Assessment of Scour Potential of a Circular Pier in Silty Sand Using ISEEP

Caractérisation par ISEEP du potentiel d'érosion d'une pile circulaire dans un sable silteux

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ABSTRACT: Work in this paper describes an approach for the assessment of soil scour potential through the use of an In Situ Erosion Evaluation Probe (ISEEP) that is advanced by water jetting. Soil erosion parameters are assessed for silty sand in terms of critical stream power, and therefore, critical shear stress, and detachment rate coefficient. Scour depth around a circular bridge pier was computed using ISEEP data and compared with an empirical approach available in literature for estimating scour depth in soil similar to the tested in the study. The application of the idea and the utility of this technique to assess scourability profile are presented and discussed.

RÉSUMÉ: Le travail présenté dans cet article décrit une approche pour l'évaluation du risque d'affouillement d'un sol in situ en utilisant une sonde d'érosion équipée de jet d'eau. Les paramètres d'érosion sont évalués pour les sables limoneux en fonction d'une puissance critique et par conséquent en termes d'une contrainte de cisaillement critique et d'un coefficient exprimant le taux de détachement. La profondeur d'affouillement autour d'une pile de pont a été calculée en utilisant les données issues de la sonde. Elle a été comparée à celle issue de l'approche empirique pour un sol similaire au sol étudié. L'applicabilité de l'approche proposée et son utilité pour l'évaluation du profil d'érodabilité sont présentées et discutées.

KEYWORDS: Bridges, Erosion, Foundation, In Situ, Pier, Probe, Scour, Shear, Soil

1 INTRODUCTION

The assessment of scour and erosion rates of soil profiles supporting hydraulic structures and critical bridges is vital for ensuring safe performance under normal flow conditions, as well as the integrity of their foundation systems during and after severe storms. Richardson and Davis (2001) highlighted the importance of assessment of local scour around bridge piers as it is one of the most common causes of bridge failure. Several approaches ranging from simple steel sounding rods to remote sensing have been developed to assess scour depth after it has occurred. As presented by Lu et al. (2008) the more sophisticated approaches, including acoustic doppler and ground penetrations radars, have a high cost and require frequent maintenance and repair. Even then, these approaches do not provide an estimate of scour under future storm events. Current techniques for providing such information require either the removal of soil samples for laboratory testing, in a device such as the Erosion Function Apparatus (EFA) by Briaud et al. (2001), or limiting the measurements to erodibility of the surface sediments.

Gabr et al. (2012) presented a prototype device, termed ISEEP (In Situ Erosion Evaluation Probe), for assessment of scour parameters with depth. ISEEP has been constructed as simple stainless steel tubes fitted with truncated cone tip. The cone-tipped vertical probe is attached to a digitally controlled centrifugal pump that provides controllable and repeatable water velocity at the tip, with sustained flow rate against any induced back pressure. As the water jet is induced through the cone tip, it mobilizes the soil particles. The test data are analyzed using the stream power (bed shear stress multiplied by the flow velocity) concept proposed by Annandale (2006) to account for the nature of the flow conditions induced during testing. The results from the tests are reduced to provide critical shear stress (τ_c) and a rate of scour per unit shear stress (k_d). These two values are used in conjunction with the applied shear stresses ($\tau_{applied}$) per a given flow type and as appropriate to the structure being analyzed, to compute the scour rate (E) using the excess shear model as follows (Annandale 2006):

$$E = k_d\,(\tau_{applied} - \tau_c) \qquad (1)$$

In this study, experimental work and analyses are conducted, using ISEEP-estimated data, for evaluating erosion parameters for a soil with 15% clay and 85% sand. The soil is classified as silty sand according to the Unified Soil Classification System. Tests are performed with different jet velocities and critical stream power value (P_c) and the corresponding τ_c, and k_d are evaluated using the data reduction scheme proposed by Gabr et al. (2012). An example showing the computation of the scour depth around a bridge pier using ISEEP-estimated data is presented. The results are compared with values obtained using empirical equations reported in literature, and the estimated scour depth using both approaches is presented and discussed.

