Report as of FY2007 for 2006FL145B: "Measurement of erosion around hydraulic structures"

Publications

Project 2006FL145B has resulted in no reported publications as of FY2007.

Report Follows

Erosion at Hydraulic Structures

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1 Introduction

1.1 The problem addressed in this report

As a result of recent active hurricane seasons, many District waterways experienced bank and bed erosion. The erosion was more severe downstream of flow control structures, particularly spillways and weirs. These erosions cause several undesired problems, for example, erosion on the discharge canal potentially endangers the structural stability of the flow control structure. In addition, bank erosion may also result in damages to the levees. The eroded sediments may also be carried by the flow to the lakes and reservoirs and causing undesired sedimentation, and resulting in reduction of storage capability for water supply and deterioration of the water quality.

The primary objective of this report is to summarize an effort on literature survey for existing experimental studies of erosion problems, specifically at hydraulic structures and river banks and to recommend a process-based experimental approach to further investigate erosion problem at selected District field sites. A process-based approach based on physical principles allows effective field experimental design and data analysis so that eventually a general formulation for evaluating erosion problem can be proposed for District's management purposes.

In the past several decades, there have been extensive studies on bridge pier and abutment scour for both cohesionless and cohesive sediments. Many of these studies adopted process-based approach and had greatly advanced our physical understanding on local scour and common sediment erosion processes. Therefore in this report, after a general discussion on scour types, we begin our investigation by summarizing some of the major finding from bridge pier scour studies (section 2). Some of the lessons learned from these studies, such as the concept of equilibrium, timescale to equilibrium and differences between noncohesive and cohesive sediments, are very important guidelines to our major objective regarding erosion at hydraulic structures and bank erosion. In section 3, erosion below spillway and culvert outlets are discussed, respectively. Section 4 focuses on bank erosion. In each of the sections 3 and 4, we begin with a general description of the problem, and a literature survey on existing approaches. By the end of each section, recommendations for new field experiments to improve our current predictive capability are described and planned. Finally, in section 5 a brief literature survey on recent advances in sediment transport modeling and three-dimensional numerical approach based on computational fluid dynamics (CFD) for erosion at structures is discussed. Major conclusions from this investigation are remarked in Section 6.

1.2 Scour types

Scour is the loss of soil by erosion due to the flow. Scour is generally divided into several types (e.g., Mueller & Wagner 2005; Briaud et al. 1999) and each scour type does not necessarily have a precise definition in all physical aspects when compared to other types. Therefore, for the purpose of clarity and relevance to this report, they are first defined here.

In terms of the mechanism, scour is a result of acceleration of the flow (and possibly enhancement of flow turbulence) and it is generally a time-dependent process. Considering the stream flow and the sediment bed as one system in equilibrium at a specific time, then the scour is a process that represents how the streambed morphology is in respond to the local flow acceleration through sediment erosion/accretion and eventually arrives at another equilibrium state. The acceleration of the local flow can be resulted from increase of stream flow velocity due to flooding or due to local obstructions (e.g., contraction) to the water flow, or both. The type of flow disturbance can be due to the enhanced shear flow and bottom/wall stress near the streambed/bank (e.g., bridge scour, bank erosion) or the direct impact to the soil through a jet-like flow (scour below spillway, culvert outlets).

The *long-term scour* is the general aggradation or degradation of streambed elevation due to natural and human causes. In this study, we focus more on the *short-term scour* in which the streambed respond to short-term stream-flow runoff cycles, e.g., a stream's storm hydrograph. Within the context of short-term scour, we can further distinguish between the *contraction scour* and the *local scour*. The contraction scour is resulted from the increase of normal stream flow due to natural or manmade contractions. It includes removal of soil from a river's bed and banks and is a concern of the overall channel stability. The local scour refers to removal of soil from around piers, abutments or of more concerns here, the hydraulic control structures.

The local scour can be further classified based on the mode of sediment transport due to the approaching flow (e.g., Melville and Chiew 1999; Barbhuiya and Dey 2004). The *clear-water scour* occurs when the approaching flow intensity is not sufficient to initiate ambient sediment transport (except around the structure). Hence, there is no upstream supply of sediment relative to the local scour. On the other hand, *live-bed scour* occurs when the approaching flow is energetic enough to entrain bed sediment from the upstream and hence the local scour is continuously fed with upstream supply of sediment. The time-dependent behavior of the scour processes is rather different for clear-water scour and live-bed scour (Fig 1). The equilibrium scour depth is attained more rapidly during live-bed scour and strictly speaking, it is a quasi-equilibrium state due to for example, the migration of bedforms. The clear-water scour reaches its equilibrium more slowly. However, the resulting magnitude of the maximum equilibrium scour depth is greater (about 10%) than that for live-bed condition (Graf 1998).

If the interest here is erosion due to storm, live-bed scour may be more likely to occur. However, the duration of the storm becomes another critical factor to be incorporated. In this case, the timescale to reach equilibrium must be a competing factor with the storm duration. It is well-known that the erodibility for non-cohesive (sand) and cohesive (clay, fully consolidated) sediments are rather different and hence the timescale to reach equilibrium must also depend on the cohesion property of the sediment. In the field condition the size of sediment is often non-uniform and hence the armoring effects due different sizes of sediments become another concern. Also because of the non-uniform sediment, most upstream approaching flow may consist of the fines (or at least some washloads) which is recently shown to be sensitive to the local scour (Sheppard et

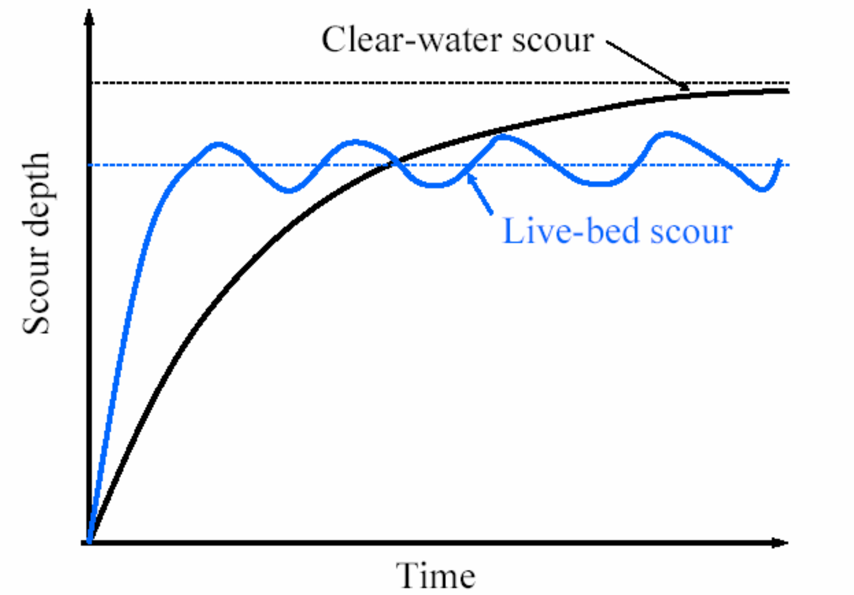

Fig. 1: Schematic descriptions for clear water scour (black curve) and live-bed scour (blue curve). See Graf (1998) for a similar plot.

al. 2004). Therefore, the definition of clear-bed and live-bed scour can not be definite. These are the critical issues that are relevant to both the fundamental sediment transport and various kinds of erosion problems that we will address in this report through a comprehensive literature survey.

2 Lessons Learned from Bridge Scour

In the Unite States, there are about 500-thousand bridges that are over water (National Bridge Inventory 1997). In the past 30 years, about 60% of the bridges failed were due to scour (Shirole & Holt 1991; Briaud et al. 1999). Therefore, there has been an extensive research on bridge scour ranging from theoretical analyses, laboratory/field experiments, and numerical modeling. Research findings resulted from these studies, especially those related to the physical processes of erosion, can certainly provide useful guidelines for other type of erosion problems relevant to District's interests. Hence, this section concentrate on summarizing important lessons learned from extensive bridges scour studies that will be useful for our major objective regarding erosion at hydraulic structures and bank erosion

2.1 Dimensional analysis

Bridges scour is a rather complex problem form the fluid mechanics point of view. It involved interactions among turbulent fluid flow, sediment and the geometry of the structure. Dimensional analysis is a very useful tool as the first step toward a more comprehensive study. Here we utilize a framework for analysis following Melville & Chiew (1999). This framework is concise but provides considerable insights into the dynamical processes and is also used by other researchers recently for interpreting measured scour data (Sheppard et al. 2004; Sheppard & Miller 2006).

The local scour is caused by the presence of structure that alters the original flow field from an equilibrium state. Therefore, it is reasonable to expect that after the installation of structure, flow and sediment bed may evolve to another equilibrium state through the removal of soil and the adjustment of bed morphology. The maximum equilibrium scour depth d_{sm} is perhaps the most important quantity in the scour prediction. The maximum scour depth at a bridge pier generally depends on flow parameters, bed sediment properties, pier geometry and time.

