Research Journal of
Environmental Sciences

ISSN 1819-3412

Academic
Journals Inc.

www.academicjournals.com

Research Journal of Environmental Sciences 5 (5): 399-413, 2011
ISSN 1819-3412 / DOI: 10.3923/rjes.2011.399.413

A Relationships to Determine the Critical Hydraulic Gradient and Noncohesive Sediment Transport Discharge in Rockfill Dams

[1]S.A. Mousavi, [1]E. Amiri-Tokaldany and [2]M.H. Davoudi
[1]Department of Irrigation and Reclamation Engineering, Faculty of Agricultural Engineering and Technology, University of Tehran, Karaj, Iran
[2]The Research Institute for Water Scarcity and Drought, Tehran, Iran

Corresponding Author: E. Amiri-Tokaldany, Department of Irrigation and Reclamation Engineering, Faculty of Agricultural Engineering and Technology, University of Tehran, Karaj, Iran Tel: +98 912 4984423 Fax: +98 261 2241119/201

ABSTRACT

In this research by conducting laboratory experiments along with dimensional analysis, we investigated the nondimensional sediment discharge capacity and critical hydraulic gradient in rockfill dams. Rockfill dams are a type of grade control structures used to protect river bed against fluvial erosion and also to stabilize the river banks by decreasing the bank height. One of the main issues on utilizing rockfill dams is to keep its permeability enough so that it could be able to pass the flow as well as the sediment load through its body during flooding, avoiding sediments resettlement inside the pores. In this regard, the design of rockfill dams should be carried out so that the available hydraulic gradient is always kept greater than the critical hydraulic gradient, which consequently results in transporting the sediment through the dam body. In this research, a relationship to estimate the critical hydraulic gradient to transport noncohesive sediment through rockfill dam body is introduced. We tested the new equation using a set of published data and we found MRE equal to 0.4%. Also, using laboratory data obtained from tests on a rectangular rockfill dam, performing dimensional analysis and using linear regression, an exponential relationship for the required discharge to transport the sediments through the body of rockfill dam is presented. When we tested the validity of exponential relationship, we found a good accuracy for the equation (MRE = 9.4%) indicating that the introduced relation predicts the nondimensional sediment transport capacity well.

Key words: Rockfill dams, critical hydraulic gradient, noncohesive sediments, control structures, riverbanks

INTRODUCTION

Seasonal floods may carry a huge amount of sediment particles eroded by surface flow as well as fluvial erosion of river bed. The fluvial erosion is also responsible for increasing the height and the slope of the river bank and consequently causes the bank mass failure results in damages to the fertile lands as well as existing facilities in the adjacent areas to the river banks. Thus, the flooding control as well as the bank and bed protection, are among the important issues in river engineering. In this regard, a large number of researches have been carried out to introduce the appropriate structures to deal with the flooding and its consequences. Rockfill dams are a type of those structures, build by putting freely the stones over the bed, used for flooding control and also protecting the river bed against fluvial erosion and its harmful consequences.

One of the important features of rockfill dams is its high body permeability from which both the flow and the sediment are able to pass downstream during flooding. This is important because in the other types of control structures in which the flow and sediment can only pass over the structures, most part of the sediment load trapped behind the dam and hence the relatively clear water pass over the dam toward downstream. The clear water reach to the downstream has ability to erode the bed, seriously. In rockfill dams, since the sediment particles transported by the flow, pass through the dam body and discharged to the downstream, no more bed and bank erosion is expected. Hence, it is important to keep its permeability high through the time and prevent the trapping of the sediment inside the pores of the dam body. Therefore, during the performance of this control structures, the available hydraulic gradient has to be more than critical hydraulic gradient which is required to keep the sediment particles inside the flow. Hence, determination of the critical hydraulic gradient in this structure is an important issue.

In aggregate porous media of rockfill dams, because of large size of the pores, the flow is turbulent so that Darcy's law for laminar flow is not valid and the relation between the flow velocity and hydraulic gradient is nonlinear. Prony (Li *et al.*, 1998) introduced an exponential relationship between the flow velocity and hydraulic gradient in aggregate porous media. Saktivadivel (1972) investigated the sediment movement through porous media with taking into account the effect of particle weight. Considering the balance between the motivating and resistance forces apply upon a particle standing over a slopped surface, Saktivadivel introduced the following equation to estimate the critical hydraulic gradient for sediment motion in aggregate porous media:

$$i_c = 2K_c(G_s - 1)g\,d_s(\cos\theta\tan\phi - \sin\theta) \tag{1}$$

in which, i_c is the critical hydraulic gradient, K_c is A constant coefficient obtained from tests, G_S is the relative density of the sediment particles, g is the gravity acceleration, d_s is the sediment particle diameter, θ is the slope angle of the bed and ϕ is the sediment angle of repose in aggregate porous media.

Considering N-1 moving layers of sediment and by assuming linear distribution for the velocity of sediment layers, Saktivadivel (1972) introduced the Eq. 2 to estimate the amount of sediment transport inside the porous media in laminar flow:

$$q_s = K'_{sak}.\ \rho_s.\ d_s\left(\frac{i}{i_c}\right)\left(\frac{i - i_c}{i_c}\right) \tag{2}$$

where, i is the hydraulic gradient, q_s is the amount of sediment transport, ρ_s is the density of sediment particle and K'_{sak} is a constant coefficient. Note that to maintain the laminar flow, Saktivadivel (1972) used oil instead of water.

