5-2005

Minimum Velocities for the Suspension of Fine Sediment in the Green River Canal

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MINIMUM VELOCITIES FOR THE SUSPENSION OF FINE
SEDIMENT IN THE GREEN RIVER CANAL

by

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A report submitted in partial fulfillment
of the requirements for the degree

of

MASTER OF SCIENCE

in

Civil Engineering

Approved:

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UTAH STATE UNIVERSITY
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2005

ABSTRACT

MINIMUM VELOCITIES FOR THE SUSPENSION OF FINE

SEDIMENT IN THE GREEN RIVER CANAL

by

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Utah State University, 2005

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This paper focuses on a canal in the Gunnison Valley, located in central Utah, which diverts water from the sediment-laden Green River. Grain size analyses were performed on sediment samples taken from the canal. These grain size analyses were used to determine the grain size distribution of the fine sediment, classify the fine sediment, and compare the fine sediment to a sediment deposit in the Green River. The critical incipient velocity for the D_{50} and D_{100} and the critical grain entrainment velocity corresponding to the D_{95} were determined through flume experimentation. Two methods, developed by prior researchers, were chosen by the author to calculate the critical incipient velocity for the D_{50} and D_{100} of the fine sediment in the Green River Canal. The calculated critical incipient velocity for the D_{50} and D_{100} was compared to the critical incipient velocity for the D_{50} and D_{100} as determined from flume experimentation. Using flume data and one of the previously mentioned methods, the critical grain entrainment

velocity was calculated and compared to the critical grain entrainment velocity as determined by flume experimentation. This study concluded that a minimum required average velocity of 1.14 feet per second will retain in suspension the fine sediment sampled from the Green River Canal. This minimum average critical grain entrainment velocity corresponds to the D_{95} of the fine sediment deposited in the Green River Canal. However, it is recommended that further research be conducted to determine if critical incipient velocity formulae accurately estimates the critical grain entrainment velocity. If so, the further research should address the grain diameter that should be used in the calculations.

(74 pages)

ACKNOWLEDGMENTS

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INTRODUCTION

Sedimentation in canals is a common problem. Proper canal design and operation will mitigate the effects of sedimentation by ensuring that the sediment will be transported by the water. The purpose of this paper is to determine the minimum velocity required to transport fine sediment in the Green River Canal that continues to fill with fine sediment deposits. The Green River Canal diverts water from the Green River in the Gunnison Valley, located in central Utah, near the town of Green River. The canal is approximately 10 miles long and ranges in width from 15 ft to 20 ft, with an average slope of 0.00039 feet per foot near the northern end of the canal. The canal transports water for both agricultural and municipal use. The main source of fine sediment into the canal is due to the high concentration of fine suspended sediment in the Green River (Allred, 1997). At the headwaters of the canal, a settling basin was built to settle the fine sediment before entering the canal. At the end of the settling basin, turbines have since been installed for the purpose of power generation. These turbines have increased the water velocity in the settling basin. If the velocity in the settling basin is greater than the velocity required to deposit the fine sediment, the fine suspended sediment concentration intake into the canal may be increased. Other possibilities that may contribute to the increased suspended sediment concentration intake into the canal are geomorphic changes occurring in the Green River. The location map and photographs of the canal may be reviewed in Appendix D.

A possible solution to mitigate sedimentation in the Green River canal is to operate the canal at a minimum velocity that will transport the sediment introduced into the canal. This paper differentiates between two different minimum velocities that must

be considered for proper canal operation. These minimum velocities are the critical incipient velocity and the critical grain entrainment velocity. The minimum velocity at which grain particles just begin to move is defined as critical incipient velocity. The minimum velocity that retains all sediment grains in suspension is defined as the critical grain entrainment velocity.

Critical incipient velocity is a function of the sediment grain diameter. The critical incipient velocity for a particular sediment grain diameter is different than the critical incipient velocity for another (i.e. the critical incipient velocity for the D_{50} will be greater than the critical incipient velocity for the D_{40}). Twenhofel (1939), referring to work performed by a previous researcher, claimed that the velocity required to sustain a grain in motion is less than the velocity required to initiate the motion. Twenhofel (1939) demonstrated that the critical incipient velocity for a sediment grain the approximate size of a hazelnut was 1.35 meters per second. Once the sediment grain was in motion, a velocity of 0.923 meters per second retained the sediment grain in motion. Therefore, the critical grain entrainment velocity has a magnitude less than the critical incipient velocity for the D_{100} of the fine sediment grains that are to be transported by the water. This idea was implied by Lindley, as quoted in Cheema et al. (1997), who stated "under any set of conditions, there is some latitude in difference between velocity that just fails to cause scour and that which just suffices to prevent deposit". The critical grain entrainment velocity will provide canal operators with the minimum velocity to operate in canals which will transport fine sediment and not allow deposition.

The first objective of this paper is to present the grain size distribution of the sediment samples taken from the canal and classify the soils. The second objective is to

present critical incipient motion in terms of the minimum velocity and shear stress for the D_{50} and the D_{100} of the sediment samples taken from the canal as determined by flume experimentation. The third objective is to present the minimum velocity and shear stress required to retain the fine sediment in suspension as determined by flume experimentation. The fourth objective is to compare the critical incipient velocity, determined by the author, with critical incipient velocity relationships developed by prior researchers. The fifth objective is to determine the minimum velocity to operate in the Green River Canal such that the fine sediment introduced into the canal will not deposit. The sixth and final objective is to determine an acceptable, applicable method which canal operators may determine the minimum velocity to operate in canals to reduce deposition.

BACKGROUND

Incipient Motion

Non-Cohesive Soils

From the literature reviewed by the author, there exist many formulae that are used to calculate critical incipient motion of fine grained, non-cohesive soils. These researchers and their derived methods calculate the critical incipient velocity and do not directly calculate the critical grain entrainment velocity. Minimum velocities to retain solids in suspension have been studied, but are limited to pipelines and slurries (Spells, 1955).

Some formulae that calculate critical incipient motion are in terms of velocity and others in terms of shear stress. The reasons for the different equations are due to the assumptions used by researchers, the terms researches use to define the hydrodynamic

forces, and how researchers manipulate their equations. The hydrodynamic forces all researchers agree upon are the drag force (F_D), the lift force (F_L), and the submerged weight of the grain (W_S). The author chose two previous researchers, their methods, and their resulting critical incipient velocity formulae to compare against the critical incipient velocity the author measured during flume experiments. The two researchers chosen were Dingman (1984) and Baker (1980).

<u>Dingman's Analysis</u>

Dingman (1984) begins by deriving a dimensionless ratio θ_e from the erosive force and the submerged weight of the grain. The erosive force is defined by Dingman (1984) as the vector sum of the lift force and the drag force:

$$F_E = K_2 * \frac{\rho * V^{*2}}{2} * D^2 \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots(1)$$

Where:

K_2 is a constant of proportionality

ρ is the density of water

V^* is the friction velocity

D is the grain diameter

The submerged weight as defined by Dingman (1984) is:

$$F_g = K_1 * (\rho_s - \rho) * g * D^3 \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots(2)$$

Where:

K_1 is equal to $\pi/6$ for a sphere

ρ_s is the density of the sediment

g is the acceleration due to gravity

D is the grain diameter

Particle motion depends on the relative magnitudes of the two opposing forces. Therefore, Dingman (1984) ignored the constants and defined the dimensionless ratio θ_e:

$$\theta_e = \frac{\rho * V^{*2} * D^2}{(\rho_s - \rho) * g * D^3} = \frac{\rho * V^{*2}}{(\gamma_s - \gamma) * D} \quad \text{...(3)}$$

Where:

γ_s is the weight density of the sediment

γ is the weight density of water

The above equation can be related to the dimensionless shear stress first developed by Shields (1936) through the relationship:

$$\tau_o = \rho * V^{*2} \quad \text{...(4)}$$

Where:

τ_o is the average bed shear stress

Shields (1936) who also studied incipient motion concluded that the initiation of sediment was a function of two dimensionless numbers, the dimensionless shear stress and the particle shear Reynolds number. The dimensionless shear stress equation is:

$$\tau_* = \frac{\tau_o}{(\gamma_s - \gamma) * D} \quad \text{..(5)}$$

The equation for the particle shear Reynolds number according to Grayson et al. (2004) takes the form of:

$$\mathfrak{R}^* = \frac{V^* * D}{\nu} \quad \text{...(6)}$$

Where:

ν is the kinematic viscosity of the water

Yalin and Karahan (1979) using Shields data as well as other data determined that for particle shear Reynolds numbers greater than 70 the dimensionless shear stress is a constant value of 0.044. Using Yalin and Karahan's (1979) data Dingman (1984) was able to formulate the critical shear force as a function of the grain diameter:

$$\tau_{oc} = 713 * D \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots(7)$$

Where:

τ_{oc} is the critical shear force in newtons per square meter

D is the grain diameter in meters

In order to express incipient motion as an average velocity Dingman (1984) expressed the above equation as a function of the hydraulic radius (R) in meters and slope (S):

$$D_c = 13.7 * R * S \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots(8)$$

Where:

D_c is the critical sediment diameter to be eroded

The above equation is only valid for product values of the hydraulic radius and slope greater than or equal to 1.5E-04 meters. For product values of the hydraulic radius and slope less than 1.5E-04 meters, see Figure 8.7 of Dingman (1984).

Dingman (1984) then uses a vertical velocity profile for turbulent flow which yields the critical erosive velocity when used with the associated hydraulic radius and slope determined from Figure 8.7 of Dingman (1984):

$$V = 2.5 * V^* * \left[\ln\left(\frac{Y}{y_o}\right) - 1 \right] \dots\dots\dots\dots\dots\dots\dots\dots(9)$$

Where:

Y is the total flow depth

$$y_o = \frac{0.11 * \nu}{V^*} \text{ for smooth flow } \dots\dots\dots\dots\dots\dots (9.a)$$

$$y_o = 0.033 * k_s \text{ for rough flow} \dots\dots\dots\dots\dots\dots (9.b)$$

Smooth flow occurs when the thickness of the laminar sublayer (y_l) is greater than or

equal to the roughness height (k_s) which can be approximated by (D_c). Rough flow

occurs when the laminar sublayer is less than the roughness height. The thickness of the

laminar sublayer may be calculated using the following equation:

$$y_l = \frac{4 * \nu}{V^*} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (10)$$

The process to determine critical incipient velocity using Dingman's (1984)

method is rather simple. A hydraulic radius and slope must first be specified. Once the

slope and hydraulic radius is specified the particle shear Reynolds number may be

calculated and the critical sediment diameter determined from Figure 8.7 of Dingman

(1984). The laminar sublayer thickness is then calculated and checked against the

roughness height. Dingman (1984) assumes that $k_s = D_c$ therefore, y_o may be

calculated based on rough flow or smooth flow. The critical incipient velocity is then

easily calculated. Figure 8.8 of Dingman (1984) depicts the average critical velocity as a

function of particle diameter for a range of hydraulic radii from 0.1 meters to 10 meters.

Figures 8.7 and 8.8 of Dingman (1984) may be reviewed in Appendix A.

