bed surface showing initiation of detachment in the form of mass erosion.

Table 1. Sediment and Hydraulic Parameters for Incipient Motion Condition of Cohesive Sediments Formed by Clay-Gravel Mixtures[a]

Run	P_c, %	d_a, mm	W, %	γ_d, KN/m^3	UCS, KN/m^2	e	h, m	U, m/s	S_f	τ_{cc}, N/m^2
1	10	2.790	8.92	16.59	0.00	0.564	0.1171	0.680	0.0024	2.378
2	10	2.790	10.42	17.19	0.00	0.509	0.1100	0.655	0.0024	2.247
3	10	2.790	4.88	16.00	0.00	0.622	0.0317	0.526	0.0085	2.219
4	10	2.790	7.23	17.98	0.00	0.443	0.0354	0.584	0.0085	2.390
5	20	2.480	6.64	17.64	3.12	0.468	0.0722	0.631	0.0037	2.302
6	20	2.480	7.54	17.55	3.74	0.476	0.0743	0.620	0.0036	2.227
7	20	2.480	13.03	17.31	0.00	0.496	0.0623	0.590	0.0038	2.095
8	20	2.480	11.51	18.96	5.20	0.366	0.0630	0.636	0.0045	2.446
9	20	2.480	8.13	18.54	5.00	0.397	0.0642	0.632	0.0043	2.396
10	20	2.480	7.49	17.23	0.00	0.503	0.1134	0.640	0.0024	2.110
11	20	2.480	13.91	17.82	0.00	0.453	0.1210	0.702	0.0024	2.424
12	20	2.480	15.38	17.47	0.00	0.482	0.1050	0.644	0.0025	2.152
13	20	2.480	8.01	18.75	6.38	0.381	0.0427	0.617	0.0073	2.633
14	20	2.480	11.78	18.39	1.26	0.408	0.0397	0.568	0.0072	2.322
15	20	2.480	13.38	17.72	0.00	0.462	0.0355	0.530	0.0080	2.149
16	30	2.171	8.27	19.27	13.86	0.341	0.0725	0.67	0.0044	2.543
17	30	2.171	8.94	18.97	9.11	0.363	0.0652	0.656	0.0044	2.464
18	30	2.171	11.13	17.17	1.25	0.505	0.0547	0.598	0.0043	2.132
19	30	2.171	15.00	17.35	0.00	0.490	0.0526	0.588	0.0049	2.148
20	30	2.171	12.55	18.28	3.12	0.414	0.0625	0.6393	0.0042	2.343
21	30	2.171	14.50	17.44	0.00	0.482	0.0578	0.602	0.0042	2.141
22	30	2.171	11.07	19.87	20.36	0.301	0.1357	0.793	0.0026	2.871
23	30	2.171	12.57	18.47	6.15	0.400	0.1336	0.745	0.0023	2.535
24	30	2.171	16.23	16.82	0.00	0.537	0.1210	0.705	0.0023	2.334
25	30	2.171	17.65	17.55	0.00	0.473	0.1185	0.702	0.0024	2.344
26	30	2.171	16.75	17.97	0.00	0.438	0.1053	0.648	0.0025	2.100
27	30	2.171	12.14	18.38	4.25	0.406	0.1316	0.747	0.0024	2.573
28	30	2.171	19.81	16.75	0.00	0.543	0.1028	0.642	0.0024	2.050
29	30	2.171	17.85	17.25	0.00	0.499	0.1088	0.673	0.0025	2.223