Cunningham *et al.* (1987) by conducting a series of laboratory tests on river bed material in a flume with a length of 7.6 m in which the flow passes through and over the non moving bed materials, investigated the blocking of pores results from sedimentation inside and over the bed materials. To find the variation of material permeability over time while there was sediment feeding, continuously, Cunningham *et al.* (1987) used four different sediment concentrations, for each they measured the magnitude of discharge flow passing the material in a time interval during 5 days. Using dimensional analysis, Cunningham *et al.* (1987) introduced Eq. 3:

$$\frac{Q_i}{Q_o} = f\left(\frac{V.R_h}{\upsilon}, C, \frac{t_c}{d_s}, \frac{R_h}{H}\right) \quad (3)$$

where, Q_i is the magnitude of initial discharge passing from bed material, Q_o is the magnitude of discharge in time of t after sediment feeding, V is the average flow velocity in the flume, R_h is the hydraulic radius, υ is the kinematic viscosity of water, C is the weighted concentration of sediment materials, t_c is thickness of sediment layer form over the materials and H is height of water over the materials. The test results show that the reduction of permeability (represented by Q_i/Q_o), had a good correlation with $V.R_h/\upsilon$, t_c/d_s but had no good correlation with the two others (Cunningham *et al*., 1987).

Joy *et al*. (1991) investigated the sediment transport through porous media in non-linear flow conditions. They carried out fourteen experiments on a sample of porous media with the length of 600 mm, width of 279 mm and 300 mm in height. The sample was located in a flume with a variable slope in which sediment injection was performed in upstream edge of the flume. Joy *et al*. (1991) used three different sizes of sediments and also materials of porous media while the ratio of porous materials to the sediments differed from 13.9 to 91.7, the bottom slope of flume was taken between 0.09 to 0.6 and Reynolds numbers for the experiments varied from 180 to 940. By applying dimensional analysis, Joy *et al*. (1991) found four non-dimensional parameters and using the test results, they found Eq. 4:

$$q_* = 26.2\,(R_e)^{-1.23}\,(\lambda_d)^{0.54}\,(S_p)^{-1.39} \quad (4)$$

Where:

$$\begin{aligned} q_* &= \frac{q_s.n}{\rho_s.d_s.V_b} \\ R_e &= \frac{V_b.d}{n.\upsilon} \\ \lambda_d &= \frac{d}{d_s} \\ S_p &= \tan(\varphi - \theta) \end{aligned} \quad (5)$$

and q_* is the non dimensional sediment transport parameter, d is the size of porous material, Re is the reynolds number in porous media and S_p is the slope parameter. Joy *et al*. (1991) used slopes between 9 to 60% in which hydraulic gradient usually was more than critical gradient. Hence, there was no possibility to achieve critical point in their tests.

Schalchi (1995) investigated the gradual reduction of hydraulic conductivity of river bed results from sedimentation effects. Schalchi (1995) reported Eq. 6:

$$K(t) = \frac{g\,L}{\upsilon\sqrt{\beta^2 + \frac{2g\Delta h}{\upsilon} r\,C_w\,t}} \quad (6)$$

where, K(t) is the hydraulic conductivity of material at time (t) after sediment injection, t is the time after sediment injection, L is the seepage length, Δ_h is the height of water over sample, C_w is the weighted concentration, r is the specific hydraulic resistance which varies from 2×10^{10} to 2×10^{12} and is a function of material uniformity, Reynolds number, hydraulic gradient and shear stress, β is the hydraulic resistance when no sedimentation has taken place and defined as:

$$\beta = \frac{g\,L}{K_o\,\upsilon} \tag{7}$$

where, K_o is the initial hydraulic conductivity. Schalchi (1995) noted that Eq. 6 is valid for:

$$0 < i < 0.93 \quad , \quad 2400 < R_e < 24800 \quad , \quad 0.008 \le C_w \le 1.5 \quad , \quad 0.0115 < d_{10}/d_b < 0.178 \tag{8}$$

in which d_{10} is the size that 10% of bed material are finer of that size and d_b is the mean diameter of bed particles.

Moradlo (1999) investigated the gradual reduction of hydraulic conductivity of porous media during passing two-phased fluid through dam body with different sediment concentration under relatively high hydrostatic pressure. He conducted tests in which water, including different concentration of bentonite, passed through soil samples and the variation of soil permeability was measured. In addition, to study the variation of soil permeability at different depths of the soil samples, Moradlo (1999) installed some piezometers within different depths so that he could be able to get the variation of soil conductivity in different depths as well as time after beginning of each test. His results showed that the above variation followed Eq. 9:

$$K = a\,e^{-bt} \tag{9}$$

where, K is the hydraulic conductivity in time t and a, b are the coefficients depend on sediment concentration.

Moreover, based on Saktivadivel's equation, Samani and Emadi (2003) introduced following equation for turbulent flow passing through aggregate porous media:

$$q_s = 0.0461\ \rho_s\ d_s \left(\frac{Q - Q_c}{Q_c}\right)^{0.6236} \tag{10}$$

Emadi *et al*. (2005) investigated the movement of noncohesive sediments in aggregate porous media where the Reynolds number varied from 1000 to 4000, the average particle size of porous media of 14.5 and 21 mm and the average particle size of sediments of 0.256, 0.362 and 0.512 mm. Emadi *et al*. (2005) assumed that the variation of critical hydraulic gradient in aggregate porous media is nonlinear and finally obtained the Eq. 11:

$$i_c = 1.667\ (G_s - 1)g\ d_s(\cos\theta\ \tan\phi - \sin\theta)^{4.699} \tag{11}$$

In this research we carried out a set of laboratory experiments to determine the critical hydraulic gradient and also to determine the repose angle of sediment within aggregate porous media. Moreover, we performed a dimensional analysis to obtain a relationship for discharge of sediment transport. The method of performing experiments and dimensional analysis are described below.