Baker's Analysis

Baker (1980) analyzed the hydrodynamic forces acting on a grain particle at rest

for scour around bridge piers. Baker (1980) assumed the drag force, lift force, and the

submerged weight of the grain to be (respectively):

$$F_D = C_D * \frac{\rho * V_c^2}{2} * \frac{\pi * D^2}{4} \quad\text{...(11)}$$

$$F_L = C_L * \frac{\rho * V_c^2}{2} * \frac{\pi * D^2}{4} \quad\text{...(12)}$$

$$W = \frac{\pi * D^3}{6} * (\rho_s - \rho) * g \quad\text{...(13)}$$

Where:

V_c is the characteristic velocity acting on the grain

C_L is the lift coefficient

C_D is the drag coefficient

Baker (1980) assumed that incipient motion occurs when the moments created by the drag force and lift force are equal to the moment created by the submerged weight of the grain summed about the point of contact between grains. Baker (1980) and Wiberg and Smith (1985) assumed that the point of contact between the grains is the particle angle of repose and used an angle of 60° to represent it. Wiberg and Smith (1985) claim the particle angle of repose is a measure of resistance that needs to be overcome to move a grain at rest on an alluvial bed. Wiberg and Smith (1985) further claim that the particle angle of repose is not to be confused with the particle's mass angle of repose which is about 35°. Baker (1980), using his definitions of the hydrodynamic forces and assumptions, derived the formula to calculate the velocity that causes incipient motion using a particle packing angle of 60°. Grayson et al. (2004) presented Baker's (1980) work and presented the critical incipient velocity equation as a function of the particle packing angle and called it the Simple Rotational Model (SRM). The equation takes the form:

$$V_{c-bed}^2 = \frac{4*\sin(\phi_o - \varepsilon)*(\rho_s - \rho)*g*D}{3*\rho*(C_L*\sin(\phi_o)+C_D*\cos(\phi_o))} \quad \text{..}(14)$$

Where:

ϕ_o is the particle packing angle

ε is the slope of the alluvial bed

Grayson et al. (2004) determined that V_c was the bed critical incipient velocity at a height D/2 above the bed. In order to use the SRM, the drag and lift coefficients for the particle are needed. The drag coefficient for spherical shapes has been determined and plotted as a function of the particle Reynolds number Chang (1988), Crow et al. (2001). The equation to determine the particle Reynolds number is:

$$\Re = \frac{\omega_s*D}{\nu} \quad \text{...}(15)$$

Where:

ω_s is the particle's fall velocity

Cliff and Gauvin (1970) as cited in Crow et al. (2001) developed a formula to directly solve for the drag coefficient as a function of the particle Reynolds number for particle Reynolds numbers greater than 3.0E05. The equation takes the form of:

$$C_D = \frac{24}{\Re}*(1+0.15*\Re^{0.687}) + \frac{0.42}{1+4.25E04*\Re^{-1.16}} \quad \text{................................}(16)$$

Church and Ferguson (2004) derived a computationally simple formula by which the drag coefficient may be calculated. The formula takes the form of:

$$C_D = \left(\frac{2*C_1*\nu}{\sqrt{2*(G_s-1)*g*D^3}} + \sqrt{C_2}\right)^2 \quad \text{..}(17)$$

Where:

$$C_1 = 18$$

$$C_2 = 1.0$$

Church and Ferguson (2004) explain that for natural sediments, the constants C_1 and

C_2 take on the values of 18 and 1.0 when using sieve diameters. The lift coefficient may

then be calculated directly using a formula derived by James (1990). The lift coefficient

is a function of the particle shear Reynolds number and the drag coefficient. The

equation takes the form:

$$\frac{C_L}{C_D} = -0.560 + 0.212 * \ln\left(R^*\right) \text{ for } R^* < 150 \dots\dots\dots(18)$$

$$\frac{C_L}{C_D} = 0.5 \text{ for } R^* \geq 150 \dots\dots\dots(19)$$

Since the critical incipient velocity for the SRM is located at a distance D/2 above

the bed it must be converted to an average velocity in a vertical section by some method.

Grayson et al. (2004) suggested the use of the modified Prandtl and Einstein velocity

distribution and mean velocity equations for hydraulically rough flows. The average

critical velocity for the SRM is calculated by:

$$V_{c-avg} = V_{c-bed} * \frac{\log\left(12.27 * \frac{\chi * Y}{k_s}\right)}{\log\left(30.2 * \frac{\chi * \frac{D}{2}}{k_s}\right)} \dots\dots\dots(20)$$

Where:

k_s is taken as D_{65}

χ is the Einstein's multiplication factor

Einstein's multiplication factor is a function of the ratio of the roughness height to the
thickness of the laminar sublayer and can be determined from graphs such as Figure 3.6
of Chang (1988). A curve fit for Einstein's multiplication factor was determined by
Rahmeyer (2005):

$$Y = 1.622653 + 0.099472 * X - 2.83296 * X^2 + 1.189237 * X^3 + 2.566298 * X^4 - 1.64 * X^5$$

$$\text{for } 0.1 < \frac{k_s}{y_l} \leq 8$$

$$\text{for } \frac{k_s}{y_l} > 8 \quad \chi = 1$$

Where:

$$Y = \chi$$

$$X = \log\left(\frac{k_s}{y_l}\right)$$

The SRM originally derived by Baker (1980) and the critical incipient velocity
developed by Dingman (1984) were chosen by the author due to their relative simplicity
and ease of calculation. There are other more complex analyses used to compute critical
incipient velocity such as the analysis by Wiberg and Smith (1987). A study conducted
by Grayson et al. (2004) concluded that the SRM is just as accurate as the other methods,
gave similar results, and is easier to compute the critical incipient velocity for fine, non-
cohesive sediment. According to the author's knowledge no work has been done to
determine the accuracy of Dingman's (1984) method to calculate the critical incipient
velocity of fine grained, non-cohesive sediment.

<u>Cohesive Soils</u>

In addition to hydrodynamic forces there is another force that acts on cohesive soils. Cohesive soils consist of silts and clays which are grains that have diameters less than 0.062 millimeters (mm). However, Dingman (1984) claims that for grains less than 0.1 mm in diameter electrostatic forces are significant. USACE (1995) claim that the force that inhibits cohesive grains to be eroded are due to electrochemical forces and that the erosion rate is a function of the bed shear stress. Although, USACE (1995) never gave the relationship for the erosion rate as a function of the bed shear stress, there has been published data for estimating the maximum permissible velocity for cohesive channels. Chang (1988) defined permissible velocity, "the maximum mean velocity of a channel that will not cause erosion of the channel boundary" and claims that it is often called critical velocity. Table 7.3 of Julien (1995) lists the soil type and the maximum permissible average flow velocity for cohesive channels and may be reviewed in Appendix B.

METHODOLOGY

Definitions

As mentioned in the introduction, this paper focuses on two velocities. These velocities are the critical incipient velocity and the critical grain entrainment velocity. It is necessary to define the terms of incipient motion and critical grain entrainment for the purposes of this paper. Critical incipient velocity will be defined as the minimum velocity which initiates motion of fine grains on the bed according to visual inspection. Critical grain entrainment velocity will be defined as the velocity that retains grains in suspension by the fluid, also according to visual inspection. The difficulty in defining

critical grain entrainment comes from the question: What quantity of sediment is required to be held in suspension by the fluid to be classified as the critical grain entrainment velocity?

Although critical incipient motion formulae have been derived by prior researchers for fine grained, non-cohesive soils through flume experimentation, none of the previous researchers mentioned the minimum velocity required to retain sediment in suspension (critical grain entrainment velocity). Therefore, the author considered it necessary to conduct flume experiments with the fine sediment from the Green River Canal and measure critical incipient velocity and the critical grain entrainment velocity.

Sediment Samples

Two different types of sediment were sampled. The first sample consisted of a cohesive soil which was taken from the downstream end of the Green River Canal and the second sample consisted of a non-cohesive soil which was taken from the upstream end of the canal just downstream of the second inverted siphon. By noting the location of the sediment samples, the non-cohesive sediment is retained in suspension until downstream of the second siphon. The cohesive sediment is then carried further downstream until deposited. This suggests to the author that the canal operates at a velocity which retains the non-cohesive sediment in suspension until a location just downstream of the inverted siphon and that the fluid's ability to suspend the cohesive sediment is further reduced as the fluid continues downstream. If the canal continued to operate at the velocity retaining the non-cohesive soil in suspension there would be no deposition. Also noteworthy is the lack of deposition in the inverted siphons. The velocity in the inverted siphon is sufficient to retain all of the fine sediment in

suspension. Special attention should be given to the areas of deposition and further analysis be carried out to determine the reason for deposition in these areas.

Grain size analyses were performed for these soils using the techniques outlined in Bowles (1992). A hydrometer test was conducted for 50 grams of the cohesive sediment. A hydrometer test was also conducted on 50 grams of the non-cohesive sediment that passed the number 40 sieve. A mechanical sieve analysis was conducted on 200 grams of the non-cohesive sediment in order to obtain the full range of grain sizes for the non-cohesive sediment sample. A specific gravity of 2.65 was assumed for both soil types and used in all the calculations for both the grain size distribution analyses and critical incipient velocity calculations.

Flume Setup

The flume used to determine critical incipient velocity and critical grain entrainment velocity was 24 feet long, 23.25 inches wide, and had a bed slope of approximately zero. The sediment was placed in a test section located 16 feet from the beginning of the flume. The test section was inset 1.25 inches and approximately 5 feet long. The sediment was placed in the test section of the flume so the top of sediment was approximately at the same elevation as the bed of the flume. At the beginning of the flume a 12 inch diameter pipe transitioned into 6 rectangular chutes, which produced undesirable flow conditions. A free fall condition at the end of the flume also produced undesirable flow conditions. An apparatus was installed just downstream of the rectangular chutes and at the end of the flume to break up the flow and create favorable flow conditions through out the entire length of the flume.

Velocity Measurement and Test Methods

The test to determine critical incipient velocity and critical grain entrainment was initiated by introducing water into the flume at a slow rate. The flume was allowed to fill to a depth until the water reached steady state conditions. A vertical velocity measurement was taken and sediment was introduced at the water surface. The retention of suspended sediment was observed and noted. The flow was then increased until critical incipient motion occurred. A vertical velocity profile was then measured with the flow meter. Sediment was then introduced at the water surface and the retention of suspended sediment was visually observed. This process was followed until the bed began to visually scour. Steady state conditions were achieved before any vertical velocity measurements were taken for the above methods.

A Marsh McBirney flow meter was used to measure the vertical velocity profiles. All vertical velocity measurements were taken at the longitudinal center of both sediment samples. Measurements were taken at 5 different vertical stations in the cross-section for the non-cohesive soil. The first vertical velocity profile station was located at the center of the flume. Two vertical velocity profiles were measured at 6 inches and at 10 inches to the left and to the right of the center vertical profile station. The purpose of these 5 vertical stations throughout the cross section was to produce a three-dimensional view of the one-dimensional vertical velocity profile of the flow as a visual aid. Pictures of the flume as well as the three-dimensional view of the one-dimensional vertical velocity profile may be viewed in Appendix C.