Assuming uniform sediment properties, fully turbulent flow and simple pier geometry, the maximum scour depth at a cylindrical pier of diameter D can be written as (Melville & Chiew 1999):

$$\frac{d_{sm}}{D} = f\left(\frac{U}{U_c}, \frac{h}{D}, \frac{d}{D}\right) \tag{1}$$

where U is the averaged stream velocity at a significant distance upstream of the structure (stream velocity without the obstruction of structure), U_c is the critical velocity for sediment entrainment, h is the mean flow depth, and d is the mean sediment particle diameter, usually calculate from d_{50}.

The 1st parameter on the right-hand-side of (1) represents the nondimensional flow intensity. This parameter not only characterizes the intensity of the stream flow but also differentiates between the clear-water scour ($U/U_c < 1$) and live-bed scour

($U/U_c > 1$, see their definitions in section 1). The 2nd parameter represents the effect of flow shallowness. The 3rd parameter represents the sediment coarseness. The dependence of maximum scour depth with respect to these parameters reveals important mechanisms controlling scour processes and is discussed in more details in section 2.2.

The timescale to reach the equilibrium scour depth and the time-dependent behavior of scour are not incorporated in equation (1). However, this is another important aspect of the scour processes and has been studied in details by several studies (e.g., Melville & Chiew 1999; Briaud et al. 1999) for both non-cohesive and cohesive sediments. The importance of timescale in scour processes is discussed in section 2.3 and 2.4.

2.2 Prediction for the maximum equilibrium scour depth

The maximum equilibrium scour depth is the most important quantity for a scour prediction and has received the most investigations in the literature. It represents the maximum scour damage that can occur for a given flow condition, sediment properties and structure dimension if the duration of the flow forcing (i.e., a storm) is long enough to attain the equilibrium. Therefore, the maximum equilibrium scour depth is also the most conservative engineering design guideline.

Using the dimensional analysis described in section 2.1. Melville (1997) and Melville and Chiew (1999) proposed an empirical relation using several laboratory data sets conducted in 4 different flumes (totally 70 cases). The data used in this study is for relatively small structure due to the constraint of the laboratory facility and hence the largest ratio D/d is about 200. This value is smaller than what typically encounter in the field condition. Following Melville and Chiew (1999), Sheppard et al. (2004) and Sheppard and Miller (2006) further utilize a proto-type scale flume facility (at USGS and University of Auckland), extend the database for D/d as high as 4155, and propose a new empirical formulation to estimate the maximum equilibrium scour depth. On the other hand, one of the most commonly used scour prediction equation is the HEC-18 equation (Richardson and Davis 2001) recommended by Federal Highway Administration, U.S. Department of Transportation. Since the original HEC-18 was proposed, it has been revised few times by calibrating with new field data (Mueller & Wagner 2005). Other empirical equations for scour prediction can be found in a recent review paper by Barbhuiya and Dey (2004).

Before presenting several widely-used formulae for predicting maximum equilibrium scour depth, it is useful to exam the general dependence of d_{sm} on each of the nondimensional parameters on the right-hand-side of (1). Summarizing the results and analyses presented by Melville and Chiew (1999) and Sheppard et al. (2004), their most important conclusions are shown here graphically in Fig 2-4.

As the nondimensional flow intensity increases, the scour depth has two peak values (Fig. 2). The maximum clear-water scour occurs when $U/U_c = 1$. Subsequent increment of flow intensity initiates sediment movement over the entire streambed (not just the scour hole area). In such live-bed condition, the upstream flow (before approach the scour hole) already consists of suspended sediment and the suspension capacity of the overall flow is reduced (e.g., suspended sediment reduces flow turbulence, Hsu et al. 2003). The maximum equilibrium scour depth thus reduces according (usually about 10-20%). Even at the live-bed scour maximum (the second peak in Fig. 2), its magnitude is still smaller than that at clear-bed scour maximum (i.e., at $U/U_c = 1$ in Fig 2).

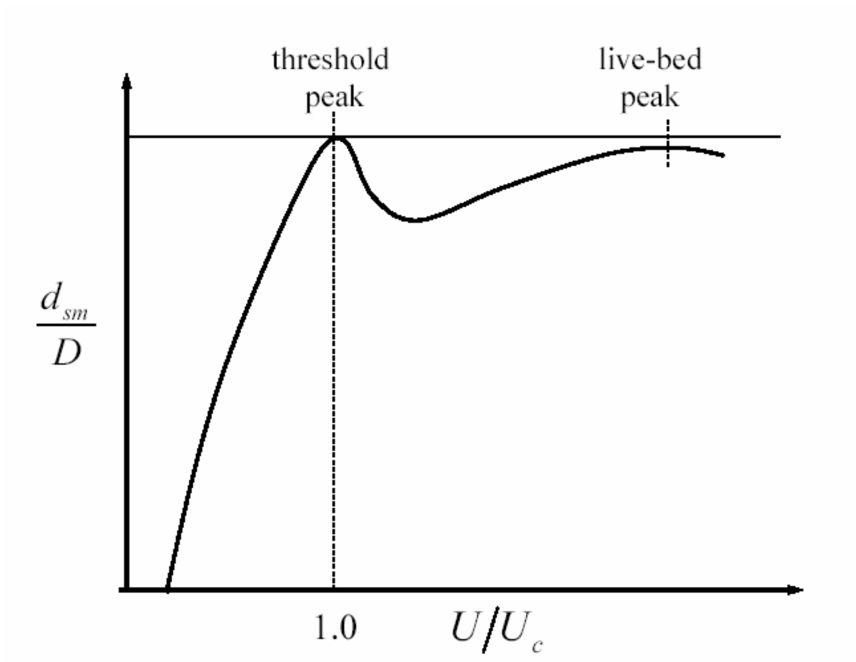

Fig2: Influence of flow intensity U/U_c on nondimensional scour depth. $U/U_c = 1$ differentiates clear-water scour and live-bed scour.

As the flow shallowness increases, the nondimensional scour depth increases until it reaches an asymptotic value (Fig. 3). Approximately, when the water depth is about several times larger than the pier diameter, further increment of water depth has no effect on the scour depth. The flow turbulence around the pier, which more or less determines the amount of sediment transport, can be approximately characterized by the largest size of the turbulent eddy. When the water depth is sufficiently deep, it has no effect on the local flow and the largest turbulent eddy size is determined by pier diameter and the so is the scour depth. On the other hand, when the water depth is relatively small compared to the pier diameter (or relatively wide pier), the largest turbulent eddy size must be confined by the water depth and the scour depth must scale with the water depth.

Based on most of the small-scale laboratory results, it is generally believe that then grain size has no effect on scour depth except for relatively coarse grain (*D/d<50*, Fig 4, black curve), the scour depth decreases because coarse grains provide significant bed roughness and porous effect that dissipate the flow energy (e.g., Ettema 1980). However, this conclusion is made from small-scale laboratory results with limited size of

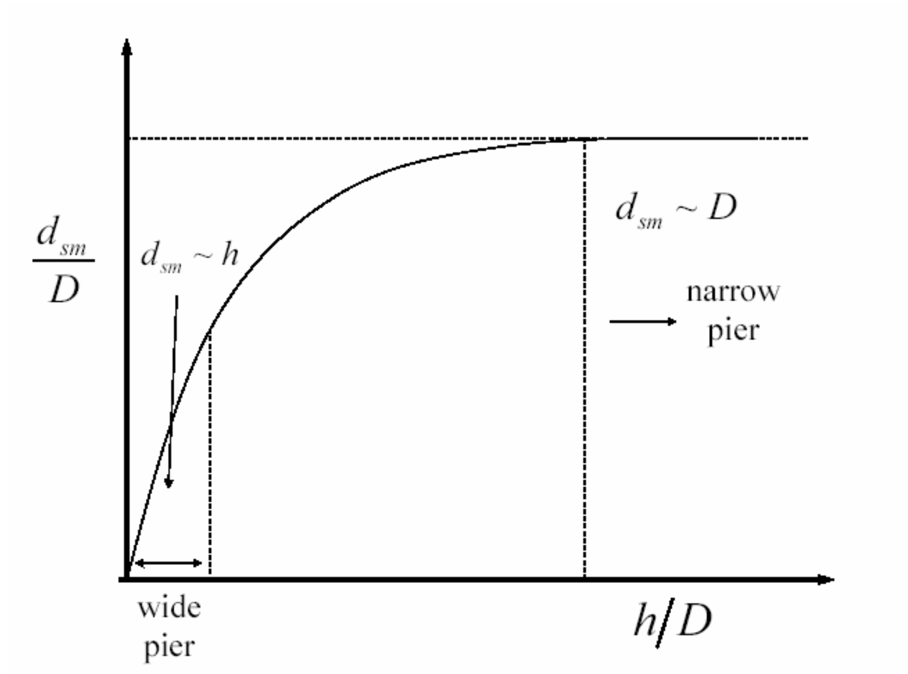

Fig. 3: Influence of flow shallowness on nondimensional scour depth.

pier and *D/d* value is no more than about ~100. Recently, new evidences based on prototype experiments, with *D/d* as large as 1000~4000 suggest nondimensional scour depth clearly decreases as *D/d*>>50 (Sheppard et al. 2004). There is no definite explanation at this point for the reason why nondimensional scour decreases for fine sediment (Sheppard, personal communication). One possible explanation could be due to the effect of suspended sediment on damping the flow turbulence (Ross and Mehta 1988; Hsu et al. 2003; 2006), which has been proved to be important in controlling the lutocline dynamic of soft fluid mud at estuary or continental shelf (Trowbridge and Kineke 1994) when mud concentration is greater than about 10g/l. This important finding in scour process by Sheppard et al. (2004) also demonstrates the importance and the justification for pursuing field experiments on scour processes.

