STONE STABILITY UNDER DECELERATING OPEN-CHANNEL FLOW

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The current research is aimed at finding a proper relation between flow forces acting on the bed and the bed response. To this end, experiments were carried out in which both the bed response (quantified by a dimensionless entrainment rate) and the flow field (velocity and turbulence intensity distributions) are measured. The three available stability parameters, which are used to quantify for the flow forces, were evaluated using the measured data. The focus of the evaluation is on the correlation of these stability parameters with the measured bed damage expressed in terms of the dimensionless entrainment rate. The experimental results confirm that the Shields stability parameter fails to predict bed damage for non-uniform flow conditions ($R^2=0.18$). In contrast, the stability parameters of Jongeling et al. (2003) and Hofland (2005) give better damage predictions ($R^2 = 0.77$). The results confirm the strong influence of the velocity and turbulence intensity distributions on the stability of bed material.

INTRODUCTION

In the design of bed protections the choice of stone sizes and weights to be used is essential. The Shields (1936) formula was developed for uniform flow conditions and is widely used to determine the required stone sizes. In the Shields formula the near-bed shear stress is the only quantity representing the flow forces on the bed. Previous studies (Jongeling et al, 2003; Hofland, 2005 and Mosselman & Akkerman, 1998, among others) have shown that not only the near-bed shear stress (or an estimate thereof based on the mean velocity) but also the turbulence influences the stability of the bed material in flowing water. Therefore, it is important to take turbulence effects into account, especially for the design of bed protections in non-uniform flow near hydraulic structures.

GOVERNING VARIABLES

A relation between flow forces (Ψ) acting on the bed and bed response (Φ) can be expressed as follows

$$\Phi = f(\Psi) \quad (1)$$

This cause-and-effect relation can be established if the flow forces and the bed response are properly described. In this section those governing variables will be discussed in details.

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Flow forces

A stability parameter - expressed in the form of a dimensionless relation between hydraulic load and bed strength - is often used to quantify the influence of hydraulic forces on the bed. As the flow forces acting to move the stone (i.e., drag, shear and lift forces) are proportional to $(\propto \rho u^2 d^2)$ and the force that resists movement is mainly the gravity force $(\equiv (\rho_s - \rho)gV)$, a general form of a stability parameter can be expressed as

$$\Psi = \frac{u^2}{\Delta g d} \quad (2)$$

where Ψ denotes a general mobility parameter, u is a typical velocity scale, Δ represents the specific submerged density of stone $(\equiv (\rho_s - \rho) / \rho)$, ρ_s represents the density of stone, ρ represents the density of water, g represents the gravitational acceleration, $V(\propto d^3)$ represents the volume of the stone and d denotes the stone diameter. For uniform flow Shields (1936) used the shear velocity u_* to form the well-known Shields stability (mobility) parameter Ψ_s :

$$\Psi_s = \frac{u_*^2}{\Delta g d} = \frac{\tau_b}{\rho \Delta g d} \quad (3)$$

where τ_b denotes the near bed shear stress ($\tau_b = \rho u_*^2$). If Ψ_s is larger than a critical value Ψ_c , movement of bed material will occur. In the Shields parameter the near-bed shear stress is the only quantity representing the flow forces on the bed. Turbulence effects are only incorporated implicitly through the empirically estimated values of Ψ_c . This is a valid approach for uniform flows, for which the ratio of turbulence intensity to u_* is virtually constant (Hofland, 2005). In non-uniform flows, Jongeling et al. (2003) proposed a stability parameter as follow

$$\Psi_{WL} = \frac{\left\langle \left(\bar{u} + \alpha \sqrt{k} \right)^2 \right\rangle_{hm}}{\Delta g d} \quad (4)$$

where $\bar{u}$ denotes the mean velocity, α is an empirical parameter, k represents the turbulence kinetic energy and $\langle \dots \rangle_{hm}$ is a spatial average over a distance of hm above the bed. The values of α and hm were chosen so that at incipient motion of stones the stability parameter Ψ_{WL} had more or less an equal value for all considered geometries. From the flow simulation it appeared that the variation of the stability parameter Ψ_{WL} is smallest when a value $\alpha = 6$ is selected (Jongeling et al, 2006). A value of hm is chosen as

$$hm = 5d_n + 0.2h \quad (5)$$

where d_n is the nominal stone diameter and h is the local water depth.