RESULTS

Non-Cohesive Soil

Grain Size Distribution

From the grain size analysis the author determined the D_{16}, D_{50}, D_{65}, D_{85}, and D_{90} to be 0.042 mm, 0.12 mm, 0.16 mm, 0.18 mm, and 0.20 mm respectively. The gradation coefficient of the non-cohesive soil is 2.18. Figure 1.0 depicts the percent finer versus the grain diameter on a semi-log plot. The triangle markers represent the data determined from a mechanical sieve analysis on 200 grams of the non-cohesive sediment sample washed through the number 200 sieve. The square markers represent data determined from a mechanical sieve analysis on the coarser fraction of the non-cohesive sediment sample used in the hydrometer test. The diamond markers represent data obtained by a hydrometer test. The percent error for the grain distribution is approximately 0.6% for the hydrometer test and approximately 0.54% for the mechanical sieve analysis.
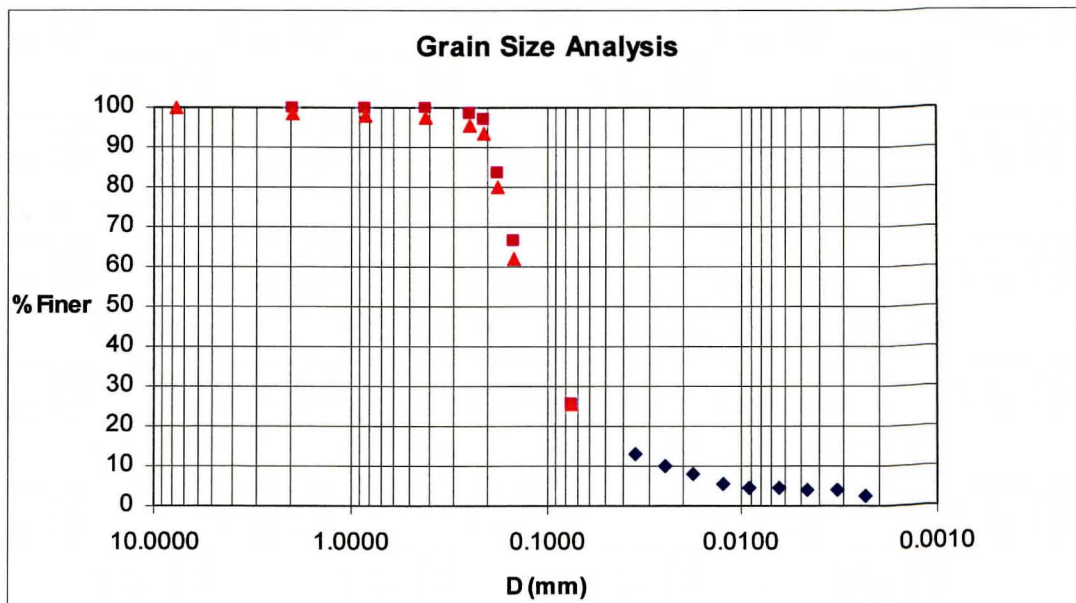

Figure 1.0 Grain size distribution for the very fine sand.

The classification of the non-cohesive soil was classified using the American Society of Civil Engineering (ASCE) sediment size classification. Table 1.0 depicts the size fraction for each sediment class and the related percent of the sample for both the 200 gram soil sample used for the mechanical sieve analysis and the 50 gram sample used in the hydrometer analysis. Both sediment samples are mostly comprised of fine sand ($\approx$ 33%) and very fine sand ($\approx$ 39%). The author classified the soil as very fine sand.

Table 1.0 Percent of sediment sample for the associated size class.

Non-Cohesive Sediment Sample			
Soil Class	Size (mm)	% of 200 gram sample	% of 50 gram sample
Medium Gravel	16.0-8.00	0.145	
Fine Gravel	8.00-4.00	1.25	
Very Fine Gravel	4.00-2.00		
Very Coarse Sand	2.00-1.00	2.865	
Coarse Sand	1.00-0.5		0.02
Medium Sand	0.5-0.25		1.42
Find Sand	0.25-0.125	33.855	32.2
Very Fine Sand	0.125-0.062	36.21	41.02
Coarse Silt	0.062-0.031	25.675	14.556
Medium Silt	0.031-0.016		7.52
Fine Silt	0.016-0.008		0.94
Very Fine Silt	0.008-0.004		0.188
Coarse Clay	0.004-0.002		1.692
Medium Clay	0.002-0.001		0.444
Fine Clay	0.001-0.0005		
Very Fine Clay	0.0005-0.0002		

Velocities for Critical Incipient Motion and Critical Grain Entrainment

Flume experimentation showed that a near bed velocity of 0.4 feet per second caused no motion of grain particles on the bed. This velocity also failed to retain most of the sediment in suspension when the very fine sand was introduced at the water surface. A near bed velocity of 0.7 feet per second initiated grain movement and is designated as the critical incipient velocity for median grain diameter. At a near bed velocity of 0.7

feet per second the very fine sand, when introduced at the water surface, was retained in suspension but at a very weak concentration and some sediment deposited on the bed. A near bed velocity of 0.9 feet per second began to lift small grains off of the bed and it appeared that at this velocity the fluid was able to suspend and retain in suspension all of the fine sediment. A near bed velocity of 1.3 feet per second rapidly lifted grains off of the bed and began to scour the bed. It is the author's opinion, and in agreement with statements made by Lindley as quoted in Cheema et al. (1997) and Twenhofel (1939), that a near bed velocity of approximately 0.9 feet per second is the critical grain entrainment velocity, or the minimum velocity at which all sediment grains are retained in suspension. All of the velocities referred to in this section (near bed velocity) represent the mean local velocity of the fluid measured at a distance of 0.063 feet above the bed.

Shear Stress Conversion

The method used to convert these velocities to a critical shear stress was through the use of the equation derived by von Karman that describes the vertical velocity profile for turbulent flow of both hydraulically smooth and rough boundaries (Rahmeyer, 2005). The equation derived by von Karman takes the form:

$$v = V^* * \frac{2.31}{\kappa} * \log\left(\frac{y}{y'}\right) \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (21)$$

Where:

$$y' = \frac{11.6 * v}{107 * V^*} \text{ for turbulent flow in a hydraulically smooth boundary}\dots\dots\dots\dots (22)$$

$$y' = \frac{k_s}{30.2} \text{ for turbulent flow in a hydraulically rough boundary } \dots\dots\dots\dots (23)$$

v is the velocity at depth y

κ is the von Karman constant and equal to 0.4

This equation allowed the author to set the theoretical velocity equal to the velocity

measured in the flume at a distance of 0.063 feet off of the bed and solve for the required

friction slope. With the friction slope known and assuming a water temperature of 60°

Fahrenheit, the critical shear stress was calculated to be 0.0027 pounds per square foot

and a shear stress for critical grain entrainment to be 0.0043 pounds per square foot.

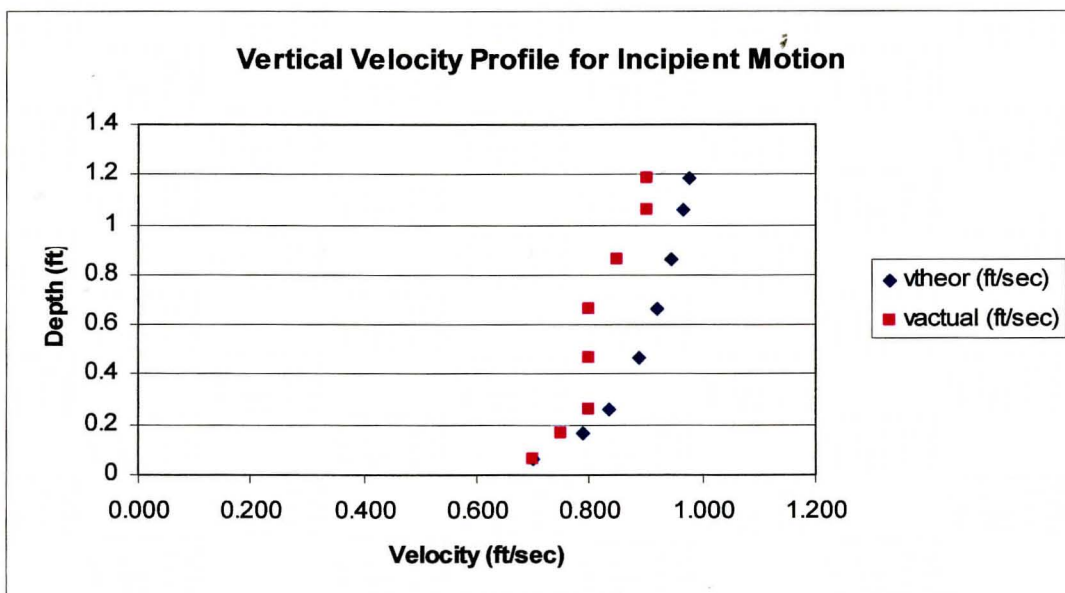Figures 1.1 and 1.2 compare the theoretical and actual vertical velocity distribution.

Figure 1.1 Vertical velocity profile at critical incipient velocity for the very fine
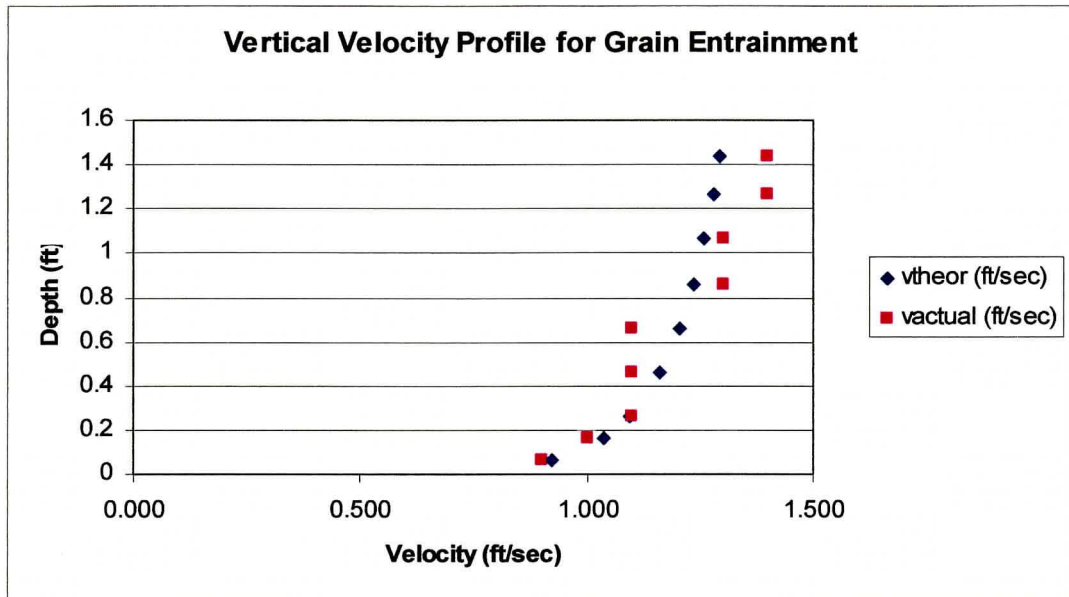
sand.

Figure 1.2 Vertical velocity profile at critical grain entrainment velocity for the very fine sand.

Comparison of Critical Incipient Velocity

Dingman's (1984) method to calculate critical incipient velocity yielded favorable results. An average critical incipient velocity of 0.88 feet per second was calculated using Dingman's (1984) method. This critical incipient velocity correlates to a critical grain diameter of 0.08 millimeters, which is approximately the D_{50} of the very fine sand. Using Dingman's (1984) method, assuming that the roughness height is approximated by the D_{65} and not the D_C, an average critical incipient velocity of 1.6 feet per second was calculated. This correlates to a critical grain diameter of 0.8 millimeters (approximately the D100 of the very fine sand).

Dingman's (1984) method expresses critical incipient velocity as an average critical incipient velocity in the vertical profile. The critical incipient velocity determined by the author through flume experimentation was not an average critical incipient velocity in the vertical profile. The critical incipient velocity determined by the author

was measured approximately 0.063 feet off of the bed. It is necessary, to accurately compare the critical incipient velocities determined by Dingman's (1984) method to the critical incipient velocities determined by the author, to convert the average critical incipient velocities calculated by Dingman's (1984) method to a local mean critical incipient velocity at a depth of 0.063 feet off of the bed.

The procedure by which the author used to convert the average critical incipient velocity to a local mean critical incipient velocity was similar to the method used by Grayson et al. (2004) to convert the critical incipient velocity calculated using the SRM model to a depth average critical incipient velocity. Instead of using equations for hydraulically rough flow the author used equations for hydraulically smooth flow. The resulting equation to convert the average critical incipient velocity to a local mean critical incipient velocity takes the form:

$$V_c = V * \frac{\frac{2.31}{\kappa} * \log\left(\frac{V^* * y * 107}{11.6 * v}\right)}{2.5 * \left[\ln\left(\frac{V^* * Y}{0.11 * v}\right) - 1\right]} \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots(24)$$

Where:

V_c is the local mean critical incipient velocity at a

depth of y

The above method resulted in a critical incipient velocity of 0.699 feet per second for the D_{50} and 1.302 feet per second for the D_{100} of the very fine sand. Both of the critical incipient velocities correspond to a height of 0.063 feet off of the bed and result in an error of 0.09% and 0.17% respectively, when compared to the velocities measured by the author during flume experimentation.