The scour prediction equation proposed in Melville and Chiew (1999) is a function flow-pier width, flow intensity and particle size. However, the equation is dimensional (even though they propose equation (1) that is nondimensional). Sheppard et al (2004) and Sheppard and Miller (2006) later followed equation (1) and propose scour formulae for bridge pier that is more complete. The Sheppard's equations are given as, for clear water scour,

$$\frac{d_{sm}}{D} = 2.5\tanh\left[\left(\frac{h}{D}\right)^{0.4}\right] f_d\left(\frac{D}{d}\right)\left\{1-1.75\left[\ln\left(\frac{U}{U_c}\right)\right]^2\right\} \tag{3.1}$$

and for scour above live-bed peak

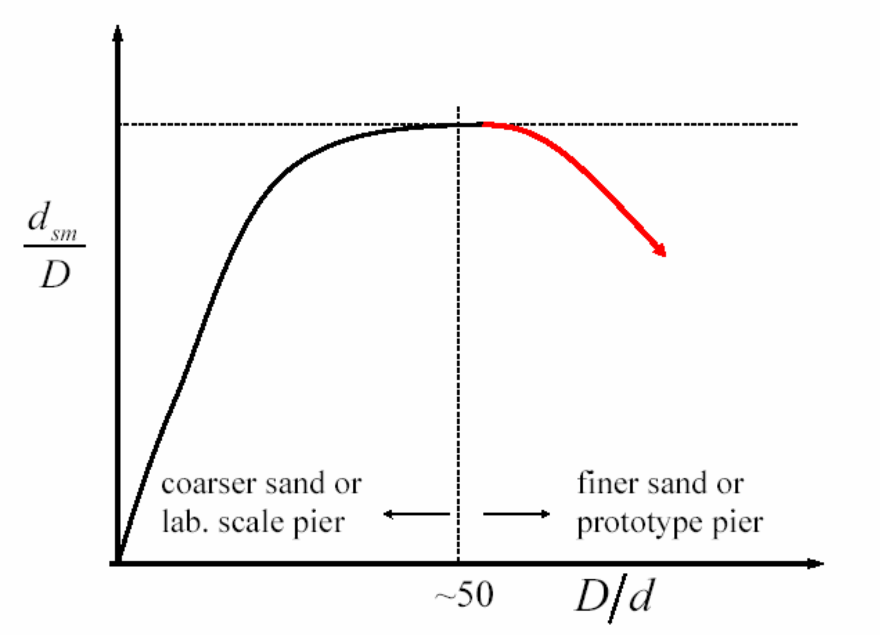

Fig.4: Influence of sediment coarseness on nondimensional scour depth. The red curve represents new findings based on prototype scale experiments.

$$\frac{d_{sm}}{D} = 2.2\tanh\left[\left(\frac{h}{D}\right)^{0.4}\right]. \tag{3.2}$$

A linear interpolation can be used in between live-bed scour range up to live-bed peak. In (3.1), a complete functional dependence of D/d is obtained through large-scale flume experiment as:

$$f_d\left(\frac{D}{d}\right) = \frac{D/d}{0.4(D/d)^{1.2} + 10.6(D/d)^{-0.13}} \tag{3.3}$$

The HEC-18 equation, which is used and calibrated in the field, is nondimensionalized in a rather different way as compared to Melville's and Sheppard's formula. HEC-18 formula is based on the Froude number (Richardson and Davies 2001):

$$\frac{d_{sm}}{D} = 2.0K_3\left(\frac{h}{D}\right)^{0.35} F_r^{0.43} \tag{4.1}$$

where K_3 is a numerical coefficient that account for bedforms (1.1 for plane bed and small dunes and up to 1.3 for large dunes). The Froude number F_r is defined as

$$F_r = \frac{U}{\sqrt{gh}} \tag{4.2}$$

2.3 Time-dependent scour behavior

The timescale to attain the equilibrium maximum scour depth has received less investigation than the equilibrium scour depth itself. This is partly because for sandy environment, the timescale to attain equilibrium is relatively short (or on the same order of magnitude) when compared to typical duration of an extreme event (e.g., storm). In addition, the maximum equilibrium scour depth already provided the most conservative design criterion (but costly). However, as our capability for predicting the scour depth advances, the time-dependent behavior received more and more interests in the past several years (Melville and Chiew 1999; Briaud et al. 1999; Sheppard et al. 2004). Predicting the time-dependent scour depth is essential when considering storm of relatively short duration or even more importantly when considering fully consolidated cohesive soil erosion (see section 2.4 for details).

Following the nondimensional form in (1), the time t for scour depth evolution can be normalized by T_e, the timescale to reach the equilibrium maximum scour depth. Hence, we can add the 4$^{\text{th}}$ nondimensional quantity t/T_e into equation (1) for predicting the general time-dependent scour depth d_s:

$$\frac{d_s}{D} = f\left(\frac{U}{U_c}, \frac{h}{D}, \frac{d}{D}, \frac{t}{T_e}\right). \tag{2}$$

According to Melville & Chiew (1999), the time evolution of scour depth d_s approaching the final equilibrium maximum scour depth d_{sm} can be well represented by the following equation:

$$\frac{d_s}{d_{sm}} = \exp\left\{-0.03\left|\frac{U_c}{U}\ln\left(\frac{t}{T_e}\right)\right|^{1.6}\right\} \tag{3}$$

This equation requires an estimate of T_e. Existing data suggest (Melville & Chiew, 1999) T_e itself when normalized by D/U, depends on flow intensity, flow shallowness and sediment coarseness. An empirical formula is suggested to predict T_e as:

$$\begin{aligned} T_e(\text{days}) &= 48.26\frac{D}{U}\left(\frac{U}{U_c} - 0.4\right), \qquad \frac{h}{D} > 6 \\ T_e(\text{days}) &= 30.89\frac{D}{U}\left(\frac{U}{U_c} - 0.4\right)\left(\frac{h}{D}\right)^{0.25}, \qquad \frac{h}{D} \le 6 \end{aligned} \tag{4}$$

Notice here that when the water depth is large enough (6 times the structure diameter), the effect of water depth on scour vanishes, consistent with that observed for maximum equilibrium depth.

An estimate of the typical time evolution for scour to reach equilibrium is insightful at this point. Considering a peak flood velocity of *U=0.8m/s*, sand diameter *d=0.22mm*, structure diameter *D=1.0m*, and water depth *h=1.2m*. The threshold velocity in this case can be confidently estimated as $U_c = 0.32$ m/s (Melville 1997). The time scale for the scour to reach equilibrium, according to (4) is calculated as 84.87 days, which is seemingly a very long time. However, the time-dependent behavior described in (3) is rather nonlinear (see Fig 5, for an example). In fact, in simply 1 day, the scour depth is as deep as 93% of the final equilibrium depth. Therefore, when considering the uncertainties in estimating maximum equilibrium scour depth itself, the scour processes reach its maximum scour depth in a rather short period of time (~1day) when compare to typical flood duration.

On the other hand, a fully consolidated cohesive soil (clay) has a rather low erodibility and the threshold velocity can be several times higher than that for sand. Let's now assuming equation (3) and (4) are equally applicable to cohesive sediment. Using a typical threshold velocity for clay of $U_c = 1.0$ m/s (Briaud et al. 2004) but with other parameters unchanged, it will take 4 day to reach 93% of the final equilibrium scour depth. This is about 4 times slower as compared to sandy condition. Therefore, for cohesive sediment the scour process is much slower and the duration of a storm is often not long enough for the scour to attain its maximum equilibrium depth. Notice that in reality, equation (3) and (4) may only qualitatively applicable to cohesive sediment and one would expect the empirical coefficient involved in (3) and (4) different from that used in sandy condition. As we will discuss in the next section (section 2.4), the scour processes for cohesive sediment is even much slower than our crude estimate here using (3) and (4) (see Fig. 5).

2.4 Scour for cohesive soils

Previous sections focus on bridge scour for non-cohesive, sandy environments (coarse-grained). The major difference between a non-cohesive and a cohesive sediment scour is that the erodibility for a fully consolidated, cohesive clay material is much less (sometime 1000 times less, Briaud et al. 2004; Ansari et al. 1999) than that of sand. Therefore, the scour depth for cohesive soil develops much slower than that for non-cohesive sandy material. An example for comparing sand scour and clay scour demonstrated by Brandimarte et al (2006) is reproduced in Fig 5.