Partly based on the approaches developed by Mosselman and Akkerman (1998), Hoffmans and Akkerman (1998) and Jongeling et al (2003), Hofland (2005) proposed a new method for evaluating the stability of bed protections under non-uniform flows using output of a 3D RANS model. The profiles of the mean velocity and turbulence kinetic energy in the water column above the bed are used to formulate a local stability parameter. The maximum over the depth of the local values of $(\bar{u} + \alpha\sqrt{k})$ weighted with the relative mixing length L_m/z is used. The stability parameter, Ψ_{L_m} , is expressed as

$$\Psi_{L_m} = \frac{\max \left[\left(\bar{u} + \alpha\sqrt{k} \right)_{L_m} \frac{L_m}{z} \right]^2}{\Delta g d} \quad (6)$$

where L_m denotes the Bakhmetev mixing length ($L_m = \kappa z \sqrt{1 - z/h}$), $\langle \dots \rangle_{L_m}$ is a moving average with varying filter length L_m , and z is the distance from the bed. A correlation between the new mobility parameter and the bed damage was analyzed based on the data of Jongeling (2003) and De Gunst (1999). From the analysis $\alpha = 6$ yielded the best collapse of data.

Bed response

Mosselman and Akkerman (1998) distinguish two ways of defining the mobility of particles: i) the number of pick-ups (n) per unit time (T) and area (A) or ii) the number of particles that is transported through a cross-section per unit time. The former if expressed in terms of volume of entrainment is often called (volume) entrainment rate, $E = nd^3/AT$. The latter is often called bed load transport, q_s . The entrainment is linked to the bed load transport by: $q_s = El_s$ where l_s is the displacement length. Both the entrainment rate and bed load transport can be used as bed damage indicators and are often expressed in dimensionless form as

$$\Phi_E = E / \Delta g d \quad \text{and} \quad \Phi_s = q_s / \sqrt{\Delta g d^3} \quad (7)$$

The use of (dimensionless) bed load transport as a bed damage indicator is conventional for uniform flow. Paintal (1971) found a strong dependence of Φ_s on Ψ_s :

$$\Phi_s = 6.56 \times 10^{18} \Psi_s^{16} \quad \text{for} \quad 0.02 < \Psi_s < 0.05 \quad (8)$$

However, bed load transport is dependent on the hydraulics upstream; all the stones passing a certain cross section (i.e., the transport) have been entrained upstream of this section. Bed load transport is therefore considered as non-local parameter. In all cases, stability parameters are local parameters, making Eq. (1) a relation of local and non-local parameters. Such kind of relation can only be valid for uniform flow where along the channel the flow condition is unchanged.

For non-uniform flow Hofland (2005) points out that Φ_E should be used as bed damage indicator because it is completely dependent on the local hydrodynamic parameters. Figure 1 shows the relations between the measured Φ_E , Ψ_{WL} and Ψ_{Lm} .

Figure 1. Top: measured Φ_E versus measured Ψ_{WL} . Bottom: measured Φ_E versus measured Ψ_{Lm} (Hofland, 2005). The curve is a tentative power law drawn more or less as an upper envelope of the data points.

Concluding remarks and objectives

From the previous review, the following conclusions can be drawn: i) for the bed damage indicator, the use of bed load transport is only suitable for uniform flow where the flow is unchanged along the channel. For non-uniform flow the dimensionless entrainment rate should be used; ii) for the stability parameters, in non-uniform flow the use of the Shields stability parameter is not sufficient as the turbulence effect is not taken into account. Two potential alternatives are the Jongeling et al parameter, Ψ_{WL} , and the Hofland parameter, Ψ_{Lm} . These two stability parameters need to be verified because they were developed based on the limited data set and a large scatter is present in the $\Phi_E - \Psi_{WL}$ and $\Phi_E - \Psi_{Lm}$ relations.

This research focuses on the evaluation of the three aforementioned stability parameters and on the formulation of relation (1) for non-uniform flow based on an experimental study.

EXPERIMENTS

Geometry

The experiment was carried out in a laboratory open-channel flume with an effective length of 13.30 m and an available width of 0.495 m. To decelerate the flow, an expansion is made near the end of the flume. To this end, the first part of the flume was narrowed at both sides. Then the extension was made by gradually increasing the width from the first segment to the width of the flume. By changing the expansion length (expansion angle), different combinations of velocity and turbulence can be obtained. Three different set-ups with expansion angles of 3, 5 and 7 degrees were built.