The SRM method calculated a lower critical incipient velocity for both the D_{50} and D_{100} than the critical incipient velocity determined by the author through flume experimentation. The SRM method calculated a critical incipient velocity of 0.12 feet per second for the D_{50} measured at a distance of 197.0E-06 feet off of the bed. The SRM method yielded a critical incipient velocity of 0.42 feet per second for the D_{100}. To convert these critical incipient velocities calculated by SRM method to a critical incipient velocity comparable to that of the author was similar to the above method. The resulting equation is:

$$V_c = V_{c-bed} * \frac{\log\left(\frac{V^* * y * 107}{11.6 * v}\right)}{\log\left[\frac{V^* * \left(\frac{D_{50}}{2}\right) * 107}{11.6 * v}\right]} \quad\text{..}(25)$$

Where:

V_c is the local mean critical incipient velocity at a

depth of y

The above method resulted in a critical incipient velocity of approximately 0.51 feet per second for the D_{50} and 0.82 feet per second for the D_{100} of the very fine sand. Both of these critical incipient velocities correspond to a height of 0.063 feet off of the bed and result in an error of approximately 27% and 37% respectively. The error was nearly 70% when the author used the suggested correction values by Grayson et al. (2004) to James' (1990) data to calculate lift coefficients for the critical incipient velocity for the D_{50} of the very fine sand.

Minimum Velocity for the Green River Canal

The last two objectives of this paper are to determine the minimum velocity to operate in the Green River Canal and an acceptable, accurate method to predict minimum operating velocities for canals in central/southern Utah. To reach these objectives, flume experiments were performed and the above results were found for the very fine sand. Both the critical incipient velocity and the critical grain entrainment velocity that were measured in the flume must be converted to an average critical incipient velocity and an average critical grain entrainment velocity in order to use and measure in the Green River Canal. The method to do so is by locating the distance from the bed that corresponds to the average velocity for the vertical velocity profile. Dingman (1984) found that the average velocity in a vertical profile is located at a distance above the bed equal to the product of 0.37 and the total water depth. The average velocities referred to in this paper correspond to the local mean velocity in the Green River Canal at a distance above the bed equal to the product of 0.37 and the water depth.

With the location of the average vertical velocity known, the critical incipient velocity and the critical grain entrainment velocity determined from flume experimentation can then be related to the Green River Canal. According to the author and the above results, the minimum average velocity to operate in the Green River Canal that will not allow any deposition is the average critical grain entrainment velocity. The average critical grain entrainment velocity is the velocity that has a magnitude less than the average critical incipient velocity for the D_{100} and a magnitude greater than the average critical incipient velocity of the D_{50}. Some deposition of the coarser fraction of the very fine sand will occur if the minimum velocity operated in the Green River Canal

is the average critical incipient velocity that corresponds to a sediment grain diameter of 0.12 millimeters (approximately the D_{50} of the very fine sand).

The average critical grain entrainment velocity was calculated using Dingman's (1984) method with the friction slope and hydraulic radius corresponding to the near bed critical grain entrainment velocity in the flume of 0.9 feet per second. This yielded an average critical grain entrainment velocity of 1.14 feet per second and corresponds to a critical grain diameter of approximately 0.25 millimeters (approximately the D_{95} of the very fine sand). Equation (24) was used to convert the average critical grain entrainment velocity to a local mean critical grain entrainment velocity of 0.913 feet per second corresponding to a height off of the bed of 0.063 feet. An error of 1.5% results from comparing the calculated critical grain entrainment velocity to the measured critical grain entrainment velocity.

Dingman's (1984) method seems to be accurate and in agreement with flume experimentation. The average critical grain entrainment velocity is greater than the average critical incipient velocity for the D_{50} and less than the average critical incipient velocity for the D_{100}. There is a clear difference between the velocity that initiates motion, the velocity that retains sediment in suspension, and the velocity that causes scour as indicated by Twenhofel (1939) and Lindley when quoted by Cheema et al. (1997).

Cohesive Soil

Grain Size Distribution

The grain size analysis yielded the D_{16}, D_{50}, D_{65}, D_{85}, and D_{90} to be 0.0009 mm, 0.005 mm, 0.009 mm, 0.024 mm, and 0.03 mm respectively. The value of the gradation

coefficient is 5.18. Figure 1.3 depicts the percent finer versus the grain diameter on a semi-log plot. The square markers depict the data obtained from a mechanical sieve analysis performed on the coarser fraction of the 50 gram sample. The diamond markers depict the data obtained from a hydrometer test.

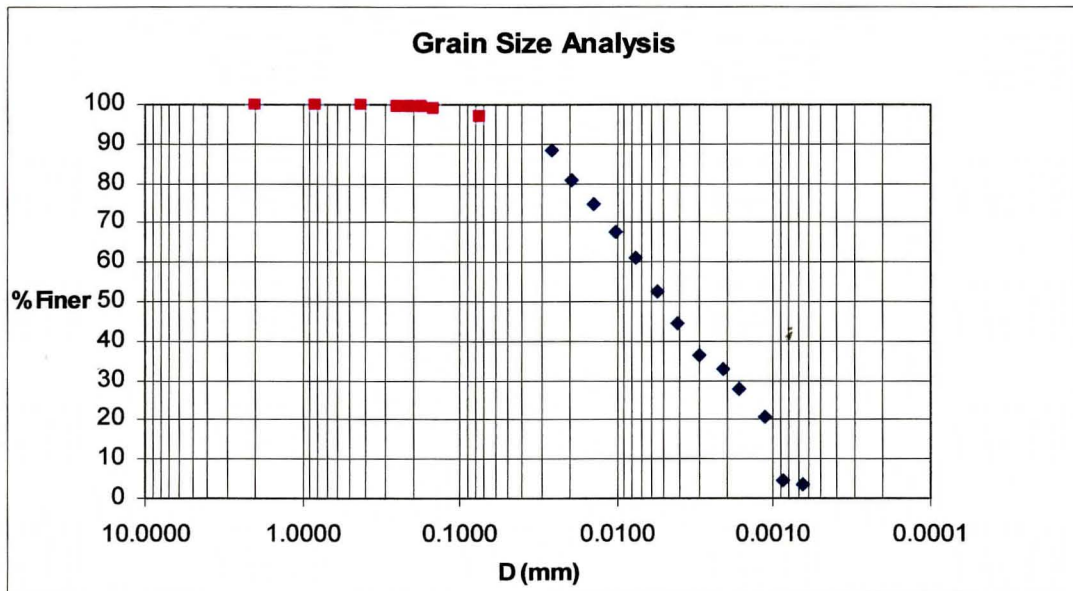**Figure 1.3** Grain size distribution for the very fine silty clay.

The cohesive soil sample was also classified using the ASCE sediment size classification. Table 1.1 depicts the percentage of the 50 gram cohesive sediment sample for each sediment class and the related grain size. The cohesive soil was mostly comprised of very fine silt (23%) and medium clay (28%). The author classified this soil as very fine silty clay.

Table 1.1 Size of sediment sample shown as percent for each soil class.

Cohesive Sediment Sample		
Soil Class	Size (mm)	% of 50 gram sample
Medium Gravel	15.24-7.62	
Fine Gravel	7.62-4.00	
Very Fine Gravel	4.00-2.00	
Very Coarse Sand	2.00-1.00	
Coarse Sand	1.00-0.5	0.02
Medium Sand	0.5-0.25	0.38
Find Sand	0.25-0.125	0.44
Very Fine Sand	0.125-0.062	3.18
Coarse Silt	0.062-0.031	
Medium Silt	0.031-0.016	16.38
Fine Silt	0.016-0.008	13.00
Very Fine Silt	0.008-0.004	23.00
Coarse Clay	0.004-0.002	12.00
Medium Clay	0.002-0.001	28.00
Fine Clay	0.001-0.0005	1.00
Very Fine Clay	0.0005-0.0002	2.60

Velocities for Critical Incipient Motion and Critical Grain Entrainment

A minimum velocity of 0.6 feet per second in the flume initiated motion of very

fine silty clay grains. The author defined this as the critical incipient velocity for the very

fine silty clay. Critical grain entrainment velocity for the very fine silty clay was very

difficult to determine due to the cohesive nature of the sediment. When the sediment was

introduced at the water surface, seldomly was there just a single grain particle retained in

suspension. Most of the particles remained as flocs and not individual grains like the

very fine sand. Due to the fact that the very fine silty clay has grain sizes considerably

less than that of the very fine sand and if the very fine silty clay is already in suspension,

the author assumes that the average critical grain entrainment velocity of 1.14 feet per

second will retain the very fine silty clay in suspension. If the very fine silty clay is

allowed to deposit, a greater velocity will be required to lift the grains off of the alluvial

bed and re-suspend the very fine silty clay due to the cohesive nature of the very fine silty clay. Average velocities in the flume were increased to approximately 2 feet per second and the minimum velocity to cause scouring of the bed was never reached for this soil type. This soil was very cohesive and individual grains were never lifted off of the bed equal to the rate the very fine sand grains were lifted off of the bed.

Shear Stress Conversion

The critical incipient velocity for the medium silty clay was converted to a critical shear stress of 0.0025 pounds per square foot using the von Karman vertical velocity profile equation for a hydraulically smooth boundary (Equation (21)). The von Karman vertical velocity profile does not appear to fit the actual vertical velocity profile measured with the Marsh McBirney current meter. The author assumes that the von Karman vertical velocity profile is still a good representation of the vertical velocity profile in the flume. The author assumes that the apparatus at the downstream end of the flume may have interfered with the vertical velocity profile in some manner. The reason for this assumption is because the apparatus was adjusted slightly different for the flume experiment performed on the very fine sand. Figure 1.4 compares the theoretical and actual vertical velocity profiles for the very fine silty clay.
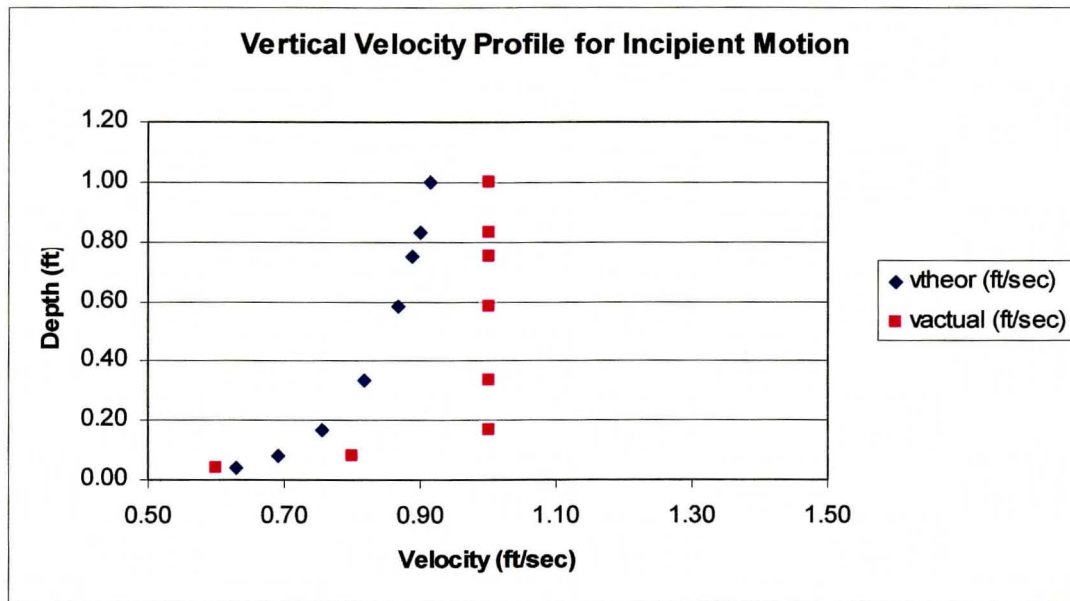
Figure 1.4 Comparison of the theoretical and actual vertical velocity profiles.