For typical peak flow duration due to storm of say 1 day, it is sufficient for sandy scour to develop to its maximum equilibrium scour depth. Hence, simply estimating the maximum equilibrium scour depth at sandy environment is sufficient for engineering purposes. However, 1-days of storm duration are too short for cohesive soil to develop to the maximum scour depth. Hence, using an estimated maximum scour depth in a cohesive sediment condition usually over-predicts the scour and hence provides a design criterion that is too conservative. For scour in cohesive sediment, it is important to study the time-dependent behavior. Accurate descriptions on the time-dependent scour process for cohesive soil can save lots of money in building a reliable structure.

Briaud et al. (1999, 2004) developed a useful approach to predict the time-dependent behavior of scour depth for cohesive soil. This method is called SRICOS (Scour Rate In Cohesive Soils). In this approach, the maximum scour depth in clay is in fact considered to be similar to that in sand (same formulae presented in section 2.3 can be use). SRICOS is more complicated in predicting the time-dependent behavior. Briefly, SRICOS method can be described in several steps:

1. Estimate maximum initial bottom shear stress around the structure (i.e., structure with an initial flat bed). This can be estimated by measurements, or CFD simulations.
2. Obtain the initial scour rate. Again, if it were non-cohesive sediment, the initial scour rate can be estimated with good confidence using the maximum bottom stress obtained in (1) and a power law (Graf 1998). However, for cohesive clay material, such a simple relation does not exist. The erodibility of cohesive sediment is too complicated to allow for developing effective mathematical formulae to relate the bottom stress and erosion rate. In SRICOS, samples of cohesive material is taken from the field and tested in a laboratory facility, called EFA (Erosion Function Apparatus) to estimate the initial scour rate.
3. Estimate maximum equilibrium scour depth using well-developed method for non-cohesive sediment (e.g., formulae presented in section 2.3).
4. Using the initial scour rate (obtained from step 1 and step 2) and maximum equilibrium scour depth (obtained in step 3), the time-dependent behavior of scour can be calculated by a hyperbolic model. It is basically a nonlinear interpolation scheme to get "scour depth versus time". This method has been validated by extensive experimental data.

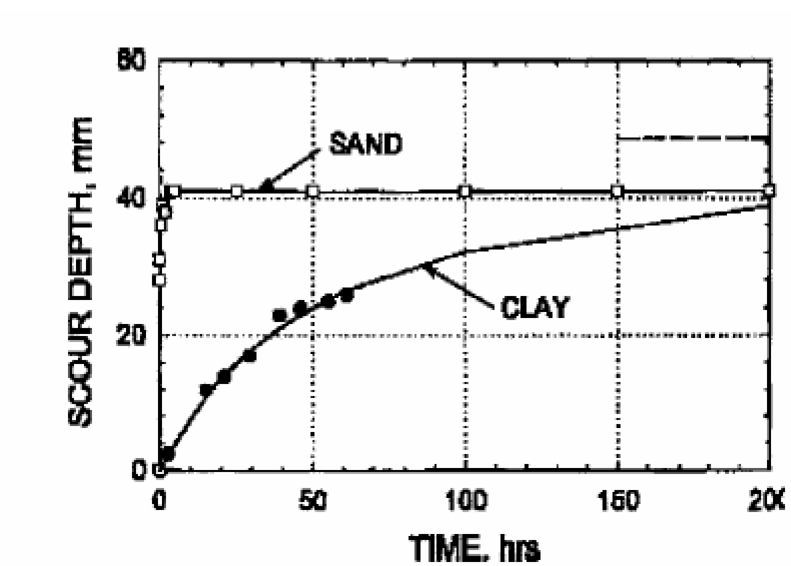

Fig 5. Scour development in clay is much slower than that in sand. (adopted from Brandimarte et al. 2006, see also Briaud et al. 2002)

The basic concept of this method appears to be rather general and hence may be applied to other type of erosion problems involving cohesive soil. For other type of erosion problem, different empirical formulae or experimental setups in getting the initial scour rate, the maximum erosion depth and the hyperbolic interpolation relation are required.

2.5 Summary

Several important experiences learned from extensive bridge scour studies that may be useful for other type of erosion problems for the District are summarize here:

(1) An equilibrium state exists for bridge scour and possibly other type of scour problems. The state of equilibrium provides the most important step toward simplifying the erosion problem from a engineering point of view because the maximum equilibrium scour can be estimated as the most conservative design criterion. Predicting the maximum equilibrium scour is the most fundamental step to study a scour problem.

(2) The time scale to attain the equilibrium state is another important parameter that needs to be estimated. The relative magnitudes between the equilibrium time scale for a specific scour problem and the duration of the episodic forcing (e.g., flooding) determine whether the time-dependent behavior of the scour needs to be further explored; or simply estimating the maximum equilibrium scour is sufficient. In general, the time scale for non-cohesive sediment (e.g., sand) scour is much shorter than that of cohesive sediment scour. If the driving force for scour is short-term stream-flow runoff, then predicting the maximum equilibrium scour depth is sufficient for non-cohesive sediment. However, for cohesive sediment the problem is more complex and time-dependent behavior of scour need to be further estimated or parameterized. The SRICOS method developed by Briaud et al. (1999, 2004) appears to be effective for predict bridge scour in cohesive soil. The concept of this method may also be applicable to other type of scour for cohesive soil.

(3) The general believe based on laboratory-scale experiment that fine sediment has no effect on scour is disproved by new prototype scale experimental finding (Sheppard et al. 2004). New finding suggests fine sediment scour is smaller than previously predicted and old design principle may be too conservative and the criterion may be too costly. This provides an important lesson for sediment transport: It is easy to match the similitude principles for pure hydrodynamics experiments but it is impossible to also match the sediment parameters concurrently. Hence for sediment transport study, it is extremely important to consider field or proto-type scale experiments.

(4) From a fluid mechanics point of view, scour formulae developed based on laboratory experiments are more complex and perhaps more complete. On the other hand, formulae developed from field studies are usually simpler. This is partly because in an idealized laboratory environment, some of the parameters are

easier to define than that in the field condition (or difficult to measure in the field). Additionally, the uncertainties in the field may also prevent more detailed calibrations if too many parameters are involved. However, we must note that the empirical coefficients in a laboratory-developed formula may suffer from scale effect and hence are often not as robust as compared to those simple formulae calibrated with extensive field data. The field scale erosion problem is certainly of more concern to the District. Hence, the suggestion is to start with formulae developed for field condition. Then, according to more detailed laboratory experimental results, we can identify one or two major mechanisms that may greatly improve the existing field-based formula. Using this hypothesis-driven approach, we can then define and design the scope of the field experiment that we will conduct.

3 Erosions below Spillway and Culvert Outlet (Plunge Pool Scour)

3.1 General

The capability to predict and control erosion near hydraulic structure is of great importance for the District. In the previous section, bridge scour problems are reviewed. Scour at bridge piers can be considered as a special case of a more general sediment erosion problem due to a shear flow (and vortices) that is primarily parallel to the bed. On the other hand, we must consider another important type of erosion problem that is due to the direct impact of the flow perpendicular (or arbitrary impact angle) to the sediment bed. This type of problem can be generally named as *plunge pool scour*. Plunge pool scour process is very important to, for example, the erosion downstream of a ski-jump bucket of a spillway or scour below a culvert outlet.

Spillways are widely used to dissipate the energy of floodwater. At the end of the long tunnels, ski-jump buckets are often used to deflect the flow, which throws the jet flow away into the air the then plunge into the tail water. Culverts are another common hydraulic control structure. Flows exiting a culvert outlet often drop from a distance into the downstream flow emulating a free jet. A the point of impact to the streambed, the free jet of water-air mixture enters the tail water, diminishes part of its energy but may eventually approach the streambed and excavating a scour hole.

There have been a great amount of empirical studies for estimating the scour depth below a ski-jump bucket of a spillway, dated back to as early as 1930's (e.g., Veronese 1937; Wu 1973; Martins 1975; Chee and Kung 1983; Mason and Arumugam 1985; Mason 1989, and reference therein). However, most of these formulae are either too simple to incorporate most of the important mechanisms or dimensionally incorrect to be generalized to various field conditions. For example, Azmathullah et al. (2005, 2006) conduct comprehensive evaluation of these formulae with 95 scour data observed at dams of India and Iran. They conclude that none of these formulae are satisfactory (correlations below 0.75). Anther study by Pagliara et al. (2004) also makes a similar conclusion.

Recently, detailed process-based laboratory study on plunge pool scour has been conducted and in the writer's opinion, has revealed systematically various critical mechanisms of jet-impinging type of scour processes and their dependence on flow and sediment parameters (e.g., Mason 1989; Aderibigbe and Rajaratnam 1996; Canepa and Hager 2003; Pagliara et al. 2006). However, none of the formulae proposed in these studies has been tested with field conditions and their practical applicability is not yet known.

In the following, several commonly used scour formulae downstream of a spillway are first summarizes, which consists of earlier empirical studies from 1930s to 1980s. More recent process-based laboratory studies on plunge pool scour is discussed later to assist our understanding on the physical processes involved in this type of scour. A new formulation following Pagliara et al. (2006) is review in more details. Finally, we will recommend a field experiment utilizing new sensors and process-based analytical framework that may improves upon the existing knowledge on scour downstream of the spillways or culvert outlets.