MATERIALS AND METHODS

Flume specifications and rockfill dam sample: In this research all laboratory tests were carried out at the Research Institute for Water Scarcity and Drought, Tehran, Iran, from August 2008 to August 2009. To determine the critical hydraulic gradient, we used a flume with the length of 15 m, width of 0.6 m and height of 0.6 m (Fig. 1). To make a model of rockfill dams, we used a mesh basket with the length of 0.3 m, width of 0.6 m and height of 0.6 m which filled by selected sizes of the aggregates. To measure the water depth within the dam body, we installed a set of piezometers located at the bottom of the basket. To delete the fluctuation of the water surface, we used a flow quieter before the location of sediment injection. Also, to control the water level, we used a rectangular gate downstream of the sample. We determined the discharge flow using a rectangular weir at the downstream end of the flume.

Determination of sediment's angle of repose in porous media environment: To obtain the sediment's angle of repose in aggregated porous media, we used a box with dimensions of 30 cm× 30 cm×70 cm made of plexiglass. We put aggregates inside the box and filled the box with the water up to the level of aggregates. Then, we injected sediments to the box from the top. At the beginning of injection, sediment deposited in a variable slope depends on the sediment nature, but after a while, the slope remained constant so that by continuing sediment injection, the deposited slope was unchanged (Fig. 2). We measured this slope and defined it as the repose angle of sediments in aggregated porous media. The results of experiments to obtain this angle are shown in Table 1.

Determination of critical hydraulic gradient: In tests which carried out to determine the critical hydraulic gradient, we used two sizes of aggregates (median diameter of 3 and 4.5 cm),

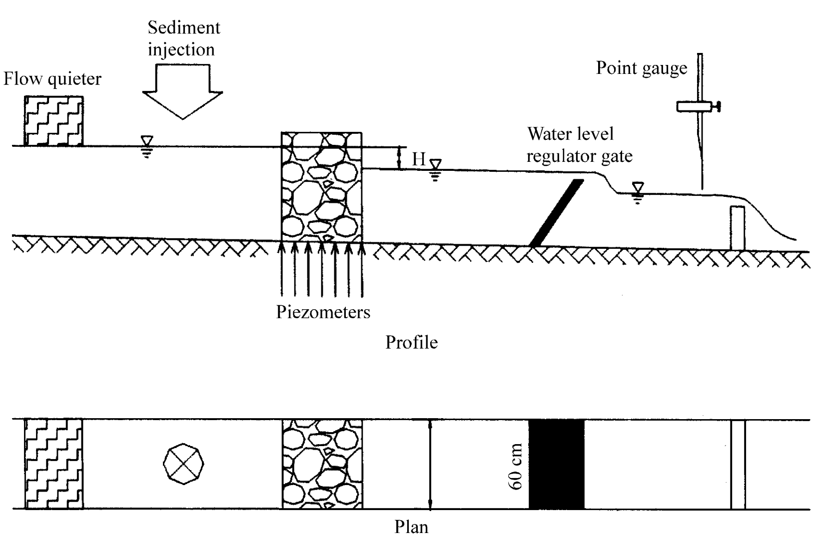

Fig. 1: A schematic view of the flume used for laboratory tests

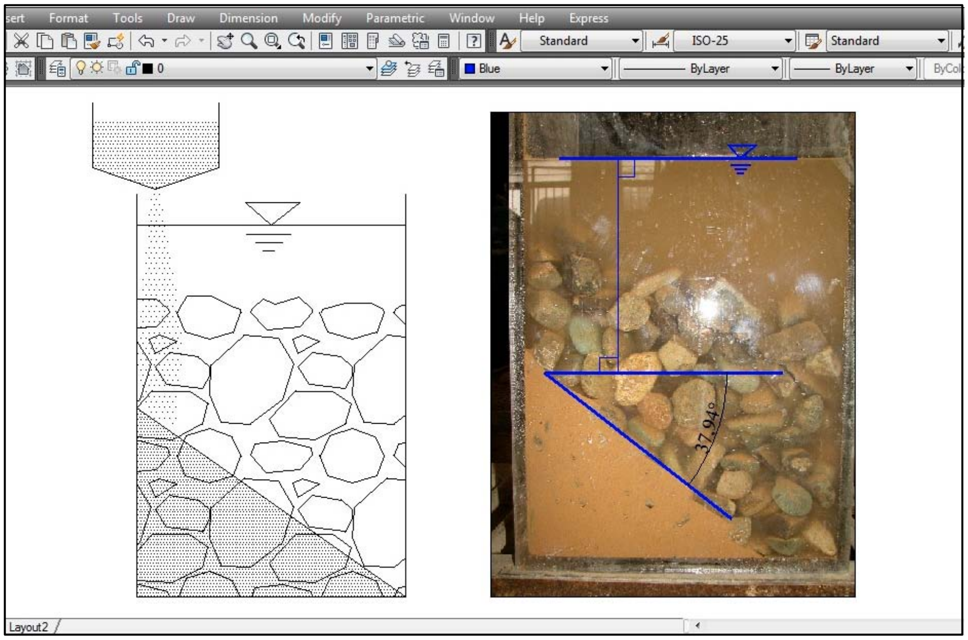

Fig. 2: Method to determine the angle of repose of the sediments in aggregate porous media

Table 1: The angle of repose for sediment particles in aggregated porous media

Diameter of aggregate (cm)	Standard deviation (cm)	Diameter of sediments (mm)	Ratio of aggregate to sediments diameters	Sediment angle of repose (degrees)
3.0	0.50	0.36	83	39.96
3.0	0.75	0.36	83	38.68
3.0	0.00	0.36	83	37.60
3.0	0.50	0.27	111	37.49
3.0	0.75	0.27	111	37.19
3.0	0.00	0.27	111	37.02
4.5	0.50	0.36	125	37.01
4.5	0.75	0.36	125	36.58
4.5	0.00	0.36	125	36.33
4.5	0.50	0.27	167	36.22
4.5	0.75	0.27	167	35.65
4.5	0.00	0.27	167	34.36
3.0	0.50	0.15	200	32.57
3.0	0.75	0.15	200	32.45
3.0	0.00	0.15	200	32.20
4.5	0.50	0.15	300	31.84
4.5	0.75	0.15	300	31.64
4.5	0.00	0.15	300	30.13

three standard deviation (SD) for each size (0.0, 0.5 and 0.75 cm) and three sediment sizes (median diameter of 0.15, 0.27 and 0.36 mm). The total number of the experiments for this part was eighteen. Regarding to discharge flow of sediment transport, we used two discharges for the above cases and totally we carried out 36 tests, separately.