Comparison to Maximum Permissible Velocities

According to Table 7.3 of Julien (1990) the maximum permissible average

velocity for fine sandy loam clay is in the range of 1.5 to 3 feet per second. Using

Equation (24) this average velocity may be converted to a local mean velocity at some

depth y. Assuming an average vertical velocity of 1.5 feet per second, Equation (24)

yields a local mean velocity of 1.2 feet per second at a corresponding depth of 0.063 feet

off of the bed. This results in a velocity that is considerably more than the critical

incipient velocity as determined by the flume experiment. Table 7.3 of Julien (1990), in

Appendix B, may be reviewed for maximum permissible velocities for other cohesive

soil types.

Summary of the Results

Table 1.2 gives a synopsis of the results obtained by the author through flume

experimentation and the results as determined by methods previously derived by prior

researchers. Dingman's (1984) method of estimating critical incipient velocity compares

well with the data obtained from flume experimentation and is recommended by the

author to be a reliable indicator of incipient motion.

Table 1.2 Synopsis of results section. All values were measured at a height of 0.063 feet off of the bed.

All measurements at a height of 0.063 feet off of bed			
Category	Method	Soil Type	
		very fine sand	very fine silty clay
Critical Incipient Velocity of the D_{50} (ft/sec)	Flume	0.7	0.6
	Dingman's	0.699	N/A
	SRM	0.51	N/A
	Table 7.3 of Julien (1990)	N/A	1.2
Critical Incipient Velocity of the D_{100} (ft/sec)	Flume	? 1.3	N/A
	Dingman's	1.302	N/A
	SRM	0.82	N/A
	Table 7.3 of Julien (1990)	N/A	N/A
Critical Incipient Shear Stress for the D_{50} (lbs/ft^2)	Flume	0.0027	0.0025
	Dingman's	N/A	N/A
	SRM	N/A	N/A
	Table 7.3 of Julien (1990)	N/A	N/A
Critical Incipient Shear Stress for the D_{100} (lbs/ft^2)	Flume	0.0081	N/A
	Dingman's	N/A	N/A
	SRM	N/A	N/A
	Table 7.3 of Julien (1990)	N/A	N/A
Critical Grain Entrainment Velocity (ft/sec)	Flume	0.9	0.9
Critical Grain Entrainment Shear Stress (lbs/ft^2)	Flume	0.0043	0.0047

CONCLUSIONS

The research objectives stated in the introduction have been achieved through a

comprehensive literature review, experimentation, and testing. Sediment samples were

obtained and analyzed to produce a grain size distribution of the sediment from the Green

River Canal. The sediment samples were classified using the ASCE sediment size

classification system. The non-cohesive soil was classified as very fine sand and the

cohesive soil was classified as very fine silty clay. Results obtained from the grain size analyses were used in all calculations and presents the reader with knowledge of the type of soil introduced into the canal.

The critical incipient velocity and an associated critical shear stress have been presented in this paper. Flume experimentation resulted in an average critical incipient velocity of 0.88 feet per second and a corresponding critical incipient shear stress of 0.0027 pounds per square foot for the D_{50} of the very fine sand. Flume experimentation also resulted in the average critical incipient velocity of 1.6 feet per second and a critical incipient shear stress of 0.0081 pounds per square foot for the D_{100} of the very fine sand. An average critical incipient velocity for the very fine silty clay was measured to be approximately 0.74 feet per second and a corresponding critical incipient shear stress of 0.0025 pounds per square foot. These values correspond to the critical incipient velocity, or the minimum velocity that initiates sediment grain motion. It is also important to note that the author is more confident in the results for the very fine sand than the results for the very fine silty clay. The very fine silty clay was very difficult to work with due to the cohesive nature of this sediment and the results obtained are not in agreement with previously-published research.

The third research objective of this paper was to determine the minimum velocity that retains all the fine sediment in suspension and its associated shear stress. This paper has referred to this minimum velocity as the critical grain entrainment velocity. The average critical grain entrainment velocity is the velocity that has a magnitude less than the average critical incipient velocity for the D_{100} of the fine sediment in consideration and a magnitude greater than the average critical incipient velocity of the D_{50} of the fine

sediment in consideration. Some deposition of the coarser fraction of the very fine sand will occur if the minimum velocity operated in the Green River Canal is the average critical incipient velocity that corresponds to a sediment grain diameter of 0.12 millimeters (approximately the D_{50} of the very fine sand). The author, through flume experimentation, has determined the average critical grain entrainment velocity to be 1.14 feet per second with an associated shear stress of 0.0043 pounds per square foot corresponding to the D_{95} of the very fine sand. Flume experimentation also resulted in an average critical incipient velocity of 0.88 feet per second and an associated critical incipient shear stress of 0.0027 pounds per square foot for the D_{50} of the very fine sand. The average critical velocity referred to in this section refers to the velocity located at a distance equal to the product of 0.37 and the total flow depth, measured from the bed of the channel as suggested by Dingman (1984).

The fourth research objective was to compare the results obtained for the critical incipient velocity as determined by flume experimentation to the critical incipient velocity determined by formulae developed by previous research. The measured critical incipient velocity as determined from flume experimentation compares the best with the theoretical critical incipient velocity relationship presented by Dingman (1984). Dingman's (1984) method calculated a critical incipient velocity for a grain diameter of 0.08 millimeters that was only 0.09% lower than the actual critical incipient velocity measured in the flume. The SRM method predicted a 27% lower critical velocity for the median grain diameter than the actual critical velocity measured in the flume. Also noteworthy is that the SRM method has a significant limitation. The limitation is due to the quantity in the denominator of the SRM formula. As the particle shear Reynolds

number approaches zero, the lift coefficient reduces at a greater rate than the drag coefficient increases. The quantity in the denominator approaches a negative value which yields an invalid result to the SRM method at very low particle shear Reynolds numbers.

The methods presented by Dingman (1984) and the SRM equation are limited to non-cohesive soils. Critical incipient velocities for cohesive soils have been discussed and compared against maximum permissible velocities presented in Table 7.3 of Julien (1990).

In conclusion, the author recommends that the Green River Canal be operated at a minimum average velocity of 1.14 feet per second under the assumption that the sediment entering into the canal is the same as the sediment samples taken from the canal. It has been shown by flume experimentation and by Dingman's (1984) method that this average velocity will transport the sampled sediment especially if the sediment is already in suspension. If the clay and silt particles are allowed to deposit, the average velocity will need to be significantly higher to re-entrain these finer particles due to their cohesive nature. The author refers the reader to Table 7.3 of Julien (1995) for maximum permissible velocities in cohesive channels.

It is important to note that Dingman's (1984) method calculates a critical incipient velocity which is the velocity at which grain particles just begin to move. Flume experimentation determined that the very fine sand introduced into the canal will still deposit if the minimum velocity is the critical incipient velocity based on the median grain diameter of the very fine sand. Using Dingman's (1984) methodology, an average critical grain entrainment velocity of 1.14 feet per second was calculated for a critical

grain diameter of 0.25 millimeters which corresponds to approximately the D_{95} of the very fine sand. Therefore, the author recommends that a grain diameter of the fine sediment be approximately equal to the D_{95} of the fine sediment being deposited when using Dingman's (1984) method to estimate the average critical grain entrainment velocity. It is noteworthy to mention that the $D_{50} - D_{90}$ of the sediment may be an acceptable grain diameter depending on the gradation coefficient of the sediment under consideration. The author used the D_{95} of the sample because it was back calculated using Dingman's (1984) method. Dingman's (1984) method compares well to the data obtained by the author for this project and seems to be an accurate method to estimate the average critical grain entrainment velocity. However, the author recommends that Dingman's (1984) method be used in conjunction with other methods. Once the maximum and minimum values are calculated from all of the methods and taken into consideration, engineering judgment should be used to select a value that is most representative of the average critical grain entrainment velocity.

The author recommends that further research be performed on the critical grain entrainment velocity. The research should address the questions: What quantity of sediment is required to be held in suspension to be classified as the critical grain entrainment velocity? Are critical incipient formulae adequate to estimate the critical grain entrainment velocity, and if so, what critical grain diameter should be used?

Further analysis of the Green River Canal still needs to be completed to determine why the very fine sand is deposited downstream of the second inverted siphon and why the very fine silty clay is deposited further downstream. The survey of slopes and cross-sections of the canal are currently in process and need to be completed. The locations

and photographs of turnouts are currently being worked on and should be completed by
May of 2005. An arc view map showing the project location, the locations of turnouts,
and other pertinent information is currently in the works and also needs to be completed.

Along with the analysis of the canal, a fundamental knowledge of the river from
which the canal diverts water is necessary. Refer to Appendix E for information
regarding the geomorphic changes occurring in the Green River in the Gunnison Valley
near Green River, Utah.

Earlier in the report the author assumed that the fine sediment sampled in the
Green River Canal is the same as the suspended sediment entering into the canal. Table
1.3 depicts the comparison between two sediment samples, one sediment sample is a
deposit analyzed by Allred and Schmidt (1999) from the Green River and the other is the
fine sediment sample from the Green River Canal.

Table 1.3 Comparison of the sediment of the Green River to the sediment in the canal.

Soil Class	*(1999) Sediment Sample	**(1999) Sediment Sample	Canal Very Fine Sand Sediment Sample	Canal Very Fine Silty Clay Sediment Sample	Average Canal Sediment Sample
Gravel	0.00%	0.00%	0.70%	0.00%	0.35%
Sand	70.50%	42.40%	73.90%	3.34%	38.62%
Silt	22.00%	45.47%	23.30%	53.06%	38.18%
Clay	7.50%	12.33%	2.10%	43.60%	22.85%

* Average of the top six inches of the deposit sediment measured by Allred and
Schmidt (1999)
** Average of the total deposit as measured by Allred and Schmidt (1999)

The soil classes averaged through the top 6 inches of the deposit are comparable
to the very fine sand sample from the canal. The soil classes averaged through the rest of
the deposit also compares well to the average of the very fine sandy soil and the very fine
silty clay sample from the canal. The fine sediment depositing in the canal appears to be

the same sediment that is depositing along the banks of the Green River but is failing to deposit before entering into the canal. There may be some geomorphic changes occurring upstream or at the headwaters of the canal that is increasing the suspended sediment concentration into the canal. It is proposed by the author that these geomorphic changes occurring in the Green River as well as the local hydraulics upstream and near the headwaters of the canal be researched further. This further research will aid in determining what impact these geomorphic changes may have on the increased concentration of suspended sediment into the canal and also to determine the reason why the sediment is failing to deposit before entering the canal.

REFERENCES

Allred, T. M., and Schmidt, J.C. (1999). "Channel narrowing by vertical accretion along the Green River near Green River, Utah." Geological Society of America Bulletin, 111(12), 1757-1772.

Baker, C. J. (1980). "Theoretical approach to prediction of local scour around bridge piers." Journal of Hydraulc Research, 18(1), 1-12.

Bowles, J. E. (1992). *Engineering properties of soils and their measurement*, McGraw-Hill, New York.

Chang, H. H. (1988). *Fluvial processes in river engineering*, Wiley, New York.

Cheema, M. N., Marino, M. A., and DeVries, J. J. (1997). "Stable width of an alluvial channel." Journal of Irrigation and Drainage Engineering, 123(1).

Church, M. and Ferguson, R. I. (2004). "A simple equation for grain settling velocity." Journal of Sedimentary Research, 74(6), 933-937.

Cliff, R. and Gauvin, W. H. (1970). "The motion of particles in turbulent gas streams." Proceedings Chemeca, '70, Vol. 1, 14-28.