2 BRIDGE PIER SCOUR

The magnitude and geometry of local scour at bridge piers in soil profiles with percent fines content have been documented in literature (e.g. Hosny 1995, Molinas and Hosny 1999, Briaud et al. 1999, 2001, and 2004). Hosny (1995), and Molinas and Hosny (1999) proposed empirical equations to assess scour depth for saturated and unsaturated compacted soil with a percent fines that lends a degree of cohesion to the soil. They reported that the scour depth decreased as compaction density was increased for the unsaturated "cohesive" soil conditions, and scour depth decreased with the decrease of the initial water content for the saturated "cohesive" soil. Briaud et al. (1999, 2001 and 2004) presented a method termed SRICOS for predicting scour in "cohesive" soils, with such an approach being the most comprehensive to date in literature.

Scouring experiments around cylinders using "clay"–sand mixtures were carried out by Hosny (1995), Ansari et al. (2002), and Debnath and Chaudhuri (2010a, 2010b), among others, with fines fraction in the range of 0.05–0.4, 0.1–0.6, 0.2–1.0 and 0.05–0.35, respectively. Hosny (1995), and Debnath and Chaudhuri (2010a, 2010b) concluded that maximum scour depth decreases with the increase of "fines" content whereas Ansari et al. (2002) indicated that the maximum equilibrium scour depth in sediments with fine contents could be higher than that of non-cohesive sediments under similar experimental conditions. Perhaps one reason for the difference in conclusions is attributed to the nature of fine being used in the study. In Ansari et al.'s (2002) study, the soil is reported as having zero Plasticity Index (PI). Table 1 shows several empirical equations to estimate scour depth, with the corresponding fines fraction and Froude number range for their applicability. The equations proposed by Hosny (1995), and Debnath and Chaudhuri (2010a, 2010b) are only applicable for a rather narrow Froude number (i.e. 0.13 – 0.33) range, in comparison to Ansari et al.'s (2002) range.

Table 1. Empirical Equations for Estimating Scour Around Bridge Piers for Soils with Fine Contents

Reference	**Equation**	**Conditions**	**Comment**
Hosny (1995)	$d_s/b = 18.9(F_r/(1+C))^2$	$C \leq 0.4$ and $0.18 \leq F_r \leq 0.33$	b = pier diameter, F_r = Froude number = $V/(gd)^{0.5}$, V = approach flow velocity, g = gravitational acceleration, d = depth of flow and C = clay fraction.
Ansari et al. (2002)	$d_{smc}/d_{sms} = 1.51(C_*/\varphi_*)^{0.2}$	PI = 0 and $0.16 \leq F_r \leq 0.69$	d_{smc} = maximum scour depth for cohesive sediments, d_{sms} = maximum scour depth for cohesionless sediments (estimated using equation proposed by Kothyari et al. (1992)), $C_* = [\%P_c.\ C_u] / [(\gamma_s-\gamma_w).d_a]$, $\varphi_* = [\%P_c .\ \tan \varphi_c + (1 - \%P_c) * \tan \varphi_s] / \tan \varphi_s$, $\%P_c$ = percentage of clay content, C_u = undrained shear strength of soil, γ_s = unit weight of soil, γ_w = unit weight of water, d_a = arithmetic mean diameter, φ_c = angle of internal friction for clay and φ_s = angle of internal friction for sand.
Debnath and Chaudhuri (2010b)	$d_s/b = 8.2F_r^{0.79}C^{-0.28}(IWC)^{0.15}(\tau_s/\rho V^2)^{-0.38}$	$0.13 \leq F_r \leq 0.20$, W.C. ≤ 0.4, $C \leq 0.4$ and $0.78 \leq \dot{\hat{V}} \leq 1.65$	C = clay fraction, IWC = initial water content, τ_s = bed shear strength, ρ = density of water, $\dot{\hat{V}} = V/V_{cs}$, V_{cs} is critical threshold velocity for sand and V = approach flow velocity.

3 EXPERIMENTAL PROGRAM

Testing was conducted in a circular chamber with a diameter of 1.0 m (3.3 ft) and a depth of 1.0 m (3.3 ft). Two 1.5 m long probe sections, with the bottom section fitted with 19 mm truncated tip, were used for testing. Figure 1 shows the probe set up prior to testing.

3.1 *Test Soil*

The test soil was composed of 15% fine grained particles and 85% sand by dry weight. Percent dispersion of the fine grained fraction was estimated by performing Double Hydrometer test. Percent dispersion is the ratio of the dry mass of particles smaller than 0.005 mm diameter, without a chemical dispersant, to the same type of data from the hydrometer test but with a chemical dispersant, expressed as a percentage. A dispersion value higher than 50% was obtained for the fine grained soil, and therefore the fine fraction is classified as dispersive. The sand and the fine soil components were mixed thoroughly with an electrical mixer, in a drum, until a uniform mix was obtained. The mixing process was repeated after the soil was transferred to the test chamber (shown in Figure 1).