Table 2: The specifications of the aggregate and sediment particles, and calculated critical hydraulic gradient

No.	Diameter of aggregates (cm)	Standard deviation (cm)	Diameter of sediment particles (mm)	Relative density of sediments	Critical hydraulic gradient
1	3.0	0.00	0.15	2.61	0.0321
2	3.0	0.00	0.36	2.64	0.0945
3	3.0	0.00	0.27	2.63	0.0679
4	4.5	0.00	0.36	2.64	0.0902
5	4.5	0.00	0.27	2.63	0.0672
6	4.5	0.00	0.15	2.61	0.0305
7	3.0	0.50	0.36	2.64	0.1090
8	3.0	0.50	0.27	2.63	0.0750
9	3.0	0.50	0.15	2.61	0.0340
10	3.0	0.75	0.36	2.64	0.0988
11	3.0	0.75	0.27	2.63	0.0734
12	3.0	0.75	0.15	2.61	0.0319
13	4.5	0.50	0.36	2.64	0.0921
14	4.5	0.50	0.27	2.63	0.0717
15	4.5	0.50	0.15	2.61	0.0321
16	4.5	0.75	0.36	2.64	0.0910
17	4.5	0.75	0.27	2.63	0.0696
18	4.5	0.75	0.15	2.61	0.0312

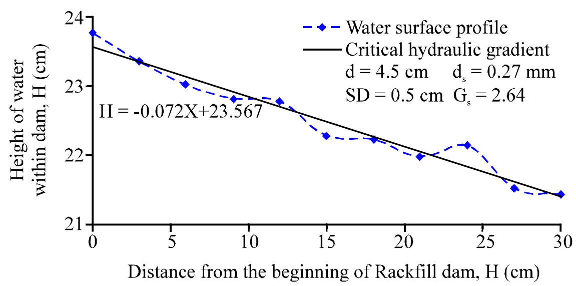

Fig. 3: Method to determine critical hydraulic gradient

To determine the water surface profile and hydraulic gradient, we used a set of piezometers installed at the bottom of the basket. Regarding the sizes of aggregates and sediments, to run the tests, we started from low hydraulic gradient and then increased it gradually until sediment transportation within the porous media was begun. At this time, sediment injection was stopped and the values of the piezometers were read. To read piezometers on time and accurately, we used photo taken technique where we used a very high resolution digital camera. The photos taken by the camera imported to the environment of Grapher 7 software from which the requested data were obtained. Data collected to determine critical hydraulic gradient in rockfill dam are shown in Table 2.

In Fig. 3 the water surface profile and the critical hydraulic gradient for one of the experiments is shown. As shown in Fig. 3, the critical hydraulic gradient in this experiment is 0.0717.

The above process was repeated for all tests and based on the results, the variation of hydraulic gradient versus repose angle of the sediments is drawn in Fig. 4.

As shown in Fig. 4, by increasing the ratio of aggregate to sediment diameter and also increasing the amount of sediment's angle of repose in aggregate porous media, the critical

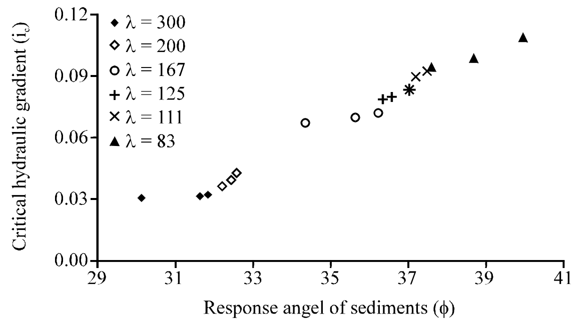

Fig. 4: The variation of hydraulic gradient versus repose angle of the sediments

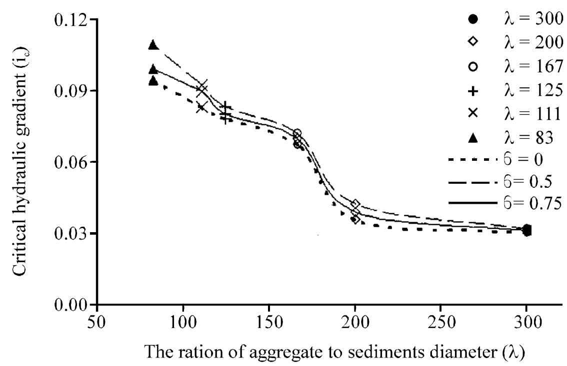

Fig. 5: Variation of critical hydraulic gradient versus the ratio of aggregate to sediment diameters (λ)

hydraulic gradient is also increased. This is because by increasing the repose angle of sediments in porous media, their stability and consequently their resistance against the flow are increased and as a results, more critical hydraulic gradient is required to transport these sediments from within dam body. In Fig. 5 the variation of critical hydraulic gradient versus the ratio of aggregate to sediment diameters (λ) is shown.

Figure 5 shows that by increasing λ, the value of critical hydraulic gradient is decreased. This is because by increasing the aggregate diameter and size of pores, the flow turbulence and fluctuation within dam body is decreased. This results in a reduction in resistance against sediment transport and hence smaller amount of the critical hydraulic gradient is required to transport the sediments from inside of the dam body.