3.2 Scour formulae commonly used in the field

Similar to bridge pier scour described in section 2, the concept of equilibrium for jet impingement still hold here. According to a vast amount of laboratory and field observations, after the initial impact of the jet flow, the scour continue for a period of time until it attains a maximum equilibrium scour depth. Such equilibrium is established because either the jet has insufficient energy at the point of impact to erode more sediment or the secondary currents are insufficient to sweep away the suspended sediment out of the scour hole (Mason and Arumugam 1985).

The time scale to attain equilibrium for plunge pool scour is not well-documented. However, generally it is believed that the time to reach equilibrium is rather fast for non-cohesive sediment (Aderibigbe and Rajaratnam 1996). According to our survey, there has been no detailed study of this problem for cohesive sediment condition.

There is a great amount of studies focus on predicting the maximum equilibrium scour depth under a spillway. Earlier studies for maximum scour depth are rather simple and empirical. According to Mason and Arumugam (1985), who analyzed 31 formulae from 1930s to 1980s with prototype and laboratory scale data, the most promising formulae are of the following form, which is of the Schoklitsch-Veronese type (Schoklitsch 1935; Veronese 1937):

$$d_{sm} = K \frac{q^x H^y}{d^z} \tag{3.1}$$

where q is the unit discharge at the point of impact, H is the head from upstream to downstream water level, d is the characteristic sediment size as defined before, and K, x, y, z are empirical coefficients. According to calibrations with data, x is about 0.6, y is less certain but ranges from 0.2 to 0.3 and it appears to be even larger variation for z ranging from 0 to 0.5 and K ranging from about 0.2 to 2.8. There is also debate on what type of grain size shall be used (d_{90}, d_{50}, or others).

Notice that several important parameters are not incorporated into equation (3.1), such as impact angle and tail water depth. Mason and Arumugam (1985) suggest a modification of (3.1) to incorporate tail water depth h:

$$d_{sm} = K \frac{q^x H^y h^w}{d^z g^v} \tag{3.2}$$

with g the gravitational acceleration included for dimensional balance such that K become a nondimensional coefficient with a numeric value of about 2~3.

Among these earlier formulations, it is worthwhile here to discuss that proposed by Martins (1975). This formulation is not necessary more accurate but the scour formula is expressed into several nondimensional parameters, which is useful for our later process-based discussion (section 3.3). We show here a more complete version of Martins' formula later modified by Chee and Kung (1983) to include the jet impact angle α:

$$\frac{d_{sm} + h}{H} = 3.3 F_r^{0.6} \left(\frac{d}{H} \right)^{-0.1} \alpha^{0.1} \tag{3.3}$$

The sum of the scour depth and tail water depth $d_{sm} + h$ is normalized by the free fall height *H*. More importantly, the Froude number $F_r = q \big/ \sqrt{gH^3}$ of the spillway system is considered. It is also noted here that the length scale used for nondimensionalization in (3.3) is the free fall height *H*. As we shall see in the next section, based on recent detailed laboratory experiments, the grain size appears to be the more appropriate length scale for nondimensionalization of local sediment scour process.

3.3 Process-based analysis

Since 1980s, more detailed process-based (laboratory) study has been conducted in order to understand various physical mechanisms involved in plunge pool scour. To facilitate our discussion next, a definition sketch for plunge pool scour under a submerged or unsubmerged flow is shown in Fig 6.

Based on their experimental observation on scour due to impinging jet, Aderibigbe and Rajaratnam (1996) characterize the flow regimes as *Strongly Deflected Jet Regime* (SDJR) and *Weakly Deflected Jet Regime* (WDJR) (similar classification has been proposed by Kobus et al. (1979)). For Strongly Deflected Jet Regime, the jet penetrates considerably into the sediment bed and hence also gets reflected more strongly. The eroded sediment in the scour hole is transported out by strong re-circulatory flow and turbulence. The time required for the scouring processes is relative short compare to WDJR. The side slope of scour hole is more or less equal to angle of repose and hence the overall shape of the scour hole is maintained by a constant depth-to-width ratio. On the other hand, Weakly Deflected Jet Regime is characterized by a relatively weak penetration into the sediment bed. The eroded sediment is transported out of the scour hole by flow that is mainly along the bottom boundary without re-circulatory flow structure. The depth-to-width ratio is very sensitive to the flow condition and sediment properties.

Another important scour depth definition for plunge pool scour is strongly related to the processes involved in SDJR and WDJR. The *Dynamic Scour Depth* refers to the maximum equilibrium scour condition in which the jet flow remains turned on. The *Static Scour Depth* refers to the maximum equilibrium scour condition when the jet flow stops. The reason to consider the scour condition with or without jet flow is because in SDJR, the strong re-circulation flow and turbulence maintain some sediment suspended in the scour hole (but not swept away) and when the jet flow ceases, these suspended sediments settle back into the scour hole. Hence, for Strongly Reflected Jet Regime (SDJR), the dynamic scour depth is larger (sometimes much larger, depending on flow condition) than the static scour depth. On the other hand, there is no difference between the dynamic and static scours for Weakly Reflected Jet Regime (WDJR).

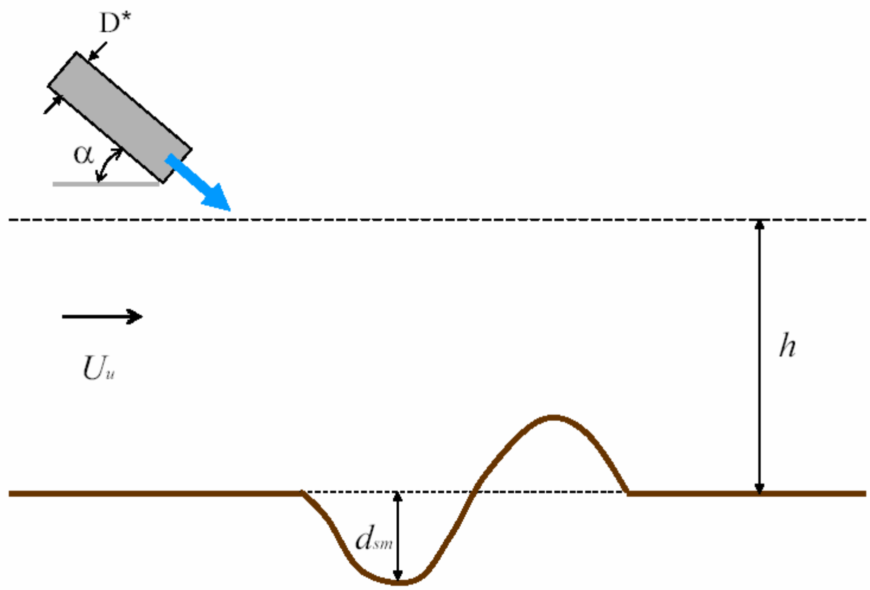

Fig 6: Definition sketch for plunge pool scour under an impinging jet.

Aderibigbe and Rajaratnam (1996) propose a semi-empirical formula for maximum (static) scour depth based on a simple theoretical formulation and laboratory data. Most importantly, they define a nondimensional parameter E_c that can appropriately parameterize the observed (static) scour depth:

$$E_c = \frac{q/h}{\sqrt{gd\,\Delta\rho/\rho}} \tag{3.4}$$

in which $\Delta\rho$ is the access density of sediment to the fluid flow. Notice that in Aderibigbe and Rajaratnam (1996) experiment, the jet nozzle is perpendicular to the bed and is placed right at the tail water depth. Therefore, *h* also represents the distance for the decay of jet velocity before it impinges to the bed. The numerator in (3.4) simply represents the approach jet velocity before impinging to the sediment bed while the denominator represents an equivalent weight (stabilizing force) of the sediment. Hence, E_c shown here for plunge scour is rather similar to the Shields parameter for shear flow (parallel to the bed) induced sediment transport. Equation (3.4) also suggests that for local scour processes, the appropriate length scale for nondimensionalization is the grain diameter.

Recently, Pagliara et al. (2006) and Canepa and Hager (2003) conduct one of the most comprehensive laboratory studies on plunge pool scour. They identify and test with several parameters that are important to the scour including, jet shape, jet velocity, jet air content, tail water depth, grain size sorting (nonuniformity), and the effect of upstream flow. A new formula for predicting the maximum equilibrium scour depth that incorporates all these effects is expressed as:

$$\frac{d_{sm}}{D^*} = f_1(F_r)f_2(\alpha)f_3(\beta)f_4(T)f_5(\sigma)f_6(F_{ru}) \tag{3.5}$$

with f_1 to f_6 the functions that each describes the dependence of normalized scour depth on a specific flow or sediment parameters, which we will discuss in details next.

Jet Shape The most important concern with these idealized laboratory studies is that whether they can well-represent the field applications. For example, idealized plane (rectangular) or circular jet is often used in the laboratory. However, the jet flow produced by spillway bucket or culvert outlet is of arbitrary shape. According to Pagliara et al. (2006), there is negligible effects of the jet shape provided that the equivalent diameter D^* is used and the corresponding velocity U is defined according to total flow rate and D^*. Therefore, for any arbitrary jet shape with cross-sectional area A, the equivalent diameter is defined as $D^* = \sqrt{4A/\pi}$ and mean jet velocity is simply $U = q/A$.