30	30	2.171	9.83	19.12	16.25	0.352	0.0420	0.65	0.0082	2.839
31	30	2.171	11.00	18.56	12.27	0.393	0.0402	0.601	0.0078	2.493
32	30	2.171	11.27	19.1	21.38	0.353	0.0431	0.708	0.0080	3.208
33	30	2.171	15.67	17.87	0.00	0.447	0.0285	0.542	0.0080	2.149
34	40	1.861	11.50	17.33	13.61	0.489	0.0636	0.653	0.0043	2.341
35	40	1.861	12.48	17.02	11.24	0.516	0.0598	0.653	0.0043	2.341
36	40	1.861	13.19	16.97	12.00	0.520	0.0565	0.607	0.0045	2.121
37	40	1.861	13.19	18.29	19.53	0.411	0.0675	0.675	0.0042	2.446
38	40	1.861	16.80	16.85	0.00	0.531	0.0605	0.652	0.0043	2.336
39	40	1.861	19.60	16.31	0.00	0.582	0.0565	0.620	0.0043	2.166
40	40	1.861	9.48	18.03	14.68	0.431	0.1228	0.731	0.0024	2.396
41	40	1.861	14.90	16.98	13.08	0.519	0.1115	0.700	0.0024	2.246
42	40	1.861	8.37	16.61	11.14	0.553	0.1038	0.680	0.0024	2.150
43	40	1.861	15.68	17.8	28.53	0.449	0.1450	0.848	0.0026	3.055
44	40	1.861	13.67	17.54	9.44	0.471	0.1040	0.682	0.0025	2.182
45	40	1.861	16.58	17.45	11.86	0.479	0.1263	0.764	0.0025	2.589
46	40	1.861	13.02	17.57	11.42	0.468	0.0320	0.617	0.0084	2.542
47	40	1.861	14.80	18.11	25.26	0.425	0.0367	0.653	0.0080	2.735
48	40	1.861	12.18	17.25	12.24	0.496	0.0287	0.593	0.0094	2.463
49	40	1.861	14.32	16.45	18.94	0.568	0.0320	0.643	0.0095	2.786
50	40	1.861	18.24	16.5	0.00	0.564	0.0295	0.580	0.0084	2.314
51	40	1.861	13.57	17.4	16.16	0.483	0.0317	0.585	0.0080	2.318
52	50	1.551	12.17	17.38	36.63	0.482	0.0275	0.627	0.0098	2.588
53	50	1.551	13.75	16.55	24.24	0.556	0.0357	0.669	0.0084	2.743
54	50	1.551	13.74	17.96	33.36	0.434	0.0362	0.639	0.0075	2.491
55	50	1.551	16.80	15.08	18.71	0.708	0.0347	0.596	0.0074	2.235
56	50	1.551	13.50	18.74	46.70	0.374	0.0382	0.705	0.0084	2.968
57	50	1.551	15.77	17.31	9.95	0.488	0.0473	0.614	0.0044	2.052
58	50	1.551	18.38	16.32	0.00	0.578	0.0458	0.577	0.0047	1.901
59	50	1.551	19.90	15.92	0.00	0.618	0.1015	0.699	0.0025	2.162
60	50	1.551	21.52	15.24	0.00	0.690	0.1058	0.684	0.0024	2.074
61	50	1.551	23.16	15.43	0.00	0.669	0.0885	0.619	0.0024	1.790
62	50	1.551	25.97	15.22	0.00	0.692	0.0825	0.600	0.0024	1.702