Crow, C. T., Elger, D. F., and Roberson, J. A. (2001). *Engineering fluid mechanics*, Wiley, New York.

Dingman, S. L. (1984). *Fluvial hydrology*, Freeman, New York.

Grayson, R. B., Marsh, N. A., and Western, A. W. (2004). "Comparison of methods for predicting incipient motion for sand beds." Journal of Hydraulic Engineering, America Society of Civil Engineers, 130(7), 616-621

James, C. S. (1990). "Prediction of entrainment conditions for nonuniform, noncohesive sediments." Journal of Hydraulic Research, 28(1), 25-41.

Julien, P. Y. (1995). *Erosion and sedimentation*, University of Cambridge, New York.

Rahmeyer, W. J. (2005). *Sedimentation engineering*, CEE 6470, Sedimentation Engineering Class Notes.

Shields, A. (1936). Anwendung der Ähnlichkeitsmechanki und der Turbulenzforschung auf die Geschiebebewegung [Application of similitude and turbulence research to bed-load movement]. *Mitteilungen der Prüssischen Versuchanstalt für Wasserbau und Schiffbau* (Berlin), Heft 26.

Spells, K. E. (1955). "Correlations for use in transport of aqueous suspensions for fine solids through pipes." Trans., Inst. Chem. Engrs., Vol 33.

Twenhofel, W. H. (1939). *Priciples of sedimentation*, McGraw Hill, New York.

United States Army Corps of Engineers. (1995). "Engineering and design – sedimentation investigations of rivers and reservoirs." <http://www.usace.army.mil/inet/usace-docs/eng-manuals/em1110-2-4000/c-9.pdf>. Accessed, March 8, 2005.

Wiberg, P., and Smith, J. D. (1985). "A theoretical model for saltating grains in water." Journal of Geophysical Research, 90(C4), 7341-7354.

Wiberg, P., and Smith, J. D. (1987). "Calculations of the critical shear stress for motion of uniform and heterogeneous sediments." Water Resources Research, 23(8), 1297-1312.

Yalin, M. S. and Karahan, E. (1979). "Inception of sediment transport." Journal of the Hydraulics Division, America Society of Civil Engineers, 105(HY-11), 1433-1443. as cited in Dingman, S. L. (1984). *Fluvial hydrology*, Freeman, New York.

Yang, C. T. (1973). "Incipient motion and sediment transport." Journal of the Hydraulics Division, America Society of Civil Engineers, 99(10), 1679-1704.

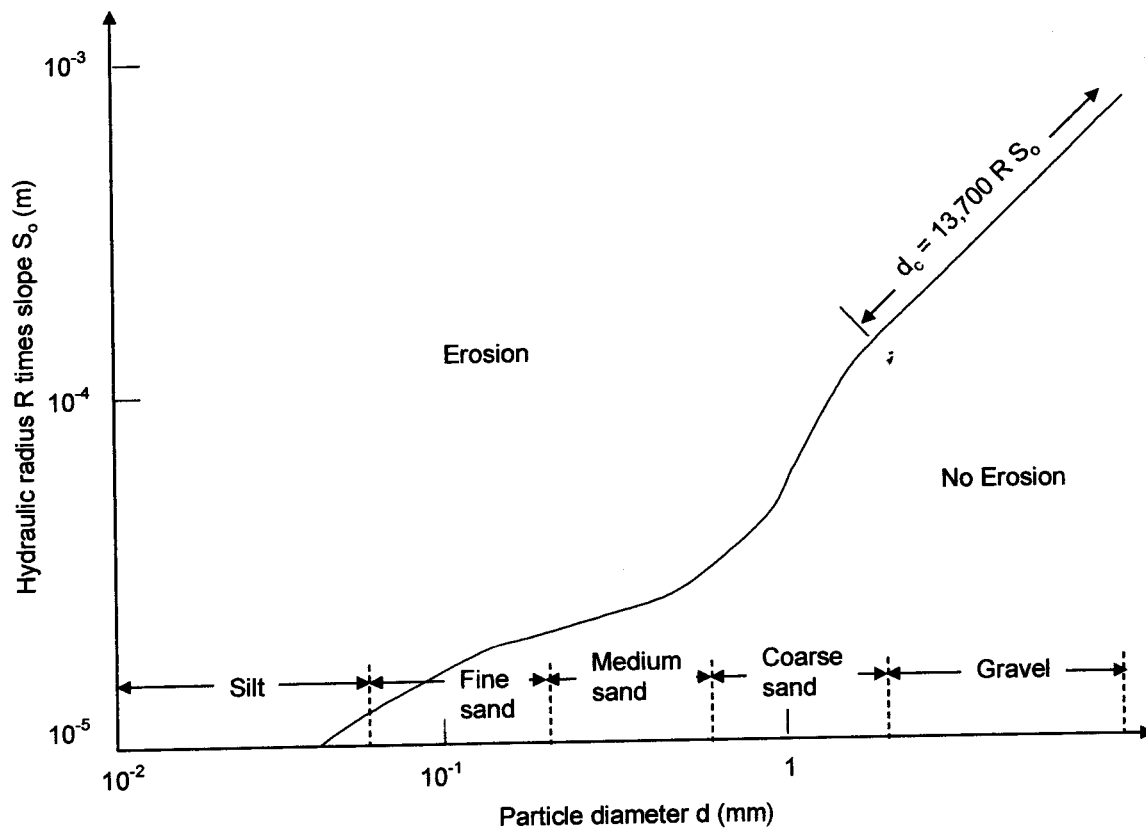APPENDICES

Appendix A. Figures 8.7 and 8.8 of Dingman (1984)

Figure 8.7 Critical product of the hydraulic radius and slope versus the critical grain diameter. Source: Dingman (1984)
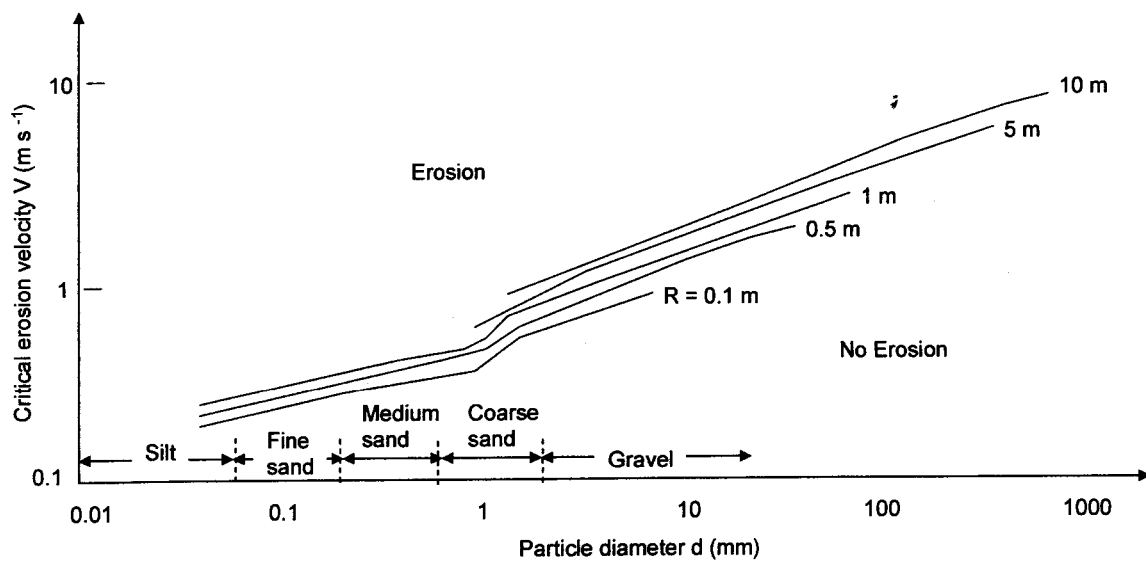
Figure 8.8 The critical erosive velocity versus critical grain diameter for computed hydraulic radii.

Appendix B. Table 7.3 of Julien (1995)

Table 7.3 Average maximum permissible velocities for cohesive channels

Soil type	Maximum velocity (m/s)
Fine sandy loamy clay	0.45-0.91
Alluvial mud	0.61-0.84
Alluvial loamy clay	0.76-0.84
Hard loamy clay	0.91-1.14
Hard clay	0.76-1.52
Rigid Clay	1.22-1.52
Clayey Shale	0.76-2.13
Hard rock	3.00-4.5

ρ_m (kg/m^3)	Loamy Sand[a] (m/s)	Non-Plastic Clay (m/s)	Clay[a] (m/s)	Heavey Clayey Soil (m/s)	Loamy Clay[a] (m/s)
1200	$0.1 \log 8.8\, h / k_s$	-	$0.12 \log 8.8\, h / ks$	-	$0.12 \log 8.8\, h / ks$
1200-1650	$0.15 \log 8.8\, h / k_s$	-	$0.3 \log 8.8\, h / ks$	-	$0.25 \log 8.8\, h / ks$
1544	-	0.32	0.35	0.4	0.45
1650-2040	-	-	$0.45 \log 8.8\, h / ks$	-	$0.45 \log 8.8\, h / ks$
1742	-	0.7	0.8	0.85	0.9
2040	-	1.05	1.2	1.25	1.3
2040-2140	-	-	$0.65 \log 8.8\, h / ks$	-	$0.6 \log 8.8\, h / ks$
2270	-	1.35	1.65	1.7	1.8

[a] h is the depth of flow and ks is the boundary roughness height.

Source: Modified after Etcheverry (1916), Fortier and Scobey (1926), and Mirtskhoulava (1988).

Source: Julien (1995)

Appendix C. Three Dimensional View of the One Dimensional Vertical Velocity Profile and Flume Photographs
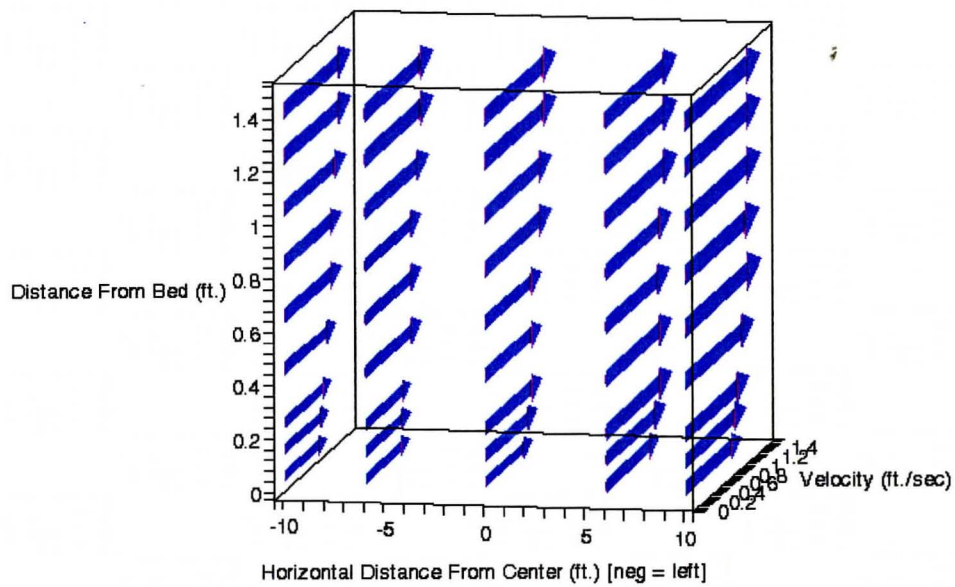
Active Bed and the Forming of Sand Ripples on the Bed