To obtain a relationship to determine the amount of hydraulic gradient, we used linear regression method. By considering Eq. 1, reported by Saktivadivel (1972) and also with respect to Eq. 7, reported by Emadi *et al.* (2005), the potential form of relationship for hydraulic gradient in aggregated porous media can be written as:

$$i = a.\ V^b \tag{12}$$

in which a and b are constant coefficients which should be determined using field and laboratorial tests.

To change the Saktivadival linear equation to a non linear form, the equation can be written as:

$$i_c = a\ (G_s - 1)g\ \ d_s\ (\cos\theta \tan\phi - \sin\theta)^b \tag{13}$$

Using statistical analysis of data obtained from experiments and employing SPSS software, the values of a and b were estimated as 22.139 and 1.066, respectively. Hence, the final equation to determine the critical hydraulic gradient is as:

$$i_c = 22.139\ (G_s - 1)g\ d_s\ (\cos\theta \tan\phi - \sin\theta)^{1.066} \tag{14}$$

Sediment transport discharge relationship: To obtain a nondimensional relationship for sediment transport within rockfill dam body, the parameters affecting the phenomenon are: the diameter of sediment particles, sediment density, diameter of aggregates, actual flow velocity (V_a) in aggregated porous media so that (V_a = V/n in which V represents the apparent velocity and n represents the porosity of aggregate materials), fluid viscosity and effective hydraulic gradient (S_e) for sediment transport. So, we can write as:

$$q_s = f(d_s, \rho_s, d, n, V_a, \upsilon, S_e) \tag{15}$$

or

$$f(q_s, d_s, \rho_s, d, n, V_a, \upsilon, S_e) = 0.0 \tag{16}$$

where, q_s is the sediment transport discharge within aggregated porous media in unit width of dam. As there are six dimensional parameters and also three dimensions of length, mass and time, three nondimensional parameters can be written as:

$$\begin{aligned} \pi_1 &= V_a^{x_1} \times d^{y1} \times \rho_s^{z_1} \times q_s \\ \pi_2 &= V_a^{x_2} \times d^{y2} \times \rho_s^{z_2} \times d_s \\ \pi_3 &= V_a^{x_3} \times d^{y3} \times \rho_s^{z_3} \times \upsilon \end{aligned} \tag{17}$$

By writing dimensional equation for each of nondimensional parameters, it can be seen that:

$$\begin{aligned} &\pi_1 = V_a^{x_1} \times d^{y1} \times \rho_s^{z_1} \times q_s \ \Rightarrow \left[L\ \ T^{-1}\right]^{x_1} \left[L\right]^{y_1} \left[M\,L^{-3}\right]^{z_1} \left[M\,L^{-1}\,T^{-1}\right] = 1 \\ &\Rightarrow \begin{cases} x_1 + y_1 - 3z_1 - 1 = 0 \\ -x_1 - 1 = 0 \\ z_1 + 1 = 0 \end{cases} \quad \Rightarrow \begin{cases} x_1 = -1 \\ y_1 = -1 \\ z_1 = -1 \end{cases} \quad \Rightarrow \ \pi_1 = \frac{q_s}{V_a\ d\ \rho_s} \end{aligned} \tag{18}$$

$$\begin{aligned} &\pi_2 = V_a^{x_2} \times d^{y2} \times \rho_s^{z_2} \times\ d \ \Rightarrow \left[L\ \ T^{-1}\right]^{x_2} \left[L\right]^{y_2} \left[M\,L^{-3}\right]^{z_2} \left[L\right] = 1 \\ &\Rightarrow \begin{cases} x_2 + y_2 - 3z_2 - 1 = 0 \\ -x_2 = 0 \\ z_2 = 0 \end{cases} \quad \Rightarrow \begin{cases} x_2 = 0 \\ y_2 = -1 \\ z_2 = 0 \end{cases} \quad \Rightarrow \ \pi_2 = \frac{d_s}{d} \end{aligned} \tag{19}$$

$$\pi_3 = V_a^{x_3} \times d^{y3} \times \rho_s^{z_3} \times \upsilon \Rightarrow \left[L\ T^{-1}\right]^{x_3} \left[L\right]^{y_3} \left[ML^{-3}\right]^{z_3} \left[L^2 T^{-1}\right] = 1$$
$$\Rightarrow \begin{cases} x_3 + y_3 - 3z_3 + 2 = 0 \\ -x_3 - 1 = 0 \\ z_3 = 0 \end{cases} \Rightarrow \begin{cases} x_3 = -1 \\ y_3 = -1 \\ z_3 = 0 \end{cases} \Rightarrow \pi_3 = \frac{\upsilon}{V_a\ d} \tag{20}$$

So, Eq. 17 can be written as:

$$\left(\frac{q_s . n}{\rho_s . d. V}\right) = k. \left(\frac{V. d}{n. \upsilon}\right)^{\alpha 1} . \left(\frac{d}{d_s}\right)^{\alpha 2} . (S_e)^{\alpha 3} \tag{21}$$

By introducing:

$$\begin{aligned} q_* &= \frac{q_s . n}{\rho_s . d. V} \\ R_e &= \frac{V. d}{n. \upsilon} \\ \lambda_d &= \frac{d}{d_s} \\ S_e &= i \end{aligned} \tag{22}$$

where, q_* is the nondimensional sediment transport parameter and R_e is the Reynolds number, we can show that:

$$q_* = k\ (R_e)^{\alpha 1}\ (\lambda_d)^{\alpha 2}\ (S_e)^{\alpha 3} \tag{23}$$

The value of coefficients k, $\alpha 1$, $\alpha 2$ and $\alpha 3$ could be determined using laboratory test results. In Table 3, the requested data to find the relationship for nondimensional sediment transport parameter and in Table 4 the calculated values for the above nondimensional parameters are shown. Also, the estimated values for the coefficients are shown in Table 5.