Figure 2. The first experimental set-up with indication of the placement of uniformly colored artificial stone strips (not to scale)

Figure 2 shows a schematic representation of the first experimental set-up. Natural stones having a nominal diameter d_{n50} of 0.80 cm and d_{n85}/d_{n15} of 1.27 were used to create a 4-cm-thick rough bottom. These stones are practically unmovable under the experimental flow conditions. To examine the stone stability, two-layer uniformly colored strips of artificial light stones were placed at the designated locations (before and along the expansion, see Figures 2 and 3). These stones are made of epoxy resin having densities in the range of 1320 to 2023 kg/m³, mimicking shapes and sizes of natural stones. The artificial stones have a nominal diameter d_{n50} of 0.82 cm and d_{n85}/d_{n15} of 1.11.

Instrumentation

A 2-component, LDV-system was used, measuring u - and w - components of the velocity in the streamwise vertical plane. As a light source, a Helium-Neon (HeNe) laser with a power of 15mW was used. The LDV uses the forward-

scatter, reference-beam method. In the present study, a 400 mm lens was used, resulting in a measuring volume with dimensions of about 10 mm in spanwise direction and 1 mm in the other directions. Each measurement lasted 2 minutes with a sampling frequency of 500 Hz.

Figure 3. Longitudinal sections of the three experimental set-ups. Hatched areas depict the expansion regions. Dashed lines are stone entrainment-measurement locations.

As the flow structures in the present study are different from standard uniform open-channel flow, it is necessary to check the reliability of the LDV measurement. To this end, a measurement of a nearly-uniform open-channel flow on a smooth bed was conducted. The flow has a discharge of 12 l/s, a water depth of 5.9 cm and $B/h = 5.9$. The results are depicted in Figure 4, showing that the data agree well with the literature.

Figure 4. Distributions of velocity and Reynolds stress.

The water depth could be controlled by means of an adjustable gate at the end of the flume. The discharge could be regulated using an orifice plate in the water supply pipe. By measuring the difference in water pressure before and after the orifice plate, the discharge was determined. This pressure difference is expressed in the form of a water column height difference. Due to the pressure fluctuations, the water column height measurement has an accuracy of about 0.5 cm. This means an accuracy of approximately 0.2 l/s. The water levels were

measured by a needle. The needle has an accuracy of 0.1 mm. However, due to the presence of small surface waves in the flume the measurement accuracy is only about 2 mm.

Hydraulic conditions and experimental procedures

The experiments consisted of 37 series of measurements. For the first 36 series, the artificial stones having the density from 1320 to 1384 kg/m³ were used, whereas in the last series, the artificial stones with the density of 2023 kg/m³ were used. The last series was dedicated to check for the influence of stone density. Of the first 36 series, twelve with different flow conditions (from A to L) were conducted for each set-up (3 set-ups). The hydraulic conditions are summarized in Table 1. The locations of the test sections are depicted in Figure 3, namely profile 1 to profile 4. In Table 1, Q is the discharge, h the water depth, $Re = Uh/\nu$ the Reynolds number, U mean bulk velocity, ν the kinematic viscosity and $F = U/\sqrt{gh}$ the Froude number. Series names consist of the set-up number, the flow condition and the type of flume bottom. Series 1AR, for example, indicates the experiment in set-up 1 with flow condition A ($Q = 22.0$ l/s, $h \approx 12$ cm) and a rough flume bottom (R stands for rough, S stands for smooth). The twelve flow conditions are denoted as A to L (with the water depth increasing from approx. 12 cm to approx. 19 cm). Similarly, profile names consist of the series name and a profile number, i.e., 1AR2 is the name for profile 2 of series '1AR'.

Each series consists of five repetitive measurements with the same flow conditions. The experimental procedure was as follows. First, the desired discharge was generated and the desired water depth was obtained by adjusting the weir at the downstream end of the flume. The water level and velocity measurements were carried out in the first test where the whole flume bottom was covered by natural stones. The same flow condition was reproduced and repeated for the next four tests to measure stone entrainment. To this end, the uniformly colored strips of light artificial stones were placed at the designated locations. A 30-minute initial settling period was applied prior to the actual test to remove loose stones that do not determine the strength of the bed. To start an entrainment test the flume was flooded slowly to the designated condition. After two hours, the flow was stopped and the number of displaced stones was registered. The entrainment rates obtained from the four tests are averaged to get a statistically reliable entrainment rate for the series.