Froude Scaling After establishing that the equivalent diameter of the jet is the appropriate length scale to normalize scour depth (i.e., left-hand-side of (3.5)), the next major question is how to normalize the intensity of the jet. Even in the early days, the Froude scaling is acknowledged (e.g., Martins 1975; Mason and Arumugam 1985) to characterize the jet intensity. However, it is unclear what the appropriate length scale should be for the Froude scaling. Most existing scour formulae used in the field use the free fall height H as length scale. However, according to Pagliara et al. (2006) and Canepa and Hager (2003), detailed laboratory studies suggest that using the grain diameter as the length scale in Froude scaling gives very accurate fit. Therefore, the Froude number in equation (3.1) is defined as:

$$F_r = \frac{U}{\sqrt{g'd}} \tag{3.6}$$

with $g' = \Delta\rho g/\rho$ the reduced gravity. Notice that the Froude number defined here is consistent with the E_c adopted by Aderibigbe and Rajaratnam (1996) (see equation (3.4)). Based on extensive laboratory data Pagliara et al. (2006) suggest that when d_{90} is used the Froude number dependence is:

$$f_1(F_r) = 0.37F_r\,, \qquad 2 \le F_r \le 20 \tag{3.7}$$

Jet Impact Angle Pagliara et al. (2006) conduct experiments on four different jet impact angle α =30, 40, 60 and 90 degree, and suggest the following dependence:

$$f_2(\alpha) = 0.38\sin\left(\alpha + 22.5^\circ\right), \quad 30^\circ \le \alpha \le 90^\circ \tag{3.8}$$

Notice that the maximum scour depth does not occur at 90 degree but at around 60 degree possibly because a slight jet angle encourages suspended sediment to be swept away by the flow more effectively.

Jet Air Content The jet air content defined as, $\beta = q_A / q_w$ with q_A and q_w the air and water flow rate respectively, is well-mixed in the idealized laboratory experiment, which is not necessarily the case in the field condition. However, it is the first attempt to consider the effect of air content. When air is present in the jet flow, the jet velocity of the water-air mixture is calculated as $U = q_w(1+\beta)/A$, and Pagliara et al. (2006) recommend:

$$f_3(\beta) = (1+\beta)^{-m} \tag{3.9}$$

m is an empirical coefficient and depends on whether the jet is submerged. The effect of submergence is generally not sensitive to the scour depth except when jet air content is significant. When jet is unsubmerged, *m* is found to be 0.75, and the effect of air content is more pronounced than that for submerged condition with *m=0.5*. Therefore, when the water-air mixture velocity is considered (which is larger than pure water velocity), the scour depth decreases with increasing air content. However, if consider the water velocity only, the addition of air increase the scour depth (Canepa and Hager 2003).

Tail Water Depth The parameter $T = h/D^*$ represents a nondimensionalized tail water depth. According to Pagliara et al. (2006), increasing the tail water depth reduces the scour depth. This is because firstly, higher tail water depth suggests a longer attenuation distance of the jet flow before it impinges the bed. Furthermore, when the tail water depth is very low, the downstream velocity is large and it is easier to transport sediment away from and around the scour hole. Specifically, smaller tail water depth suggests a smaller ridge. The presence of the ridge is usually considered to prevent (protect) the scour. Hence smaller ridge further encourage deeper scour. It is suggested that

$$f_4(T) = 0.12\ln\left(\frac{1}{T}\right) + 0.45, \qquad \text{for } T^{-1} > 0.05 \tag{3.10}$$

Notice that the E_c parameter proposed by Aderibigbe and Rajaratnam (1996), shown in equation (3.4) already considered the effect of tail water depth and is consistent with the combined effect of equation (3.7) and (3.10). According to Aderibigbe and Rajaratnam (1996), when tail water depth is large enough such that E_c<0.35, scour is not initiated.

Sediment Sorting (nonuniformity) The sediment nonuniformity is defined by a sorting coefficient $\sigma = \left(d^{84} / d^{16} \right)^{1/2}$, the larger the σ, sediment is more well-sorted. It is well-known that for nonuniform sediments, the coarse particles impose an armoring effect on the fine particles and hence the overall transport is reduced (e.g., Armanini and Di Silvio 1988). Based on laboratory data, Pagliara et al. (2006) suggest

$$f_5(\sigma) = 0.33 + 0.57\sigma \tag{3.11}$$

Therefore, as the grain size distribution is more uniform, scour depth is larger.

Upstream Velocity A Froude number for upstream velocity is defined as $F_{ru} = U_u / \sqrt{gh}$. As upstream flow velocity increases, more suspended sediments in the scour hole tend to be transported away in the tail water, resulting in a larger scour. In addition, the sediment accumulated in the ridge is easier to be eroded and hence further enhance scour. Based on 40 separate tests, Pagliara et al. (2006) suggest

$$f_6(F_{ru}) = 1 + F_{ru}^{0.5}, \qquad \text{for } F_{ru} < 0.3 \tag{3.12}$$

In summary, based on dimensional analysis Pagliara et al. (2006) proposed equation (3.5) to predict maximum equilibrium scour depth. Further using comprehensive laboratory experiments (totally several hundred runs) empirical relations (3.7)-(3.12) are suggested. Because some of the experimental findings presented by Pagliara et al. (2006) are consistent with another study on a similar problem reported by Aderibigbe and Rajaratnam (1996), especially regarding to the Froude scaling and tail water depth, we can conclude that the physical processes involved in plunge pool scour in the idealized laboratory condition are relatively well-established. However, these new research results need to be further tested and calibrated in the field conditions before a new physical-based formulation for erosion under spillway or culvert outlet can be put forward.

3.4 Discussion and Recommendation

3.4.1 Summary on literature survey

Based on the literature survey presented in the previous sections on existing empirical scour formulae for spillways (section 3.2), and process-based laboratory study on plunge pool scour (section 3.3), several remarks can be made:

(1) It is clear that existing scour formulae for spillway are too simple (equations (3.2)-(3.3)) when compared to recent laboratory findings on plunge pool scour. Some of the physics are not included in the exiting scour formulae used for prototype, such as jet air content, sediment sorting and upstream velocity. The important effects of tail water depth and jet impact angle on spillway scour have been acknowledges in some earlier studies but are not incorporated consistently. In addition, most existing scour formulae for prototype are not developed using a complete dimensional analysis. When the number of relevant parameters increases, a formal dimensional analysis,

such as equation (3.5), needs to be adopted to provide a physical foundation for data analysis and to develop new scour formulae.

(2) From a process-based point of view, equation (3.5) used for plunge pool scour could be adopted for prototype erosion problems in the field. However, practically there are several major difficulties that need to be resolved. First of all, all laboratory studies on plunge pool scour use grain diameter as length scale to nondimensionalize local flow forcing (e.g., equation (3.4) or (3.6)). In such as formulation, the Froude number in fact becomes the ratio of two competing forces, namely, the driving force for scour in the numerator and the stabilizing force due to sediment buoyant weight in the denominator, e.g.,

$$F_r = \frac{U}{\sqrt{g'd}} \sim \frac{\text{impenging jet velocity}}{\text{immersed weigt of sedimet}} \sim \frac{\text{driving force}}{\text{stabilizing force}}$$

Using grain diameter as length scale is a plausible way to characterize the stabilizing force for sand or other cohesionless sediments. However, in the field, the sediment bed maybe of rock or fully consolidated clay (cohesive sediment). The stabilizing force for rock or clay soil is determined by intense internal bonding among particles and hence can not be solely described by its immersed weight in water. For cohesive sediment or rock, it is unclear whether one can define a simple parameter to characterize the stabilizing force based on soil strength tests (such as the EFA described in Section 2.4 for bridge scour).

(3) The effect of jet air content has not been addressed in the field condition. Even in the laboratory condition, the air is equally mixing with water in the jet. However, in field condition the air is mixing unequally with water as the jet flow coming down the spillway chute. Therefore, laboratory studies on jet air content as well as the formulae suggested (e.g., equation (3.9)) can only be used qualitatively at this point.

3.4.2 Plan for new field experiments

In order to better understand and further predict erosions below spillway or culvert outlets in the prototype field condition, new field experiments with careful planning are warranted. We recommend here to conduct a set of field experiments at selected sites in the District with several objectives. The main objective is to

- Obtain a complete field data set for erosions below spillways or culvert outlet structures, including bed scour processes, hydrodynamics, flow forcing and upstream flow conditions for at both sandy and muddy sites.

Using newly measured field data, further objectives are to

- Evaluate and calibrate existing scour formulae for erosions below spillway and culvert outlet structure.

- Incorporate several new physics into the existing formulae guided by process-based laboratory studies.

- Test the feasibility of extending the idealized process-based scour formulae to field/prototype condition.