Hence, we can write Eq. 23 as:

$$q_* = 0.0725\,(R_e)^{-0.3517} . (\lambda_d)^{-0.1346} . (S_e)^{\ 0.258} \tag{24}$$

By using Eq. 14 it is now possible to arrange the hydraulic condition of the rockfill dams so that with respect to the sediment diameter, the hydraulic gradient exceeds from the value of the critical hydraulic gradient and hence the deposition of the sediment within the dam body and consequently pores clogging, can be avoided. Moreover, using Eq. 24 we can estimate the sediment transport capacity and consequently the amount of sediment passing downstream. However, both equations have to be validated. In the following section, the process of the validation is described.

VALIDATION OF THE EQUATIONS

Validation of Eq. 14: To validate Eq. 14 we used laboratory data reported by Samani and Emadi, 2003. They carried out six tests with the bed slope of 0.033, the mean diameter of aggregates of

Table 3: Data obtained from tests to calculate the amounts of nondimensional parameters

Test No.	Diameter of aggregate, d (cm)	Standard deviation SD (cm)	Porosity n (-)	Diameter of sediment, d_s (mm)	Sediment density, ρ_s (kg m^{-3})	Sediment transport discharge, q_s (g/sec/m)	Velocity, V (m sec^{-1})
1	3.0	0.00	0.46	0.36	2640	20.77	0.130
2	3.0	0.00	0.46	0.36	2640	17.39	0.114
3	3.0	0.00	0.46	0.27	2630	22.27	0.125
4	3.0	0.00	0.46	0.27	2630	14.99	0.096
5	3.0	0.00	0.46	0.15	2610	15.69	0.109
6	3.0	0.00	0.46	0.15	2610	15.05	0.113
7	4.5	0.00	0.47	0.36	2640	27.78	0.117
8	4.5	0.00	0.47	0.36	2640	25.61	0.117
9	4.5	0.00	0.47	0.27	2630	26.58	0.126
10	4.5	0.00	0.47	0.27	2630	26.56	0.127
11	4.5	0.00	0.47	0.15	2610	11.04	0.117
12	4.5	0.00	0.47	0.15	2610	11.89	0.122
13	3.0	0.50	0.41	0.36	2640	25.14	0.121
14	3.0	0.50	0.41	0.36	2640	12.86	0.098
15	3.0	0.50	0.41	0.27	2630	10.35	0.090
16	3.0	0.50	0.41	0.27	2630	11.95	0.102
17	3.0	0.50	0.41	0.15	2610	21.30	0.101
18	3.0	0.50	0.41	0.15	2610	22.15	0.097
19	3.0	0.75	0.41	0.36	2640	19.49	0.088
20	3.0	0.75	0.41	0.36	2640	24.99	0.104
21	3.0	0.75	0.41	0.27	2630	24.26	0.093
22	3.0	0.75	0.41	0.27	2630	24.29	0.102
23	3.0	0.75	0.41	0.15	2610	12.18	0.094
24	3.0	0.75	0.41	0.15	2610	11.77	0.092
25	4.5	0.50	0.42	0.36	2640	24.08	0.131
26	4.5	0.50	0.42	0.36	2640	25.77	0.141
27	4.5	0.50	0.42	0.27	2630	25.43	0.109
28	4.5	0.50	0.42	0.27	2630	25.07	0.100
29	4.5	0.50	0.42	0.15	2610	14.84	0.093
30	4.5	0.50	0.42	0.15	2610	22.30	0.116
31	4.5	0.75	0.42	0.36	2640	24.55	0.101
32	4.5	0.75	0.42	0.36	2640	28.68	0.113
33	4.5	0.75	0.42	0.27	2630	27.31	0.112
34	4.5	0.75	0.42	0.27	2630	21.36	0.088
35	4.5	0.75	0.42	0.15	2610	15.58	0.085
36	4.5	0.75	0.42	0.15	2610	16.06	0.100

14.5 and 21 mm and the mean diameter of sediment grain size of 0.256, 0.363 and 0.512 mm. In Table 6 the data used in the experiments and the calculated values for critical hydraulic gradient using Eq. 14, along with the reported values by Samani and Emadi (2003) are shown.

To find the accuracy of Eq. 14, we determined the mean relative error (MRE) using

$$\mathrm{MRE} = \frac{1}{n}\sum_{i=1}^{n}\frac{\left|\mathrm{Observed}_{(i)} - \mathrm{Predicted}_{(i)}\right|}{\mathrm{Observed}_{(i)}} \qquad (25)$$