Figure 1. Photograph of the Probe Set Up in the Chamber Prior to Testing

The chamber was filled up to 1m mark with the silty sand soil and approximately 0.45 kN weight was applied on the top, to induce consolidation, for a week. Several specimens were then retrieved for physical characterization of the test soil. Initial water content of the mixture ranged of 18% - 23%. The results from the particle size analysis for three types of soils are shown in Figure 2, with the test soil designated as "Silty Sand" (all soils have been designated according to Unified Soil Classification System). Table 2 shows the physical and strength properties of the test soil, with the undrained strength estimated using the Fall Cone test.

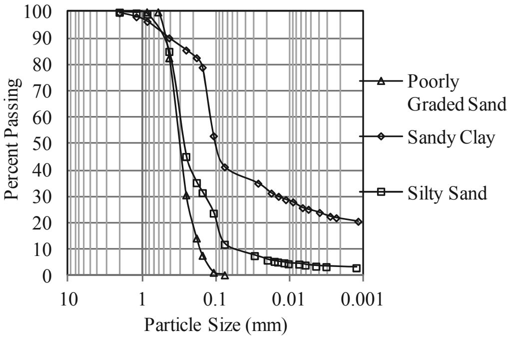

Figure 2. Grain Size Distribution of Test Soil: Silty Sand

Table 2. Properties of the Test Soil

Dry unit weight (kN/m^3) = 17.7	Mean Particle Diameter, D_{50} (mm) = 0.26	Undrained Shear Strength, C_u (kPa) =5-8	PI= Non-Plastic

4 TEST DATA INTERPRETATION

Figure 3 shows the results from the testing using four different run times. Based on the results from previous testing in a sand pit, Gabr et al. (2012) evaluated a critical stream power (P_c) value = 24 Watts/m^2 for a sand with D_{50} = 0.30 mm. Using a similar technique of extrapolation approach, the data in Figure 3 is extrapolated to zero penetration rate to yield an average P_c value of 16 Watts/m^2 for the test soil. Similar to the observation by Gabr et al. (2012), a minimum of "45 sec" run time is needed to provide a reliable measurement of the penetration rate. To calculate critical shear velocity from the P_c, the following equation is used (Annandale, 2006):

$$P_c = \gamma q h \tag{2}$$

where, γ is unit weight of water, q is the discharge per unit area, and h is the hydraulic head including the jet velocity head.

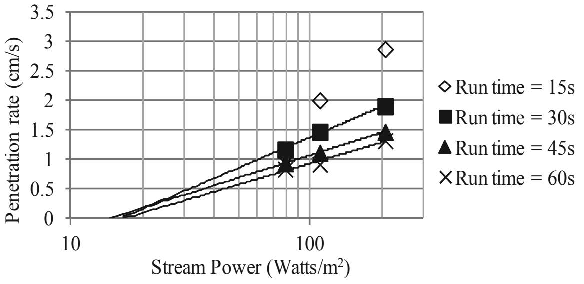

Figure 3. Extrapolation of the Stream Power to Assess Critical Stream Power Value (P_c)

Using Equation 2, a critical velocity is back calculated as 0.32 m/s corresponding to a critical stream power value of 16 Watts/m^2. As flow field changes around the pier, and therefore the flow-related shear stress, the shear stress below which no scour is assumed to take place is estimated using the equation proposed by Briaud et al. (1999):

$$\tau_{\max} = 0.094 \rho V^2 \left(\frac{1}{\log R_e} - \frac{1}{10}\right)$$

where, ρ is water density, R_e = VD/υ is Reynolds number, V is depth average flow velocity at the location of the pier if the bridge were not there, D is pier diameter and υ is kinematic viscosity of water. In this case, assuming a pier diameter of 1 m and a depth of flow= 2 m, the critical shear stress value is estimated equal to 1.75 Pa.

Figure 4 shows the equivalent penetration rate per shear stress function for the averaged test data. During testing, the probe tip is in close proximity of the soil mass, and erosion occurs within the jet potential core. The applied shear stress in this case is estimated using the relationship presented by Annandale (2006) as:

$$\tau = C_f \rho U^2 \tag{4}$$

where: τ = applied shear stress to bed in N/m^2, U = average velocity of water at the tip (m/s), ρ = density (kg/m^3), and C_f is a friction coefficient = 0.016 according to Annandale (2006).