Figure A-C0 Three dimensional view of the one dimensional vertical velocity profile measured for the very fine sand.
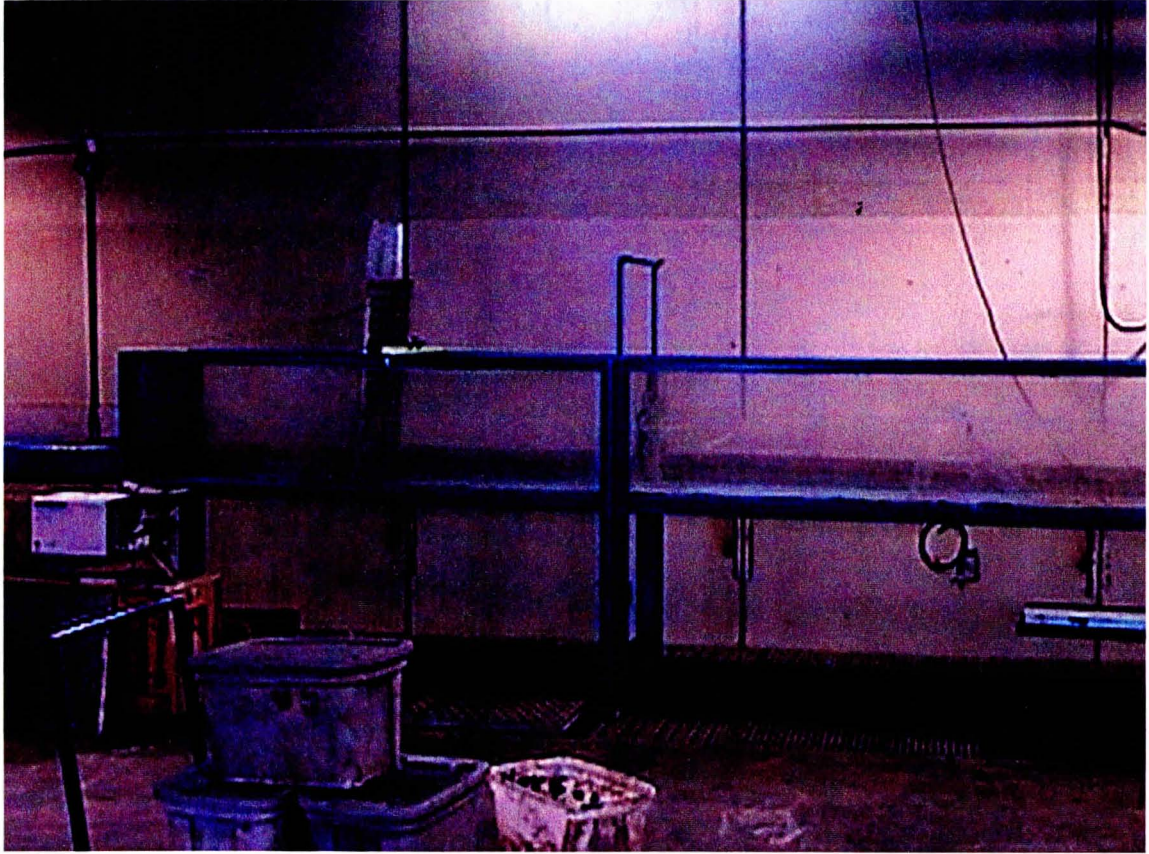
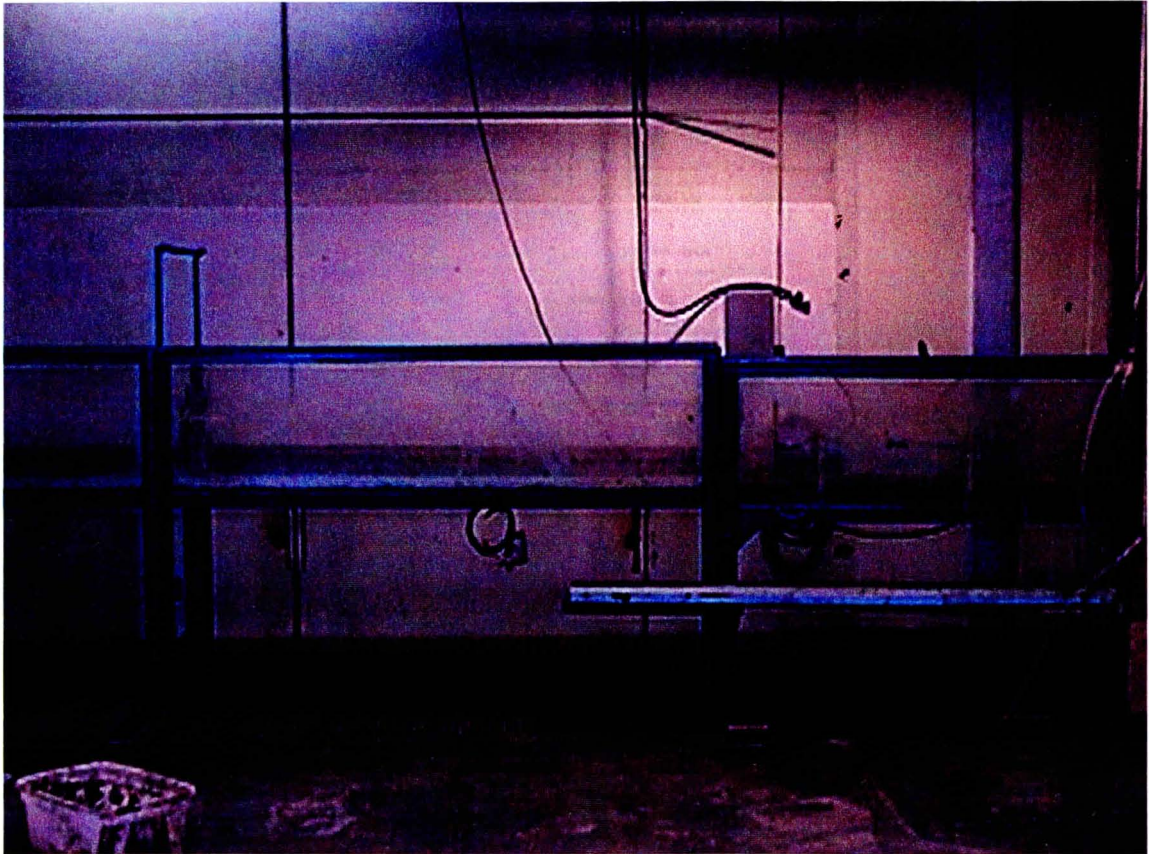**Figure A-C1** Beginning of the flume.
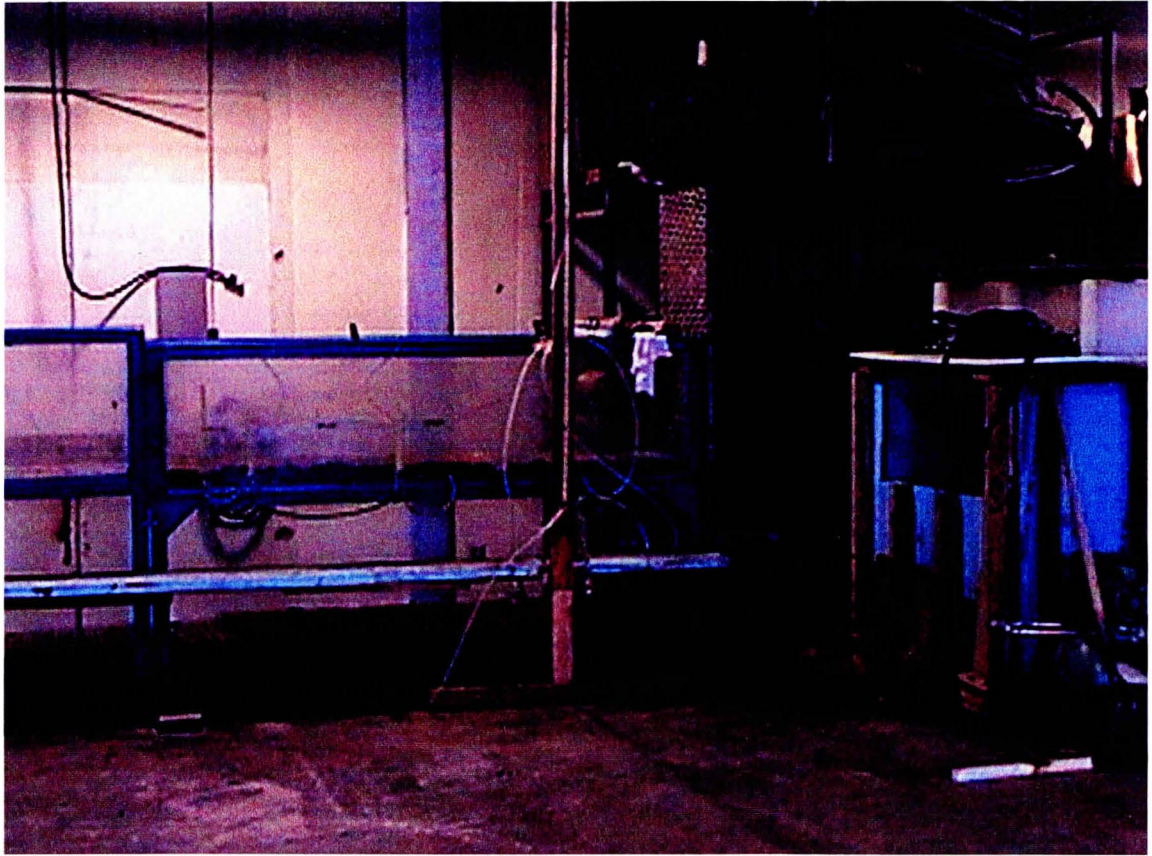
Figure A-C2 Mid-section of the flume.

Figure A-C3 The downstream end of the flume.

Figure A-C4 Rectangular chutes which caused undesirable flow conditions.

Figure A-C5 Apparatus installed just downstream of the rectangular chutes to create favorable flow conditions.

Figure A-C6 Apparatus located at the end of the flume to produce favorable flow conditions.
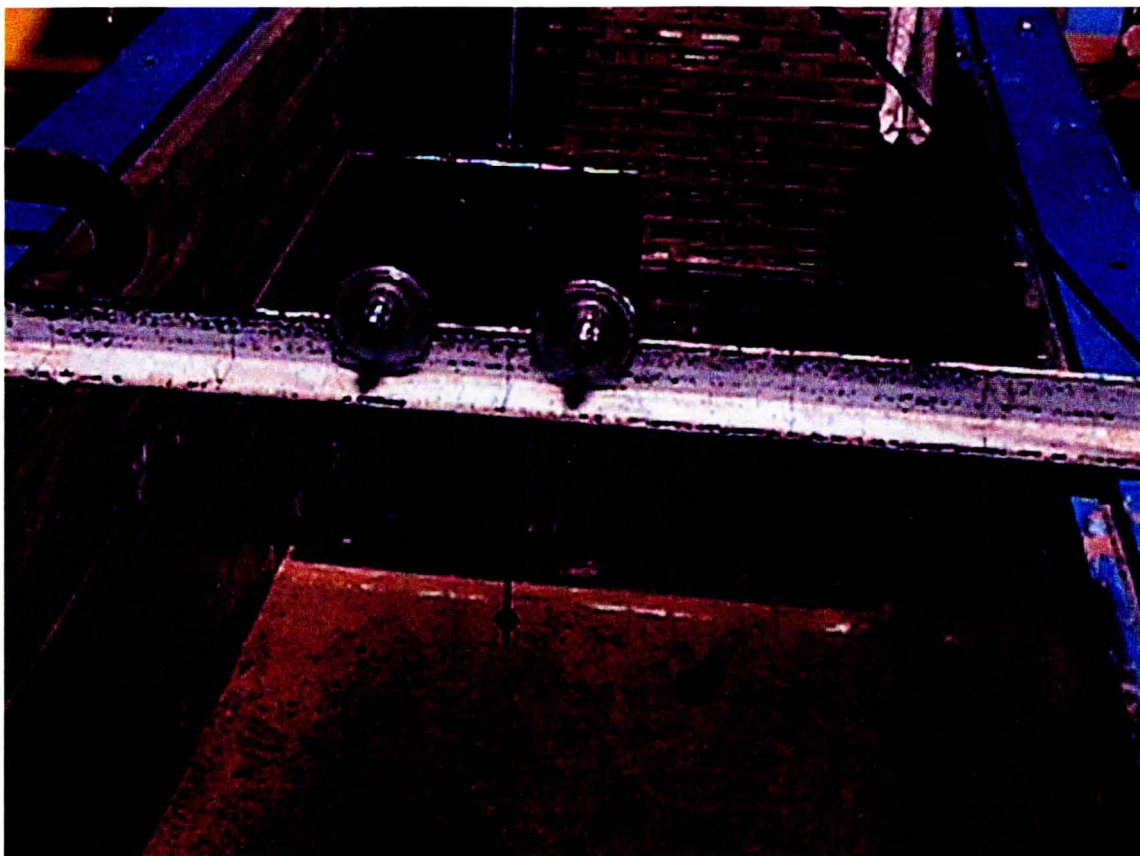
Figure A-C7 Marsh McBirney flow meter and bracket located near the longitudinal center of the test section.