Regarding the main objective, we propose to conduct a more complete field experiment using new sensors and guided by process-based laboratory experimental findings. Prior field experiments are designed to fit simple formulae developed in the earlier years and hence only limited flow and sediment parameters are measured. We believe that recent laboratory studies on plunge pool scour has reveals some important physical processes that need to be further investigated in the field condition and incorporated in predictive formulae in the future. We recommend conducting two set of field experiments, one at a sandy (non-cohesive sediment) site and the other one at a fully consolidated muddy (cohesive sediment) site. The specific locations will be later determined by consulting with the District. In each site, full bed survey at several instants (e.g., 1, 2, 5, 10, 20, 60, 180, 360 minutes) around the scour hole, including the downstream ridge will be recorded. Several acoustic sensors (2- or 3-components ADV) will be used to measure the flow velocity at

A. one or two locations along the jet trajectory to monitor the decay of jet velocity and final impact velocity.
B. one locations upstream of the scour hole to monitor the upstream flow conditions so that the effect of upstream flow condition can be studies (e.g., equation (3.12)).
C. one or two locations downstream of the scour hole, to monitor the flow condition and transport of sediment near the ridge. This will allow us to study the effect of the downstream ridge on the scour hole development.

Other measurements on the jet flow rate, shape (cross-sectional area), impact angle and tail water depth will also be conducted using traditional methods. Depending on District's interest, there is possible to also measure the void ratio (air content) in the jet flow using techniques well-developed in the surf zone processes. This will allows us to further characterize the air content in the jet. In addition, few OBS (optical backscatter sensor) can be deployed to measure the suspended sediment concentration (at same locations with the ADV described in B and C to see if sediment transport is initiated other than the local scour locations. This information will be related to whether the upstream flow is bringing in sediment into the local scour processes and its effect on local scour (i.e., clear-water or live-bed scour).

At the sandy site, samples will be taken to characterize the grain size and sorting coefficients. At the muddy site, it is expected that the problem is more complex. University of Florida has an in-house EFA system similar to that described in section 2.4 (this system at UF is developed by Prof. Sheppard in the Civil and Coastal Engineering Department). Samples of clay will be taken from the field site and tested in the laboratory of UF to characterize the strength of the clay.

The proposed new field experiments will provide the most comprehensive forcing, hydrodynamic and resulting scour and sediment transport processes which will allow us to not just validate/calibrate the existing scour formulae but also develop improved parameterizations on several new physical processes that have not been incorporated in the existing scour formulae but has been demonstrated to be important in the laboratory plunge pool scour experiments.

4 River Bank Erosion

4.1 General

River bank erosion, specifically at locations immediately downstream of the hydraulic structure, is another critical erosion problem in the District. Intense rainfall and flooding events can trigger sudden changes of stream flow intensity and causes bank erosion. On the other hand, land use or stream management, such as over-clearing of river bank vegetation can also trigger bank failure. River bank erosion has conventionally been studied in the context of fluvial geomorphology. Specifically, bank erosion is an important component for predicting river width adjustment in a time-dependent numerical modeling system for river channel morphology (e.g., Darby and Thorne 1996a, b; Nagata et al. 2000; Duan et al. 2001; Darby et al. 2002).

However, our current understanding on the mechanisms involved in bank erosion, specifically the mass failure, remain to be qualitative, despite several pioneering efforts has been put forward to improve our existing quantitative understanding (e.g., ASCE Task Committee 1998a,b, and reference therein). In this report, we will review the major findings in these studies and discuss the difficulties in characterizing the relevant parameters of a natural river system.

The overall stream flow, river morphology and local erosion are one inter-related system. Without considering the local flow disturbance due to hydraulics, one can predict the river bank stability as part of the width adjustment processes. There are existing empirical formulae (e.g., Huang and Warner 1995; Huang and Nanson 1998) that relate the river width with flow discharge, channel roughness, slope and bank material erodibility (i.e., bank strength). These formulae, when compared with measured data, have rather large uncertainties, because they attempt to parameterize a great amount of processes from small- to large-scale. Despite such empirical approaches are too simple for the present purposes, it can provide useful guideline to evaluate the vulnerability specific locations of a river, especially where District's hydraulic structures are installed.

The river bank erosion or the so-called bank stability consists of several sub-processes, including fluvial erosion, bank failure and basal removal (ASCE Task Committee 1998a). The fluvial erosion refers to removal of sediment at the river base and side banks. It is characterized by river flows imposing boundary layer shear on the river bed/bank causing sediment transport through bedload and suspended load processes. The fluvial erosion often results in the steepening of the bank slope and erosion of the bank toe, which eventually induces mass failure of the river bank soil and the river widening. The mass failure depends on the balance between gravitational force and friction/cohesion forces of the soil that resist the down-slope movement. It is generally characterized into planar failure, rotational failure, toppling failure, cantilever failure and more complex piping/sapping type failure (e.g., Darby et al. 2000), which are discuss in more details in the next section. The wasted sediment mass deposited into the toe or basal area can be entirely, or partially transported downstream. This is the basal removal stage. The balance between the removed deposits due to downstream flow and delivered debris due to bank failure determines the medium- to long-term retreat rate of the bank or the possibility of the next episodic bank failure (Thorne 1982).

The entire erosion processes is further complicated when considering stratified bank soils (layered sand and cohesive soils), vegetation (Thorne et al. 1997), seepage effects, and man-made measures, such as sand piping (Hagerty et al. 1995).

4.2 Bank Mechanics

4.2.1 Fluvial Erosion

Stream flows entrain and transport sediment away from the river base and bank, increase the bank slope, destabilize the bank toe and eventually cause bank failure. Hence, understand fluvial erosion under a given flow and flood hydrograph is the fundamental step toward effective diagnosis for potential failure location or the so-called “hotspot”. In natural river, identifying such local hot-stop is non-trivial and may first require a large-scale numerical computation of fluvial hydraulics (e.g., Darby et al. 2002). In our case, our analysis is more localized to regimes downstream of the hydraulic structures.

However, to quantify fluvial erosion, information on bottom stress distribution over the river base/bank, main flow, secondary flow structures as well as flow turbulence must be obtained. Detailed field measurements or 3D numerical simulation (after model validations) can be utilized to obtain the required information. As described in the previous sections, the bottom stress is used to estimate sediment transport rate using a given sediment transport formula which generally requires specification of empirical coefficients and the critical bottom stress (erodibility). In alluvial bank, the deposition is stratified in a general fining-upward sequence and the erodibility of bank material varies with elevation.

4.2.2 Mass Failure

When significant bank toe is eroded or when the bank slope becomes steepened by fluvial sediment transport processes, episodic mass failure occurs. Mass failure generally relocate bank materials into the near-bank and basal regimes and hence effectively reduces the bank slope and enhance the subsequent stability of the newly-widened bank. The mass failure is a complex process that depends on various flow condition and bank materials and must be analyzed with a local, physically-based approach.

The Planar Failure often occurs for relative steep river banks (Fig 7). The analysis usually involved force balance on a potential failure plane (dashed curve in Fig 7), which gives a critical height for mass failure (e.g., Lohnes and Hardy 1968; Osman and Throne 1988). Recently, more detailed analysis on Planar Failure have been proposed by Darby and Thorne (1996), Darby et al. (2000) and Duan (2006), including some probabilistic approach. From an analytical point of view, Planar Failure has received most attentions compared with other type of failures.

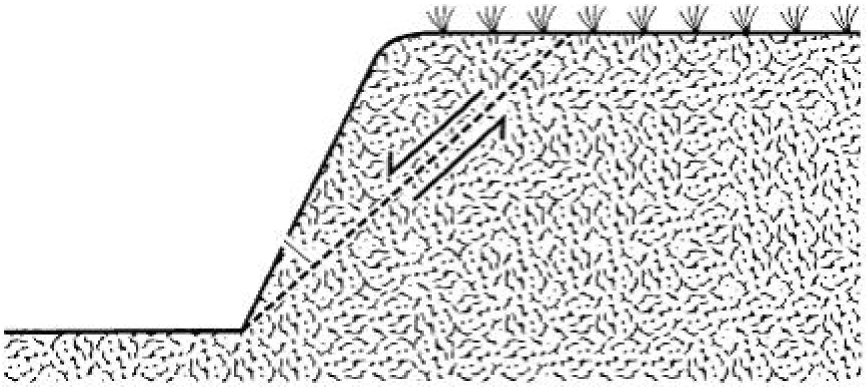

Fig 7. Planar failure occurs for relatively steep bank slope. The dashed line denotes the failure surface (Darby et al. 2000).

For banks with relatively mild slope (<60 degree), the failure slip surface is curved and is defined as Rotational Failure (Fig 8). Rotational failure can be further characterized as a base, toe or slope failure depending on where the failure arc intercepts the ground surfaces (ASCE Task Committee 1998a). Earlier analyses are based on conventional geotechnical procedures (Bishop, 1955). Later, Thorne (1982) developed a stability analysis of the slip circle called Method of Slice, which can be used as predictive guideline.