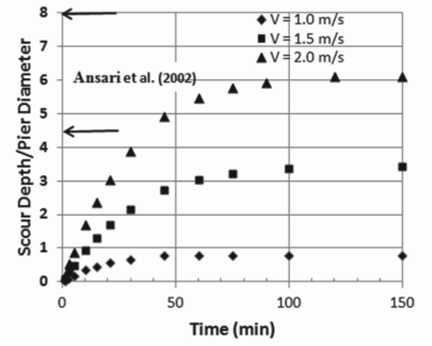

Figure 4. Computed Scour Around Circular Bridge Pier on Silty Sand Bed; Range between Arrows is Values Estimated using Empirical Equation .

The slope of the data for the 45 and 60 secs in Figure 3 provides a parameter equivalent to the detachment rate coefficient (k_d) proposed by Mehta (1991). The k_d values of 0.017 cm/sec per N/m^2 and 0.015 cm/sec per N/m^2 are estimated, respectively, for the test soil at run times of 45 and 60 seconds. In comparison, k_d values of 0.017 cm/sec per N/m^2 and 0.013 cm/sec per N/m^2 were observed respectively for sand at run times of 45 and 60 seconds by Gabr et al (2012). The K_d value obtained at 60 sec for the test soil is approximately 13% higher than the value obtained for sand. This observation agrees with the conclusion made by Ansari et al. (2002), where the authors indicated that in a lower fines content (<20%) type of soil, non-plastic fine particles are carried away as the resistance due to "cohesion" becomes insignificant.

5 SCOUR AROUND BRIDGE PIERS

Local scour around bridge piers occurs due to induced shear stresses associated with flow field changes. Ettema et al.

(2011) indicated that the estimation of time-dependent clear water scour magnitude at bridge piers remains a challenge during the limited duration of excessive flow as, for example, in the case of a storm surge. Equation 3 proposed by Briaud et al. (1999) provides an estimate of the τ_{max} as a function of the flow velocity at a round pier. However, in order to assess the scour depth with time using ISEEP data, a reduction in τ_{max} with the progression of scour depth is needed. While a significant number of studies have been performed for the assessment of maximum scour at piers, these approaches were not specifically concerned with evolution of shear stresses with time and flow field. Data presented by Briaud et al. (1999) indicated a nearly linear relationship between the τ_t/τ_{max} and δ (scour depth) /d (pier diameter), where τ_t is the shear stress with the progression of scour depth with a minimum value of $\tau_{critical}$. An iterative approach is used to estimate τ_t since the maximum depth of scour is not known a'priori.

The scour depth around a bridge pier is estimated for a flow velocity range of 1.0 m/s to 2.0 m/s (Froude number 0.23 to 0.45) with a pier diameter of 1 m and a depth of flow = 2 m. The computations are performed based on the ISEEP data and compared with the values from the empirical equation by Ansari et al. (2002), as the conditions for Ansari et al. (2002) empirical equation are in agreement with the percent fines in the test soil. Figure 4 shows scour depth, normalized with respect to the pier diameter ratio, versus time for different flow velocities. The scour depth from the equation by Ansari et al. (2002) is within 4.6-8 D (D = pier diameter) for a flow velocity range of 1.0 m/s to 2.0 m/s, which is higher than values estimated using the ISEEP data. A reason for the deviation can be attributed to the fact that the maximum shear stress equation developed by Briaud et al. (1999) was for clay, while the soil in this study is 85% sand. Furthermore, the application of Ansari et al. (2002) approach required the definition of the scour level in sand first which can widely vary depending on the parameters assumed.

6 SUMMARY AND CONCLUSIONS

The ISEEP approach developed by Gabr et al. (2012) was used to provide parameters for evaluating scour potential with time in a 15%-85% silty sand mixed bed. Soil erosion parameters included critical shear stress and detachment rate coefficient. Higher detachment rate was obtained for the silty sand than the sand soil. Application of the ISEEP data to assess magnitude of scour with time for a circular bridge pier indicated a scour depth on the order of 1-6 m versus 4.6-8 m estimated by an empirical equation in literature. The difference in results is attributed to the difference in the approach for scour computation and the limitations of estimating the evolution of shear stresses with time and flow field. In this case, the applicability of the empirical equation was somewhat limited since the testing conditions deviated in terms of soil type and moisture conditions.

7 ACKNOWLEDGEMENTS

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