Table 1. Summary of Hydraulic conditions

	α [degree]	Q [l/s]	profile 1			profile 2			profile 3			profile 4		
			h [cm]	Re [10 ⁴]	Fr [-]	h [m]	Re [10 ⁴]	Fr [-]	h [m]	Re [10 ⁴]	Fr [-]	h [m]	Re [10 ⁴]	Fr [-]
1AR	3	22	11.7	6.2	0.498	12.1	5.5	0.423	12.1	5.2	0.394	12.3	4.9	0.362
1BR		20	12	5.7	0.439	12.1	5	0.385	12.2	4.7	0.357	12.3	4.4	0.331
1CR		23	13	6.5	0.448	13	5.7	0.396	13.2	5.4	0.363	13.3	5.1	0.338
1DR		26.5	13.9	7.5	0.466	14.3	6.6	0.395	14.5	6.2	0.363	14.5	5.9	0.343
1ER		24	13.9	6.8	0.422	13.9	6	0.371	14.1	5.6	0.344	14.3	5.3	0.316
1FR		27	15	7.6	0.425	14.8	6.7	0.381	15.2	6.3	0.346	15.3	6	0.323
1GR		31	15.7	8.8	0.456	16.1	7.7	0.385	16.2	7.3	0.361	16.2	6.9	0.338
1HR		28	15.8	7.9	0.407	15.9	7	0.355	16.2	6.6	0.324	16.5	6.2	0.299
1IR		31.5	17	8.9	0.412	17.1	7.9	0.359	17.4	7.4	0.329	17.8	7	0.3
1JR		35.5	17.9	10	0.428	18.1	8.9	0.372	18.3	8.3	0.343	18.5	7.8	0.318
1KR		32	18	9.1	0.383	18.1	8	0.333	18.3	7.5	0.308	18.5	7.1	0.287
1LR		35.5	19	10	0.391	19.1	8.9	0.343	19.1	8.3	0.321	19.3	7.8	0.298
2AR	5	22	11.6	6.2	0.507	11.6	5.6	0.459	11.6	5.1	0.419	11.8	4.7	0.379
2BR		20	12	5.7	0.442	11.9	5.1	0.401	11.8	4.7	0.373	11.8	4.3	0.341
2CR		23	12.8	6.5	0.459	12.6	5.8	0.42	12.7	5.4	0.382	12.9	5	0.345
2DR		26.5	13.8	7.5	0.471	13.8	6.7	0.424	13.7	6.2	0.391	13.9	5.7	0.355
2ER		24	13.2	6.8	0.458	13.2	6.1	0.409	13.3	5.6	0.373	13.3	5.2	0.343
2FR		27	14.4	7.6	0.448	14.1	6.9	0.418	14.2	6.3	0.379	14.3	5.8	0.345
2GR		31	16	8.8	0.443	15.9	7.9	0.402	15.9	7.2	0.368	16.1	6.7	0.335
2HR		28	15.9	7.9	0.404	15.8	7.1	0.367	15.9	6.5	0.332	15.9	6	0.308
2IR		31.5	16.9	8.9	0.413	16.7	8	0.379	16.6	7.3	0.35	16.8	6.8	0.318
2JR		35.5	17.5	10	0.442	18	9	0.382	17.9	8.3	0.351	18.1	7.6	0.32
2KR		32	17.8	9.1	0.39	17.9	8.1	0.347	17.8	7.5	0.32	17.9	6.9	0.293
2LR		35.5	18.6	10	0.405	18.2	9	0.375	18.5	8.3	0.335	18.6	7.6	0.306
3AR	7	22	12.1	6.2	0.474	12.6	5.4	0.393	12.8	4.8	0.343	-	-	-
3BR		20	12	5.7	0.438	12.7	5	0.351	13	4.4	0.303	-	-	-
3CR		23	12.9	6.5	0.454	13.3	5.7	0.379	13.4	5.1	0.333	-	-	-
3DR		26.5	13.8	7.5	0.474	14.4	6.6	0.387	14.7	5.8	0.332	-	-	-
3ER		24	14.1	6.8	0.411	14.7	5.9	0.34	15	5.3	0.292	-	-	-
3FR		27	14.9	7.6	0.428	15.5	6.7	0.355	15.6	5.9	0.31	-	-	-
3GR		31	15.7	8.8	0.456	16.4	7.7	0.373	16.8	6.8	0.319	-	-	-
3HR		28	15.8	7.9	0.406	16.4	6.9	0.336	16.6	6.2	0.294	-	-	-
3IR		31.5	16.9	8.9	0.412	17.8	7.8	0.335	17.5	6.9	0.305	-	-	-
3JR		35.5	17.5	10	0.442	18	8.8	0.37	18.5	7.8	0.316	-	-	-
3KR		32	17.7	9.1	0.391	18	7.9	0.335	18.2	7	0.292	-	-	-
3LR		35.5	18.3	10	0.414	19.2	8.8	0.336	19.5	7.8	0.293	-	-	-
3MR		36	12.4	10.2	0.752	11.8	8.9	0.71	12.7	7.9	0.562	-	-	-