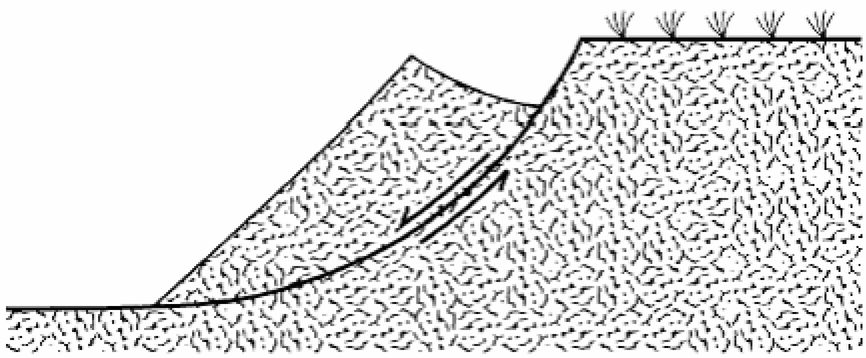

Fig 8. Rotational failure occurs for relatively mild bank slope (Darby et al. 2000).

In a stratified or composite bank (different layers of erodibility soil), the lower layer may be more erodible and undermines the overlying, more erosion-resistant layers. This is called Cantilever Failure. Fig 9 illustrates one such common scenario that a cohesive soil layer is overlying a non-cohesive sand layer. The sand layer is more easily to be eroded away and possibly by fluvial erosion process. Eventually, the overhanging bank fails due to excess gravity force or moment and tensile shear through shear failure, beam failure or tensile failure depends on the cohesion of the overhanging layer, vegetation, and flow condition. Analysis of Cantilever Failure can be found in Thorne and Tovey (1981).

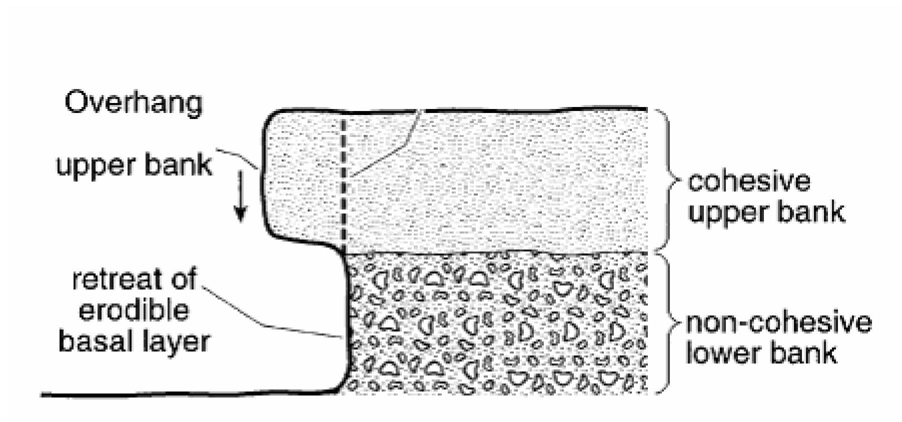

Fig 9. Cantilever Failure occurs in a stratified or composite river bank. The lower bank of cohesionless sand material is more erodible compared to the upper bank made of cohesive soil. The dashed line denotes the failure surface (Darby et al. 2000).

If piping or sipping is introduced in the sand bank. The sand layer is more erodible or more importantly destabilized by seepage outflow. In this case, the sand layer can also undermine the fine-grained upper soil layer (Hagerty 1991). This type of bank failure is more specific with respect to the site condition but shall not be overlooked.

4.2.3 Basal Removal

The failed bank sediment once deposited into the toe or basal area can be entirely or partially transported downstream. As described in the previous section, mass failure can be considered as an episodic event that changes the river geomorphology to another equilibrium state as far as the bank stability is concerned. The rate at which the deposited sediment to be eroded away determines how fast the next bank failure may occur. If the stream flow is not able to remove the debris downstream (or there are upstream supply of sediment), a berm or bench of failed material develops and bank is stable (Thorne 1982). Therefore, despite mass failure is a local process, its prediction, especially at more long-term scale, is closely related to the long-term fluvial sediment transport and geomorpology.

As was noted in ASCE Task Committee (1998a), for river banks of composite layer or man-made channel (such as piping), the basal removal guideline may not be useful. The process in composite bank is often more complicated because the upper bank mass failure can continuous occur even when basal sediment or bank toe is stable (Hagerty et al. 1991).

4.2.3 Other Critical Processes

Two crucial, but less understood processes controlling bank erosions are discussed in this section.

Seepage Effect: The effects of pore-water movement within the river bank are important to bank erosion by is often overlooked (ASCE Task Committee 1998a). Seepage effects is the most prevailing during and following a high stream flow event. As flood water rises, the seepage flow enters the banks due to enhanced hydraulic head. However, as the flood recedes the hydraulic gradient reverses and drives the seepage flow out of the banks and into the stream. During the bank drainage stage, the outflow seepage destabilizes the bank sediment and transports sediment away from the bank. The seepage effect may contribute to lots of bank failure event during inundation of bank soil followed by rapid drops of water level after flooding. For bank of composite layer of sediment, seepage effect is of special concern because the permeability of sand layer is much higher then the overlying cohesive soil layer.

Vegetation Effect: The effects of vegetation on bank erosion are complex and poorly understood (ASCE Task Committee 1998a). Earlier studies (e.g., Carson and Kirkby 1972; Smith 1976) suggest that well-vegetated bank is one or two order of magnitude more stable than the unvegetated banks due to for example, restrain of soil by strong root system and reduced near-bank flow velocity. However, more recent studies on bank vegetation conclude that vegetation may have either a positive or negative effect on bank stability (Thorne et al. 1997). For example, the roots may invade cracks of the soil or rock and weaken the soil structure, or the weight of the vegetation itself may significantly enhance the gravitational force and destabilize the bank. It is generally believe that the effect of vegetation on bank stability can not be well-understood until other critical effects mentioned before are first quantified (Darby and Thorne 1996b).

4.3 Recommendation

River bank erosion, specifically the mass failure process is highly complex and of episodic nature. The complexities can be appreciated simply from the various failure types discussed in section 4.2. A complete study requires careful consideration in several key factors including the variability in soil properties (e.g., cohesion, permeability), composite nature of the bank (see Fig. 9), the vegetation effect and the turbulent flow fields near the bank (including secondary flow), etc. Therefore, to study the river bank erosion problem downstream of District's hydraulic structure, we recommend a two-stage study.

The first stage shall focus on a bulk survey at selected sites but without getting into the detailed flow structure and sediment measurements. Consultation with District's scientists/engineers shall start early in the investigation to identify several key locations downstream of District's hydraulic structures. A preliminary survey will be conducted at these sites, which includes

(1) Bathymetry survey downstream of the hydraulic structure. Acoustic sonar survey will provide comprehensive background information, such as the bank slope and more importantly the existing erosion condition at the bank toe. As described in section 4.2.1, the bank slope and the stage of bank toe erosion is the most important syndrome for potential mass failure.

(2) Historical hydrograph information on water depth and stream flow velocity during flooding condition. Few point measurements of flow velocity around the river bank downstream of the hydraulic structure are also necessary in order to estimate the local accelerated flow velocity (compared to the flow velocity far from the structure) due to the presence of hydraulic structure.

(3) Soil sampling at the river bank, including coring. This includes identifying the cohesion of the soil (cohesive sediment) or the average grain size (non-cohesive sediment) and the characterization of the layer structure of the bank. As shown in Fig 9, cohesive sediment layer overlying a sandy layer can cause cantilever failure.

The preliminary survey can assist us to obtained critical background information on the selected site and the vulnerability of the river banks that are useful for the District.

The second stage of the field study focuses on detailed measurement at one selected site. As discussed in section 4.2.3, the seepage effect on bank erosion and mass failure is the least studied area (ASCE Task Committee 1998a). However, there is no doubt that the seepage effect is a crucial mechanism determining the bank failure processes due to numerous evidences that bank failures often take place soon after the inundation of bank soils followed by rapid decrease of water level. Therefore, we suggest to studying the seepage effects on bank erosion as the major focus of the field investigation.

The first-stage preliminary survey results will provide the most appropriate site for detailed study and the background information on the selected site. Detailed measurements on bathymetry, flow velocity field, sediment suspension and seepage flows around smaller area downstream of the hydraulic structure will be conducted during a regular stream flow condition (before flood), a flooding condition and waning condition (after flood). Specific quantities that will be measured are

(1) Three-dimensional flow velocity measurement near the river bank, including secondary flow structure will be measured during regular flow condition. This will assist identifying the general bottom stress distribution and erosion pattern (such as bank toe erosion) without (before) the mass failure. We plan to deploy Acoustic Doppler Current Profiler to measure the stream flow velocity. Several three (or two) component ADV will be deployed to measure high frequency turbulent flow and secondary flows.

(2) Seepage velocity and pore pressure measurements within the river bank will be conducted. Detailed in-situ sampling will be used to measure the permeability of the soil in the bank. (Mark, please say more)

(3) Detailed bathymetry survey will be conducted before and after the flooding event (and possibly the bank failure event).

(4) We will conduct CFD numerical modeling to characterize more detailed 3D flow structure around the bank and seepage flow within the river bank.

Through detailed measurement, we will be able to understand the fluvial erosion processes around the bank, the seepage flow in the bank at different stages of the stream flow and the bathymetry response of the river bank and base. Measure flow and soil parameters will be used to test several existing analysis on mass failure (e.g., Throne 1982; Osman and Throne 1988).

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