Table 4: Calculated values of nondimensional parameters

Test No.	Diameter of aggregate, d (cm)	Standard deviation, SD (cm)	Diameter of sediment, d_s (mm)	S_e	λ	R_e	q_*
1	3.0	0.00	0.36	0.1550	83	9149	0.000925
2	3.0	0.00	0.36	0.1517	83	8012	0.000884
3	3.0	0.00	0.27	0.1670	111	8797	0.001035
4	3.0	0.00	0.27	0.1420	111	6743	0.000909
5	3.0	0.00	0.15	0.1337	200	7638	0.000846
6	3.0	0.00	0.15	0.1320	200	7931	0.000782
7	4.5	0.00	0.36	0.1347	125	12074	0.000937
8	4.5	0.00	0.36	0.1260	125	12073	0.000864
9	4.5	0.00	0.27	0.1567	167	12922	0.000841
10	4.5	0.00	0.27	0.1500	167	13062	0.000831
11	4.5	0.00	0.15	0.1143	300	12090	0.000376
12	4.5	0.00	0.15	0.1333	300	12580	0.000389
13	3.0	0.50	0.36	0.1650	83	9538	0.001074
14	3.0	0.50	0.36	0.1027	83	7740	0.000677
15	3.0	0.50	0.27	0.1177	111	7096	0.000596
16	3.0	0.50	0.27	0.1153	111	8004	0.000610
17	3.0	0.50	0.15	0.1453	200	7968	0.001101
18	3.0	0.50	0.15	0.1117	200	7612	0.001199
19	3.0	0.75	0.36	0.1337	83	6892	0.001152
20	3.0	0.75	0.36	0.1900	83	8154	0.001249
21	3.0	0.75	0.27	0.1320	111	7288	0.001361
22	3.0	0.75	0.27	0.1520	111	8037	0.001236
23	3.0	0.75	0.15	0.1407	200	7433	0.000675
24	3.0	0.75	0.15	0.1253	200	7203	0.000673
25	4.5	0.50	0.36	0.2460	125	15061	0.000651
26	4.5	0.50	0.36	0.1463	125	16210	0.000648
27	4.5	0.50	0.27	0.0793	167	12614	0.000824
28	4.5	0.50	0.27	0.0973	167	11521	0.000890
29	4.5	0.50	0.15	0.0850	300	10764	0.000568
30	4.5	0.50	0.15	0.0883	300	13362	0.000688
31	4.5	0.75	0.36	0.1100	125	11636	0.000859
32	4.5	0.75	0.36	0.1493	125	13030	0.000897
33	4.5	0.75	0.27	0.1450	167	12917	0.000865
34	4.5	0.75	0.27	0.0897	167	10137	0.000861
35	4.5	0.75	0.15	0.0650	300	9791	0.000656
36	4.5	0.75	0.15	0.0840	300	11479	0.000576

Table 5: Estimated values for coefficients in Eq. 23

Parameters	Estimated values
k	0.0725
$\alpha 1$	-0.3517
$\alpha 2$	-0.1346
$\alpha 3$	0.2580

where, n is the number of tests and $\text{Observed}_{(i)}$ and $\text{Predicted}_{(i)}$ are the observed and predicted values of the parameters of interest, respectively. Using Eq. 14 we found MRE equal to 0.4% indicated that a very good agreement exists between the results of Eq. 14 and the values reported

Table 6: The amount of critical hydraulic gradient obtained by Emadi and Samani (2003) and Eq. 14

No.	Mean diameter of aggregates (mm)	Mean diameter of sediments (mm)	Sediment repose angle (degrees)	Critical hydraulic gradient: Emadi and Samani (2003)	Critical hydraulic gradient: Eq. 14
1	21.0	0.512	45	0.1790	0.1833
2	14.5	0.512	55	0.2860	0.2856
3	21.0	0.363	35	0.0645	0.0670
4	14.5	0.363	42	0.1016	0.1166
5	21.0	0.256	30	0.0336	0.0352
6	14.5	0.256	36	0.0646	0.0657

Table 7: Comparison between observed versus calculated values for nondimensional sediment transport capacity

No.	Diameter of aggregate particles (cm)	SD (cm)	Diameter of sediment particles (cm)	Non dimensional sediment transport discharge: Laboratory results	Non dimensional sediment transport discharge: Eq. 24
1	0.36	0	3.0	0.000979	0.000925
2	0.36	0	3.0	0.000982	0.000884
3	0.27	0	3.0	0.000993	0.001035
4	0.27	0	3.0	0.000982	0.000909
5	0.15	0	3.0	0.000911	0.000846
6	0.15	0	3.0	0.000867	0.000782
7	0.36	0	4.5	0.000880	0.000937
8	0.36	0	4.5	0.000813	0.000864
9	0.27	0	4.5	0.000808	0.000841
10	0.27	0	4.5	0.000796	0.000831
11	0.15	0	4.5	0.000432	0.000376
12	0.15	0	4.5	0.000490	0.000389

by Samani and Emadi (2003). So, we can use Eq. 14 to estimate critical hydraulic gradient with reasonable accuracy.

Moreover, we calculated the critical hydraulic gradient using the relationship of Samani and Emadi (2003) and the new relationship. We then compared those results with data provided from our laboratory tests. Using Eq. 25 we find MRE values of 127 and 2.7% for Samani and Emadi (2003) and the new relationship, respectively, indicates the better estimation of critical hydraulic gradient by the new relationship.

Validation of Eq. 24: To obtain Eq. 24, we used the results of twenty four experiments in which we used aggregated with the mean diameters of 3 and 4.5 cm, standard deviation of 0.5 and 0.75 cm and sediment particles with the mean diameter of 0.15, 0.27 and 0.36 mm. Also, the Reynolds Number differed from 6743 to 16210. To validate Eq. 24, we used the results of the other twelve experiments in which the mean diameter of the aggregates and sediments particles were the same but the standard deviation of the aggregate particles was 0.0. Table 7 indicates the input data to estimate the amount of the nondimensional sediment transport parameter, q_* and the values of q_* obtained from laboratory tests and Eq. 24.

Similar to Eq. 14, we calculated the value of MRE for Eq. 24. We found MRE equal to 9.04% indicated that Eq. 24 predicts the amount of q_* well.

We again calculated the nondimensional sediment transport parameter using the relationship of Samani and Emadi (2003) and Joy *et al.* (1991) and the new relationship. Having compared those results with data provided from our laboratory tests and using Eq. 25, we find MRE values of 448.1, 92.7 and 5.4% for Samani and Emadi (2003) and Joy *et al.* (1991) and the new relationship, respectively. The above values of MRE indicate a very good agreement between the observation and estimated values of nondimensional sediment transport parameter by the new relationship comparing with the other available models.

Applications of Eq. 24: In Fig. 6 the variation of sediment transport discharge versus the ratio of aggregate to sediment diameter (λ) for three different values of standard deviation of the aggregates are shown.