RESULTS

A detailed analysis of the flow data in Hoan et al, (2007) has shown that the turbulence intensity distributions deviate from the empirical curve reported for uniform flow. The flow is non-equilibrium and the ratio of turbulence intensities to the shear velocity cannot be expressed by any universal function. Therefore, the turbulence effect on the stability of stone should be modeled explicitly. In this paper, the correlation between the available mobility parameters and the measured entrainment rate was analyzed.

Relation between Ψ_s and Φ_E

The shear velocity is needed to calculate the measured Shields stability parameter (Eq. (3)). This can be determined in the three following ways: i) u_{*1} from the Reynolds stress distribution; ii) u_{*2} from the log-law applied to the flow near the bottom, and iii) u_{*3} from the water level slope, i.e., $u_{*3} = \sqrt{ghi}$. Of the three methods the last one is considered less reliable because of the uncertainty in the accuracy of the water level measurement and the unknown sidewall influence. Because of the presence of small surface waves, this third method will not be used. The values of u_{*1} and u_{*2} for all flow conditions are presented in Hoan et al (2007), showing that u_{*1} and u_{*2} are in fairly good agreement. Correlation analysis between Ψ_s and Φ_E shows less scatter when u_{*1} is used.

Table 2 summarizes the correlation analysis between the measured Shields stability parameter and the measured dimensionless entrainment rate for set-up 1, 2, 3 and all set-ups. The results show that the correlation become less when the expansion angle is larger (i.e., the flow is more non-uniform). Figure 5a shows the correlation between the Shields stability parameter and the dimensionless entrainment rate for all data. The coefficient of determination $R^2 = 0.18$, showing that virtually no correlation is found to exist between Ψ_s and Φ_E for non-uniform flow. This clearly shows that the near bed shear stress only is not sufficient to quantify for the flow forces acting on the bed.

Table 2. Coefficient of determination

	set-up			
	1	2	3	1+2+3
Expansion angle, α	3°	5°	7°	
No of data	48	60	60	168
R^2	0.48	0.21	0.08	0.18

Figure 5. a) [left] measured Ψ_s versus measured Φ_E . b) [right] Sensitivity analysis of α in Eqs. (4) and (6).

Relation between Ψ_{WL} , Ψ_{Lm} and Φ_E

As only two velocity components (u - streamwise and w - upward) are available, the turbulence kinetic energy in Eqs (4) and (6) was approximated by assuming that $\sigma(v) = \sigma(u)/1.9$. The approximation is based on the EMS measurement of the flow conditions where both u - and v - velocity components were measured. The results show that, in contrast to the Shields stability parameter, the Jongeling et al. and Hofland stability parameters are strongly correlated to the measured dimensionless entrainment parameter:

$$\Phi_E = 5.4 \times 10^{-14} \Psi_{WL}^{4.9} \quad \text{for } 18 < \Psi_{WL} < 40 \quad (R^2 = 0.78, \alpha = 6) \quad (9)$$

$$\Phi_E = 1.2 \times 10^{-9} \Psi_{Lm}^{4.5} \quad \text{for } 2.5 < \Psi_{Lm} < 5.5 \quad (R^2 = 0.77, \alpha = 6) \quad (10)$$