Figure A-C8 Marsh McBirney flow meter output display.

Figure A-C9 Upstream half of the flume.

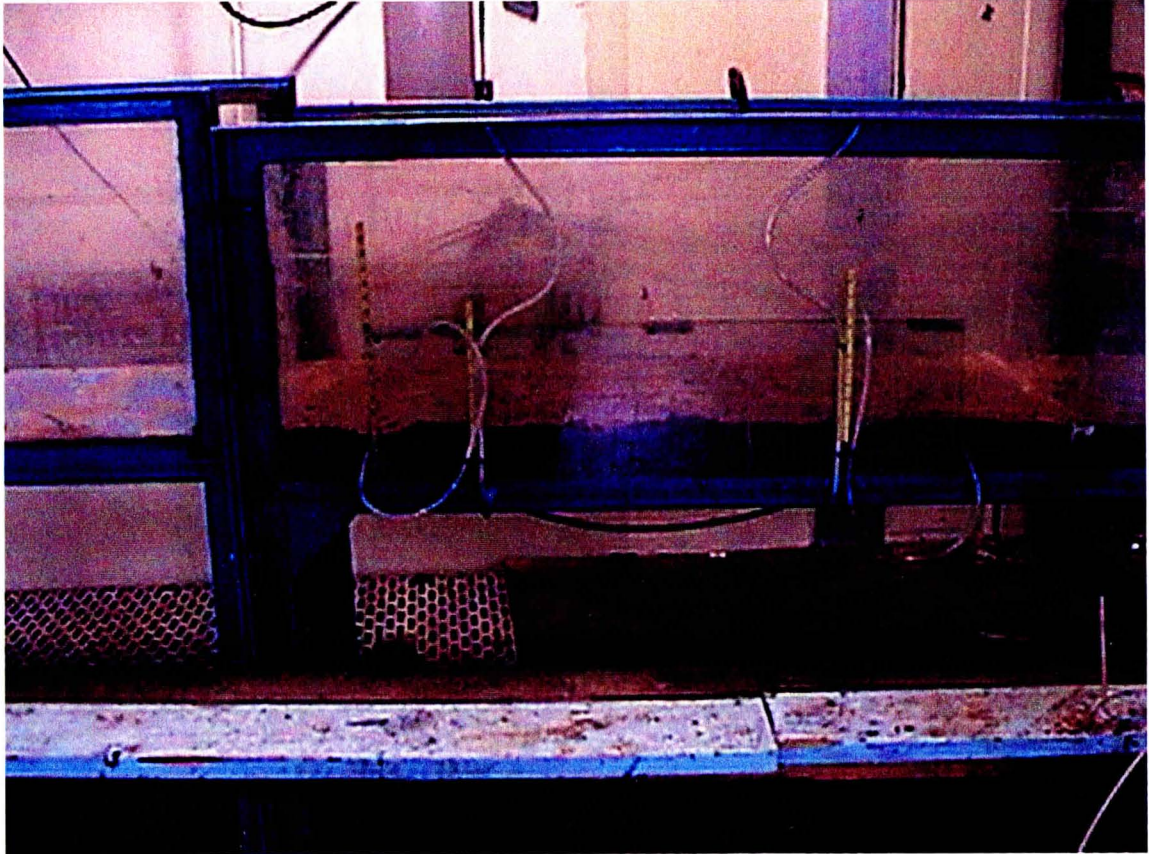**Figure A-C10** Downstream half of the flume.

Figure A-C11 Sediment test section.

Appendix D. Canal Photographs and Selected Cross-Sections

Figure A-D0 Green River Canal bridge crossing (downstream of location where the very fine sand has been deposited).

Figure A-D1 Green River Canal bridge crossing (no fine sediment deposition).

Figure A-D2 Green River Canal culvert crossing (no fine sediment deposition).

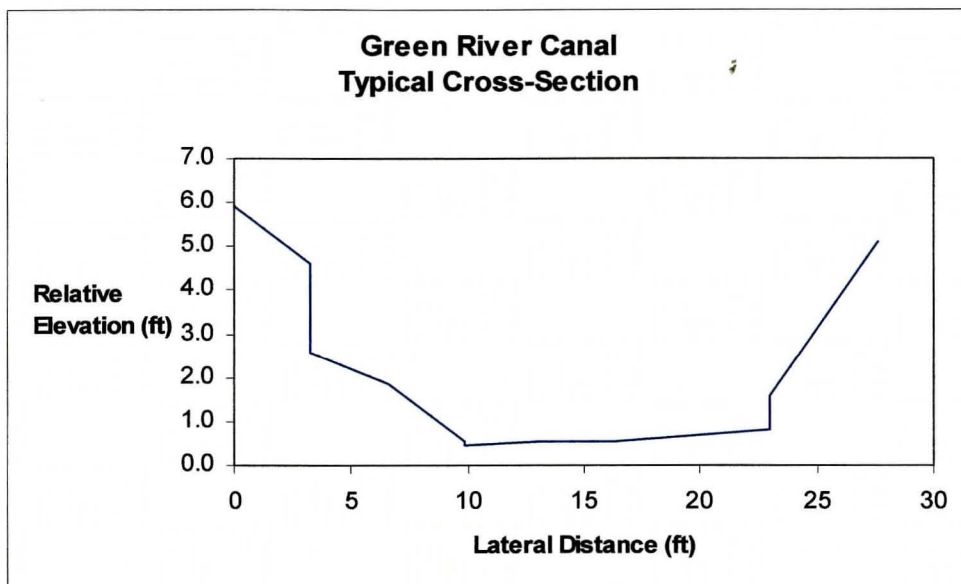**Figure A-D3** Green River Canal turnout (local fine sediment deposition).

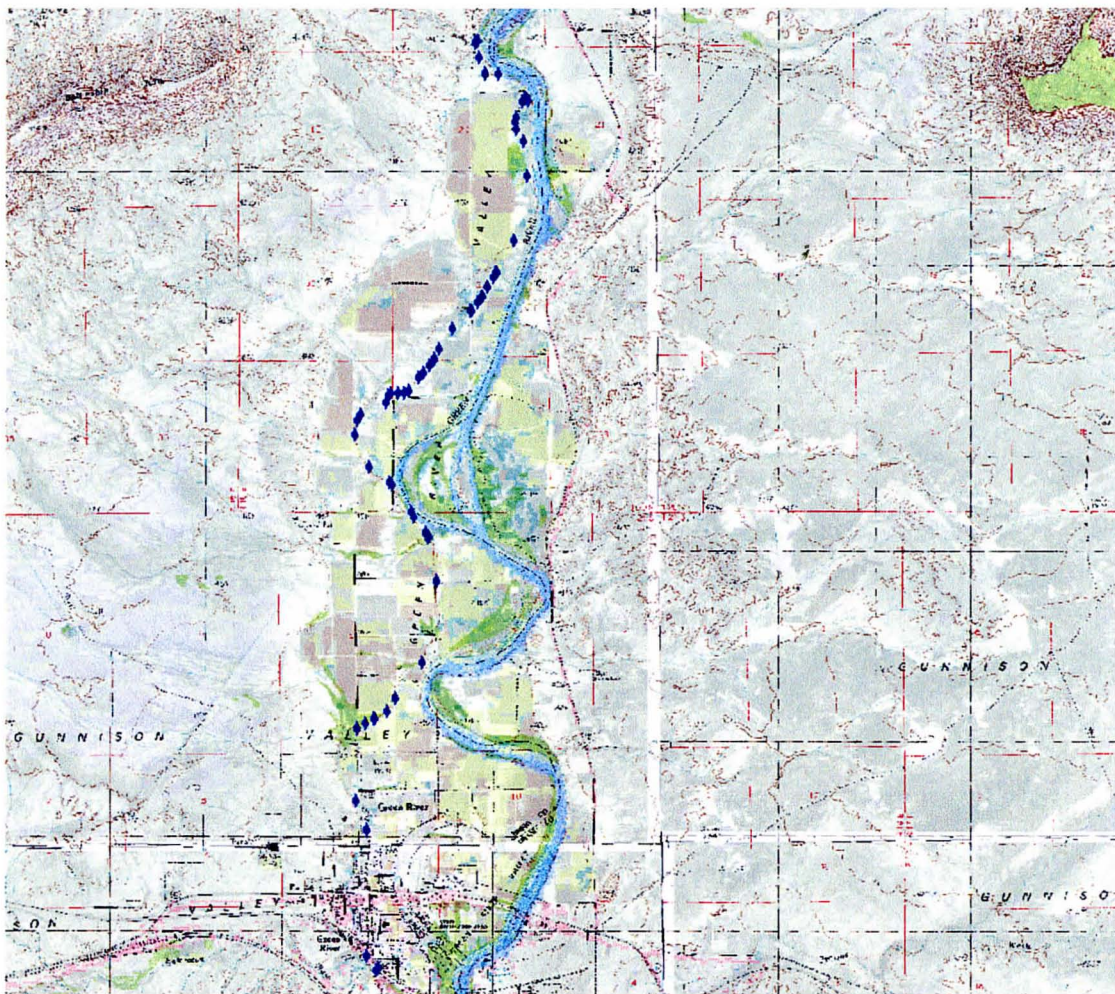**Figure A-D4** Typical cross-section of the Green River Canal.

Figure A-D5 Location Map of Green River and Green River Canal (blue diamonds).

Appendix E. Geomorphic Changes Occurring in the Green River

Tyler M. Allred, a graduate student at Utah State University, and John C. Schmidt, a professor from Utah State University, conducted a study of channel narrowing of the Green River in the Gunnison Valley. From their study, the authors formulated two main changes of the Green River that may impact the concentration of sediment introduced into the canal. The first change is in the channel cross-section. The channel has narrowed by a magnitude of 15 meters from 1930 to 1993 at the present day cableway. Secondly, the effective discharge has decreased by approximately 54%. The modal effective discharge between the years of 1894 and 1929 was 1,077 cubic meters per second (m^3/s). The modal effective discharge has decreased to a value of 494 m^3/s after the completion of Flaming Gorge Dam.

There appears to be no substantial change in the size of the suspended sediment or in the concentration of suspended sediment. A deposit studied by Allred and Schmidt (1999) revealed that nearly all of the sand was finer than 175 microns (μm). Suspended sediment measurements resulted concentrations greater than 1,200 milligrams per liter (mg/l) for a range of grain sizes from 125 microns to 175 microns. At lower velocities, sand grains carried in suspension measured 88 microns and grains sizes finer than 125 microns had a concentration greater than 190 mg/l (Allred and Schmidt, 1999). The grain size distribution of the very fine sand taken from the canal shows that almost all of the grain particles are less than 175 microns as well.