[a]Here P_c is clay percentage, d_a is arithmetic mean size, W is antecedent moisture content, γ_d is dry density, UCS is unconfined compressive strength, e is void ratio, h is flow depth, U is mean velocity of flow, S_f is energy slope, and τ_{cc} is dimensionless critical shear stress of cohesive sediments.

Table 2. Sediment and Hydraulic Parameters for Incipient Motion Condition of Cohesive Sediments Formed by Clay-Sand-Gravel Mixtures

Run	P_c, %	d_a, mm	W, %	γ_d, KN/m^3	UCS, KN/m^2	e	h, m	U, m/s	S_f	τ_{cc}, N/m^2
63	10	1.498	7.20	19.12	0.0	0.357	0.0251	0.363	0.0058	0.991
64	10	1.498	11.40	19.21	0.0	0.351	0.0353	0.459	0.0043	1.308
65	10	1.498	7.69	20.80	0.0	0.247	0.0369	0.450	0.0046	1.291
66	10	1.498	7.20	19.32	0.0	0.343	0.0605	0.460	0.0020	1.084
67	10	1.498	13.40	18.00	0.0	0.442	0.0674	0.488	0.0020	1.184
68	10	1.498	10.26	19.22	0.0	0.350	0.0855	0.550	0.0020	1.417
69	10	1.498	9.50	17.47	0.0	0.485	0.0936	0.533	0.0020	1.352
70	20	1.332	7.20	19.12	7.51	0.355	0.0696	0.510	0.0020	1.228
71	20	1.332	10.00	19.25	15.18	0.345	0.0730	0.512	0.0024	1.292
72	20	1.332	7.42	19.01	12.50	0.362	0.0257	0.450	0.0058	1.328
73	20	1.332	10.42	18.42	5.31	0.406	0.0344	0.450	0.0043	1.232
74	20	1.332	9.72	18.86	16.85	0.373	0.0438	0.560	0.0043	1.710
75	20	1.332	10.29	17.68	4.53	0.465	0.0259	0.428	0.0058	1.232
76	20	1.332	14.05	18.12	0.00	0.429	0.0852	0.482	0.0017	1.083
77	20	1.332	12.80	18.96	0.00	0.366	0.0828	0.548	0.0020	1.367
78	20	1.332	11.25	19.15	17.87	0.352	0.0328	0.530	0.0056	1.682
79	20	1.332	13.76	19.09	0.00	0.357	0.0750	0.562	0.0025	1.501
80	20	1.332	14.70	18.17	0.00	0.425	0.0802	0.545	0.0020	1.356
81	20	1.332	15.20	17.62	0.00	0.470	0.0319	0.460	0.0044	1.281
82	30	1.166	9.51	18.33	9.63	0.410	0.0383	0.476	0.0044	1.304
83	30	1.166	13.84	18.05	6.30	0.432	0.0402	0.506	0.0042	1.413
84	30	1.166	7.42	19.01	13.61	0.360	0.0341	0.467	0.0053	1.328
85	30	1.166	11.73	18.92	23.19	0.366	0.0308	0.530	0.0075	1.751
86	30	1.166	13.69	18.25	14.58	0.416	0.0251	0.500	0.0080	1.631
87	30	1.166	16.88	17.25	4.32	0.499	0.0347	0.510	0.0059	1.557
88	30	1.166	17.28	17.05	0.00	0.516	0.0368	0.450	0.0044	1.199
89	30	1.166	17.86	16.80	0.00	0.539	0.0700	0.495	0.0021	1.150
90	30	1.166	21.69	16.27	0.00	0.589	0.0650	0.455	0.0023	1.037
91	30	1.166	14.68	17.57	16.31	0.471	0.0875	0.570	0.0023	1.453
92	30	1.166	13.06	17.96	9.25	0.439	0.0534	0.541	0.0037	1.513
93	40	1.003	10.28	17.42	15.92	0.481	0.0300	0.444	0.0056	1.200
94	40	1.003	13.89	17.43	18.49	0.480	0.0356	0.555	0.0052	1.646
95	40	1.003	14.35	17.72	20.24	0.456	0.0286	0.465	0.0054	1.274
96	40	1.003	13.46	18.39	25.18	0.403	0.0250	0.521	0.0084	1.688
97	40	1.003	13.20	18.14	10.50	0.422	0.0200	0.467	0.0085	1.437
98	40	1.003	16.15	17.47	0.00	0.477	0.0408	0.455	0.0035	1.107
99	40	1.003	18.82	16.47	0.00	0.567	0.0314	0.462	0.0040	1.171
100	40	1.003	19.19	16.05	0.00	0.607	0.0236	0.387	0.0050	0.949
101	50	0.834	10.06	18.15	12.35	0.419	0.0287	0.422	0.0050	1.033
102	50	0.834	16.07	17.05	9.96	0.510	0.0395	0.495	0.0040	1.241
103	50	0.834	17.72	16.93	14.23	0.521	0.0323	0.538	0.0055	1.523
104	50	0.834	17.41	16.37	16.93	0.573	0.0305	0.530	0.0055	1.489
105	50	0.834	17.87	17.15	14.50	0.502	0.0278	0.532	0.0060	1.531
106	50	0.834	19.11	16.12	0.00	0.597	0.0250	0.453	0.0055	1.177
107	50	0.834	20.34	16.71	0.00	0.541	0.0200	0.379	0.0060	0.920
108	50	0.834	16.57	17.72	5.56	0.453	0.0300	0.456	0.0044	1.124

determine the value of plasticity index was not possible due to presence of gravel in them.