Figure 6 shows that for a specific discharge and standard deviation, increasing the amount of λ, results in reduction of sediment transport discharge. This is because of by increasing the pores diameter of aggregates, velocity decreases and consequently, the water fluctuation inside aggregate media is reduced so that a more suitable condition for sediment settlement is obtained. Furthermore, Fig. 6 indicates that for a constant discharge, increasing the amount of standard deviation results in a decrease in the sediment transport discharge. For a fixed value of standard deviation of the aggregates, increasing flow discharge causes a considerable increase in the sediment transport discharge.

In Fig. 7 the variation of sediment transport discharge versus hydraulic gradient of the flow is shown. As shown in Fig. 7, increasing the amount of hydraulic gradient, results in increasing the

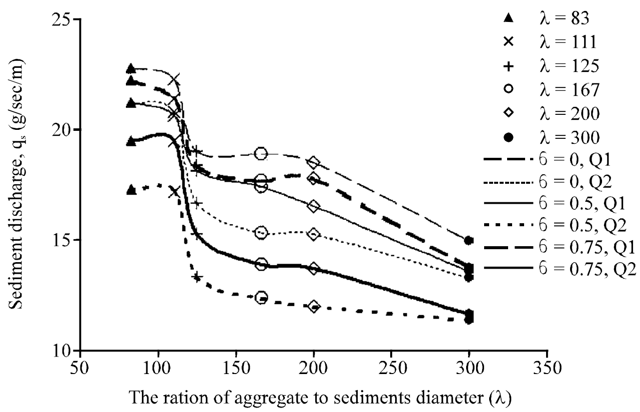

Fig. 6: The variation of sediment transport discharge versus the ratio of aggregate to sediment diameter (λ)

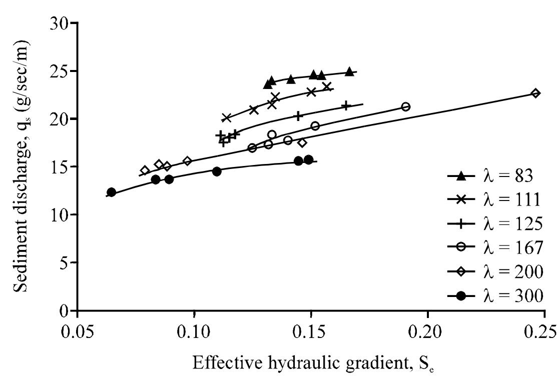

Fig. 7: Variation of sediment discharge versus effective hydraulic gradient of the flow

amount of particle transport force and consequently, the sediment transport discharge is increased. Moreover, Fig. 7 shows that for each value of λ, a certain range of hydraulic gradient (around $S_e = 0.17$) is exist in which the sediment transport capacity reached its maximum level. For hydraulic gradients more than this value, sediment transport capacity remains constant.

CONCLUSION

Using a set of laboratory tests on a rectangular porous aggregates, we investigated the critical hydraulic gradient of noncohesive sediment materials in turbulent flows. Carrying out eighteen tests, we used aggregates of maximum diameter of 4.5 cm with three different standard deviation and also sediment particles of maximum diameter of 0.36 mm, when the Reynolds number varies from 3174 to 9667. The results showed that the minimum hydraulic gradient to prevent sedimentation of suspended materials inside the pores of the aggregates is a function of bed slope (θ), the repose angle of sediments inside aggregate porous media (ϕ), the mean diameter of sediment particles (d_s) and the relative density of the sediment particles (G_s), as shown in Eq. 14. Using available laboratory data, we found a very good agreement (MRE = 0.4%) between the predicted and observed values of critical hydraulic gradient. To find a relationship for calculating the nondimensional sediment discharge capacity, we carried out thirty six laboratory tests using the above materials for Reynolds number varied between 6743 to 16210. We used the results of twenty four tests to establish Eq. 24 and then we used the results of twelve other tests to investigate the validity of this equation. We found a good accuracy for the equation (MRE = 9.4%) indicating that Eq. 24 predicts the nondimensional sediment transport capacity well.

ACKNOWLEDGMENTS

Laboratory tests in this research were carried out at the Research Institute for Water Scarcity and Drought. The authors would like to express their sincere thanks for this collaboration. This research was financially supported by the Iran National Science Foundation, Presidency, I.R. Iran, through contract No. 84115/30. The second author would like to express his sincere thanks for this support.

REFERENCES

Cunningham, A.B., C.J. Anderson and H. Bower, 1987. Effects of sediment-laden flow on channel bed clogging. J. Irrigation Drainage Eng., 113: 106-118.

Emadi, A., J. Samani and K. Ghaderi, 2005. Determination of noncohesive sediment capacity in Rockfill dams. Proceedings of the 5th Iranian Hydraulic Conference, Nov. 8-10, Kerman, Iran, pp: 1-4.

Joy, D.M., W.C. Lennox and N. Kouwen, 1991. Particle transport in porous media under non-linear flow condition. J. Hydraulic Res., 29: 373-383.

Li, B., V.K. Garga and M.H. Davis, 1998. Relationship for non-darcy flow in rock-fill. J. Hydraulic Eng., 124: 206-212.

Moradlo, J., 1999. Study of physical clogging of porous media in tailing dams. M.Sc. Thesis, Tarbiat Modares University

Saktivadivel, R., 1972. Sediment Transport Through a Porous Column. In: Sedimentation, Shen, H.W. (Eds.). Colorado State University, Fort Collins, USA.

Samani, J. and A. Emadi, 2003. Introducing the relationship of critical hydraulic gradient to noncohesive sediment transport in Rockfill dams. Proceedings of the 4th Iranian Hydraulic Conference, Oct. 21-23, Shiraz, Iran, pp: 4-7.

Schalchi, U., 1995. Basic equation for siltation of riverbed. J. Hydraulic Eng., 121: 274-287.