In Eqs (9) and (10) the turbulence magnification factor α of 6 was used as suggested by Jongeling et al (2003). This was chosen based on the smallest variation of the stability parameter Ψ_{WL} at the incipient motion state. In this paper we will use another criterion to choose the value of α . The value of α that give the best correlation between the stability parameters and the dimensionless entrainment rate will be chosen. This sensitivity analysis of α in Ψ_{WL} and Ψ_{Lm} is shown in Figure 5b. It turns out that $\alpha = 3.5$ (for Ψ_{WL}) and $\alpha = 3.0$ (for Ψ_{Lm}) give the best correlation ($R^2 \approx 0.81$ for both). Figure 6 shows the correlation between these two stability parameters and the dimensionless entrainment rate. The results also confirm that the influence of stone density is well incorporated in the formula. The relations between the two modified stability parameters and the entrainment rate are expressed as follow:

$$\Phi_E = 1.16 \times 10^{-12} \Psi_{WL}^{4.57} \quad \text{for } 10 < \Psi_{WL} < 25 \quad (R^2 = 0.82, \alpha = 3.5) \quad (11)$$

Figure 6. Left: measured Ψ_{WL} versus measured Φ_E . Right: measured Ψ_{Lm} versus measured Φ_E .

$$\Phi_E = 1.9 \times 10^{-8} \Psi_{Lm}^{4.32} \quad \text{for} \quad 1.3 < \Psi_{Lm} < 3.2 \quad (R^2 = 0.81, \alpha = 3) \quad (12)$$

CONCLUSIONS

1. The experiments reported herein indicate that the Shields stability parameter is not sufficient for presenting the flow forces acting on the bed in non-uniform flow.
2. Jongeling et al (Ψ_{WL}) and Hofland (Ψ_{Lm}) stability parameters properly represent the flow forces on the bed and these parameters can be successfully used to predict bed damage, i.e., using Eqs (11) and (12). $\alpha = 3.5$ (for Ψ_{WL}) and $\alpha = 3.0$ (for Ψ_{Lm}) give the best correlation to the measured entrainment rate.
3. Based on these experiments, no preference for Ψ_{WL} or Ψ_{Lm} can be found.
4. The results confirm the strong influence of the velocity and turbulence intensity distributions on the stability of bed material.

REFERENCES

- De Gunst, M. (1999). *Stone stability in a turbulent flow behind a step*. M.Sc. thesis, Delft University of Technology, Delft.
- Hoan, N. T., Booij, R., Stive, M. J., Verhagen, H. J. (2007). Decelerating open-channel flow in a gradual expansion. In *Asian and Pacific Coasts 2007*. Nanjing, China.
- Hoffmans, G. J. C. M., & Akkerman, G. (1998). Influence of turbulence on stone stability. In *7th international conference on River Sedimentation*.

- Hofland, B. (2005). *Rock & roll: turbulence-induced damage to granular bed protections*. Ph.D. thesis, Delft University of Technology.
- Jongeling, T.H.G., Blom, A., Jagers, H.R.A., Stolker, C., Verheij, H.J. (2003). *Design method granular protections* (Technical Report Q2933 / Q3018). WL|Delft Hydraulics. (In Dutch)
- Jongeling, T.H.G., Jagers, H.R.A., Stolker, C., & Verheij, H.J. (2006). *Design of granular bed protections using a RANS 3D-flow model*. The Third International Conference on Scour and Erosion, Amsterdam, The Netherlands.
- Mosselman, E., Akkerman, G. J. (1998, October). *Low-mobility transport of coarse grained material*. (Tech. Rep. No. Q2395.40). WL|Delft Hydraulics.
- Paintal, A. S. (1971). Concept of critical shear stress in loose boundary open channels. *Journal of Hydraulic Research*, 9(3), 91-113.
- Shields, A. (1936). *Anwendung der Aehnlichkeitsmechanik und der Turbulenzforschung auf die Geschiebebewegung*. Mitteilungen der Preussischen Versuchsanstalt für Wasserbau und Schiffbau, Heft 26, Berlin. (in German)

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Bed damage

Stone entrainment

Bed load transport

Turbulence

Decelerating flow

Open-channel flow