[34] Using dimensional analysis, equation (1) is reduced to

$$\frac{\tau_{cc}}{\tau_c} = f(P_c, e, UCS^*), \tag{2}$$

with $UCS^* = UCS/\Delta\gamma_s d_a$. It is worthwhile to use the variable $(1 + P_c)$ instead of P_c such that the resulting expression for critical shear stress is valid for cohesionless sediment while $P_c = 0$. With similar consideration the variable $(1 + UCS^*)$ is used instead of UCS^* to account for both the hard consistencies and the soft consistencies of cohesive sediments. Therefore equation (2) is rewritten as

$$\frac{\tau_{cc}}{\tau_c} = f[(1 + P_c), e, (1 + UCS^*)]. \tag{3}$$

The equation (3) is the functional relationship for critical shear stress in cohesive sediments consisting of the mixture of gravel-sand-silt and clay sizes. In the absence of the availability of the values of the mean size of cohesive sediment mixture for some of the literature data, the variation of τ_{cc}, instead of τ_{cc}/τ_c, is plotted against the relevant independent variables in Figures 8 and 9. Figure 8 indicated that the τ_{cc} values of cohesive sediment mixtures containing gravel (i.e., data of present study) show a decreasing trend with an increase in the void ratio. However, the $\tau_{cc} - e$ relationship is weak for cohesive sediment mixtures not containing gravel as indicated by Figure 8 for the literature data. Figure 9 presents a plot showing the variation of τ_{cc} with the percentage clay content in the cohesive sediment mixtures. It is indicated by Figure 9 that generally the τ_{cc} values increase with increasing values of P_c. A more strong relationship

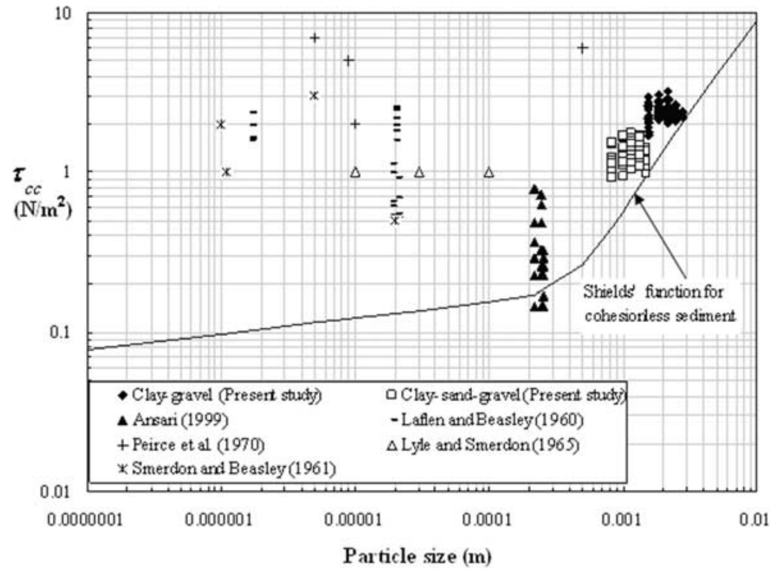

Figure 7. Comparison of τ_{cc} with Shields's function for the cohesionless sediment.

however, is noticed between τ_{cc}/τ_c and P_c as depicted by Figure 10.

[35] In order to study the influence of the parameters appearing in equation (3) on τ_{cc}/τ_c, the data collected only in the present investigation were used. Data of *Ansari* [1999] are however used subsequently for validation purposes. The rest of the investigators' data could not be used in subsequent analysis, as the values of *UCS* were not available for them. The variation of τ_{cc}/τ_c with void ratio for various percentages of clay in bed material is shown in Figure 10, which indicates that τ_{cc}/τ_c decreases with an increase in the void ratio. Figure 10 also dictates that variation of τ_{cc}/τ_c with P_c can be depicted by parallel partitioning lines indicating that the resistance of cohesive sediment mixtures to detachment by the flow varied systematically with P_c values. The parallel lines in Figure 10 were drawn by eye judgment and they merely illustrate that the data can be partitioned very well based on the values of P_c. However, it is seen from Figure 10 that while P_c = 10%, i.e., the cohesive sediments with lower percentage of clay content follow slightly different trend from the bulk data. Nevertheless, Figure 10 revealed that τ_{cc}/τ_c > 1.0, even while P_c = 10%. A very significant increase in τ_{cc}/τ_c with an increase in P_c may also be noted in Figure 10, which is in conformity with the results of *Laflen and Beasley* [1960], *Lyle and Smerdon* [1965], *Kothyari et al.* [2006], *van Ledden et al.* [2004], and *van Rijn* [2007].

[36] To quantify the effect of *UCS* on τ_{cc}/τ_c, Figure 11 was derived, by placing the data into three ranges based on their *UCS* values. It was noticed that τ_{cc}/τ_c is also an increasing function of *UCS*. Similar result has also been reported earlier by *Kamphuis and Hall* [1983]. The zones

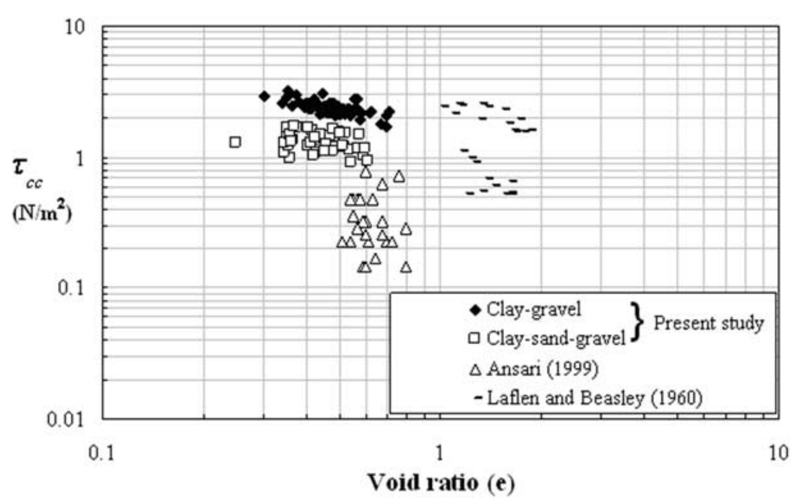

Figure 8. Variation of τ_{cc} with void ratio.

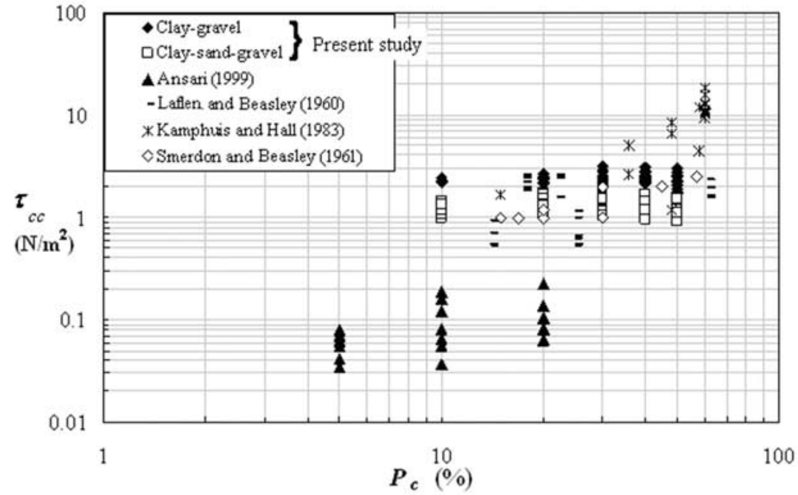

Figure 9. Variation of τ_{cc} with clay percentage.

on different *UCS* values were marked on Figure 11 by eye judgment and these are depicted by the dotted lines as a few data points encroach in to the neighboring zones, particularly while P_c is higher.

[37] After making a number of trials using all relevant dimensionless parameters, the following relationship was derived for τ_{cc}:

$$\frac{\tau_{cc}}{\tau_c} = 0.94(1 + P_c)^{3/2} e^{-1/6} (1 + 0.001 UCS^*)^{9/20}. \tag{4}$$

For a graphical comparison the observed values of τ_{cc}/τ_c are plotted in Figure 12 against its values computed by equation (4). It is evident from Figure 12 that equation (4) predicts the results with a maximum error of ±20% for about 95% of data. Such accuracy is considered as satisfactory due to the complexities involved in the process being investigated as even estimation of τ_c can at best have similar accuracy [*Garde and Ranga Raju*, 2000]. Several other dimensionless groups were also used to study the variations in τ_{cc}/τ_c. The best results obtained only are however reported above. The

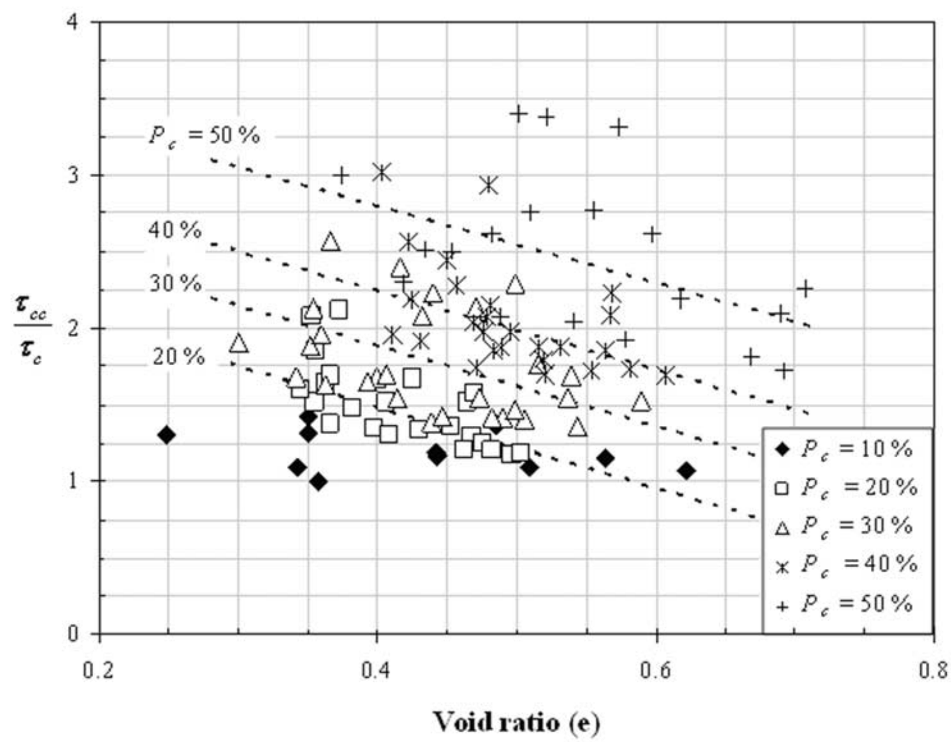

Figure 10. Variation of τ_{cc}/τ_c with void ratio and clay percentage.

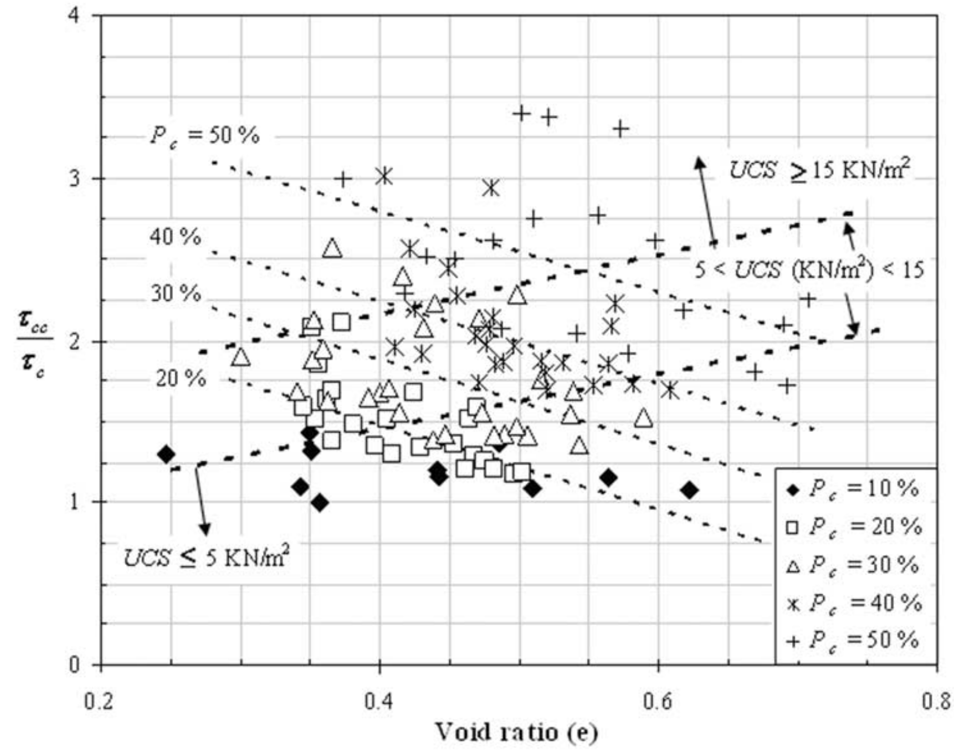

Figure 11. Variation of τ_{cc}/τ_c with void ratio, clay percentage, and unconfined compressive strength.

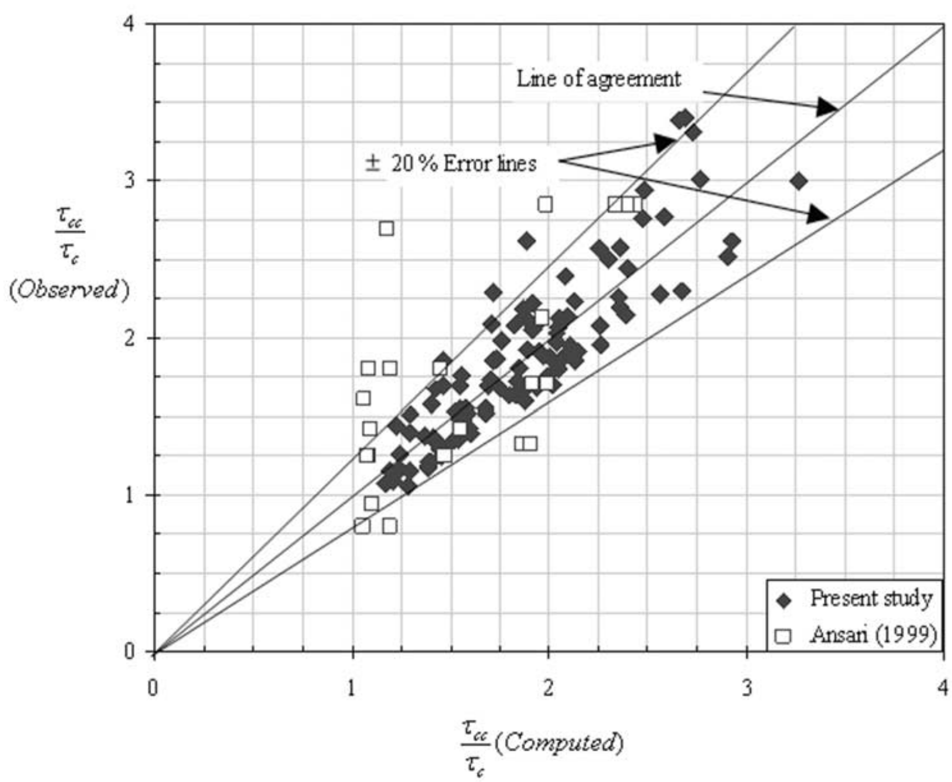

Figure 12. Comparison of observed values of τ_{cc}/τ_c with those computed by equation (4).

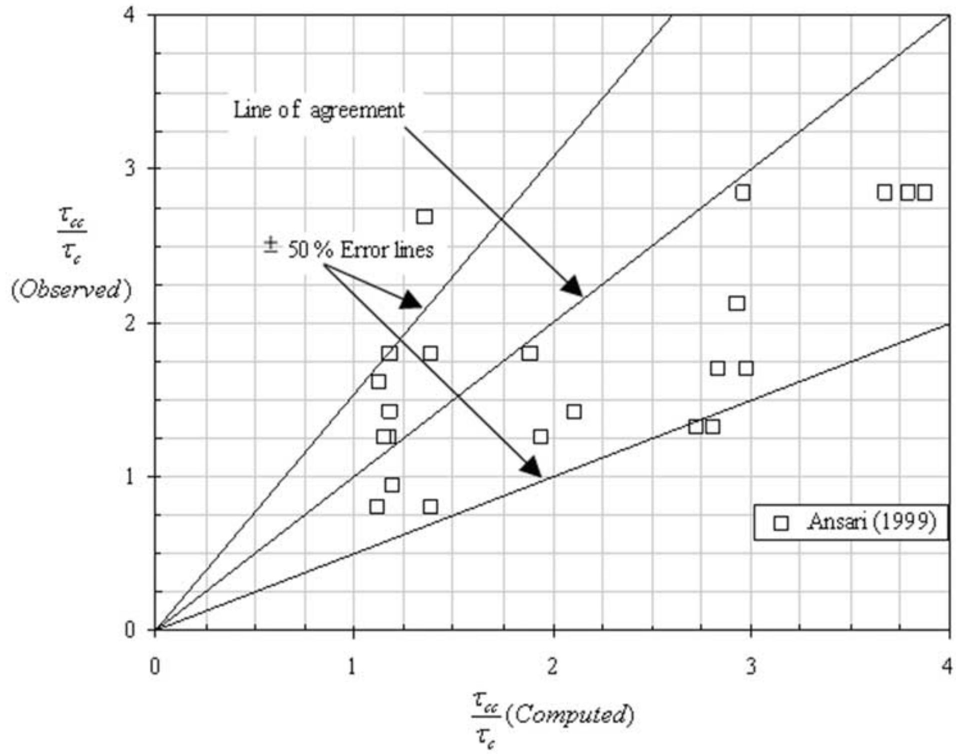

Figure 13. Comparison of observed values of τ_{cc}/τ_c with those computed by equation (5) for data of *Ansari* [1999].

parameter *UCS** appropriately explains the engineering (mechanical) behavior of the cohesive sediments originating from different hydrogeological regions and therefore having varying physicochemical characteristics. Hence equation (4) is expected to produce realistic results also for the field data from different geographical locations.

[38] Data of *Ansari* [1999] are used at first instance for validation of equation (4) and therefore computed values of τ_{cc}/τ_c by equation (4) for these data are also plotted with the corresponding observed values in Figure 12. Much larger scatter in Figure 12 for these data is attributed to the fact that gravel size sediment was not present in the investigation of *Ansari* [1999]. Figure 12, however, hinted that the following modified form of equation (4) could produce more systematic scatter of estimated τ_{cc}/τ_c values for clay-sand mixtures about the line of agreement for the data of *Ansari* [1999]:

$$\frac{\tau_{cc}}{\tau_c} = 1.88(1 + P_c)^{3/2} e^{-1/6} (1 + 0.001 UCS^*)^{9/20} - 1.0. \quad (5)$$

Comparison of the corresponding observed values of τ_{cc}/τ_c for Ansari's data with those computed by equation (5) indeed confirmed this notion (see Figure 13). The scatter of ±50% shown by Figure 13 albeit is large, but is similar to that of *Kothyari et al.* [2006] and other relationships for such data on clay-sand mixtures [*Ansari*, 1999].

[39] For the natural cohesionless sediment the value of porosity can be considered to be about 0.4 [*Garde and Ranga Raju*, 2000] which corresponds to a void ratio value equal to 0.66. In case of uniform cohesionless sediments, like gravel, and nonuniform cohesionless sediment mixtures τ_{cc} computed by equations (4) and (5) must be equal to τ_c; the critical shear stress for arithmetic mean size of the cohesionless sediment mixture. Values of P_c and *UCS* in that case shall be equal to zero and since $e = 0.66$; thus as per equations (4) and (5) indeed $\tau_{cc} = \tau_c$ under these conditions.

6. Conclusions

[40] A total of 108 experimental runs were conducted for the determination of τ_{cc} of cohesive sediments consisting of two types of mixtures: one of clay and fine gravel, and the other with clay, sand and fine gravel. The clay content in the sediment mixtures was varied in the mixtures from 10% to 50% by weight. Three modes of initiation of motion namely; pothole erosion, line erosion and mass erosion of sediment mixtures were noticed. The modes of initiation of motion mainly changed with: clay percentages in the mixture, its antecedent moisture characteristics, and the applied shear stress. The amount of increase in the critical shear stress of the sediment mixtures due to the presence of clay is quantified. The data collected from the present investigation as well as the data available from literature were analyzed to parameterize the variations of τ_{cc}. Depending upon P_c and other characteristics of the sediment mixture, the ratio τ_{cc}/τ_c is noticed to change from a value of slightly more than one to about more than three. On the basis of dimensional consideration, the functional form of relationship for determination of τ_{cc}/τ_c was developed. Next equation (4) was derived for the estimation of τ_{cc}/τ_c in

terms of easily measurable parameters, for the cohesive sediment mixtures containing gravel which is valid for $10 \leq P_c$ (%) ≤ 50. Similarly equation (5) is proposed for τ_{cc}/τ_c estimation for the cohesive sediment mixtures without gravel. In the absence of cohesion, the proposed relationships reduce to the critical shear stress relationship for the cohesionless sediment having the arithmetic mean size equal to that of the cohesive sediment mixture. Thus in the present study, the condition for threshold of motion by the flow of cohesive sediment mixtures with gravel, is parameterized for use by the practitioners.

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