Hydraulics of gorge type reservoirs.

Allan P. Sweetman

University of Windsor

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HYDRAULICS OF GORGE TYPE RESERVOIRS

A Thesis
Submitted to the Faculty of Graduate Studies through the
Department of Civil Engineering in Partial Fulfilment
of the Requirements for the Degree of
Master of Applied Science at the
University of Windsor

by

Allan P. Sweetman

Windsor Ontario
1971
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ABSTRACT

Reservoir deposition experiments have been carried out under various combinations of flow, sand discharge rates, grain sizes and reservoir depths.

A study has been made to compare the results obtained from the experiments with the well known bed load formulae of DuBoys, Shields, Einstein and Kalinske.

Finally a comparison of the data obtained was made with the stream geometry equations of Lacey, Leopold and Maddock.
ACKNOWLEDGEMENTS

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\( p \) coverage factor in Kalinske's bed load argument

\( p_S \) probability of grain dislodgement

\( Q \) volumetric rate of discharge

\( q \) discharge per unit width

\( q_s \) sediment discharge per unit width

\( R \) hydraulic radius

\( S \) longitudinal bed slope

\( S_s \) solid:fluid density ratio

\( t \) time

\( V \) velocity of fluid

\( v_s \) particle velocity

\( W \) channel bed width

\( w \) particle fall velocity

\( x \) power of hydraulic radius in Liu-Hwang method

\( y \) power of slope in Liu-Hwang method

\( f \) functional sign

\( \lambda \) coefficient in Einstein's bed load argument

\( \nu \) kinematic viscosity

\( \rho \) fluid density

\( T_o \) shear stress at the bed

\( T_c \) critical shear stress on the threshold of movement

\( \Theta \) specific weight of water

\( \Phi \) Einstein's bed load function

\( \Psi \) reciprocal of Shields' entrainment function

\( \sigma \) standard deviation

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.
1.1 Description and Object of the Investigation

A great deal of concern lately has arisen about the loss of storage and flood control capacity of many of the nation's reservoirs due to the build up of sediment deposits.

Many questions must be answered; some of these are:

1) How long will it be until the capacity of a reservoir will be so reduced that it will not serve its original design functions?

2) Where will the deposits settle out to enable the proper location of the outlets (spillways, intakes, bypasses, etc.)?

3) What is the mechanics governing the mode of advance of the sediment beds?

4) How could reservoir operations be carried out to minimize siltation?

Since the flow into existing reservoirs has varying discharges and sediment loads throughout the year it has been very difficult to estimate the effect of each variable on the final position of the deposits by using field data.

The present study was not undertaken to answer the above questions but rather to take one particular shape of reservoir and examine the factors controlling the hydraulic characteristics of the flow in this basin to enable us to design future experiments that will answer these problems. In attempting this, various flow rates, sand discharge rates, reservoir depths and grain sizes were used.
and the experimental results were compared with the classical mobile bed hydraulic equations.
2.1 Bed load formulas

Many formulae have been proposed by various researchers to explain the transport of bed material in channels under uniform flow conditions. In the present study, even though the overall flow conditions in the model are non-uniform due to the sand being deposited in the reservoir, the water and sand flow does become constant in the upper reaches of the reservoir. A comparison of the following formulae, with the conditions in this region, has been made.

a) Du Boys' formula

The classic analysis of bed load was first published by Du Boys in 1879. The analysis assumes that the bed moves in a sort of laminar flow, with layers sliding over each other, the velocity decreasing linearly with depth.

This leads to the equation:

$$q_s = C_s T_0 \left[ T_0 - T_c \right]$$

(2.1)

where $q_s$ is the sediment discharge per unit width (by volume)

$C_s$ is the Du Boys' coefficient

$T_0$ is the shear stress at the bed

$T_c$ is the maximum allowable tractive force in the bed material

Straub (8) found that the coefficient $C_s$ was not constant but followed the relation:

$$C_s = \frac{0.173}{3/4} \frac{d}{d}$$

(2.2)

where $d$ is the grain size in millimeters
Du Boys' formula was based on empirical values of $T_c$ that differ somewhat from Shields and are plotted along with Shields' values in figure 1.

b) Shields' formula

Shields (8,13) based his formula on the Nikuradse velocity distribution equations and a qualitative analysis of the drag forces exerted on the individual sand grains at the bed. The equation is dimensionally homogeneous, so that any consistent system of units can be used and has been successful in correlating numerous laboratory sediment studies, although it has an appreciable scatter. It may be written as follows:

$$\frac{q_S S_s}{q S} = 10 \left[ \frac{T_o - T_c}{T (S_s - 1) d} \right]$$

where $S_s$ is the solid:fluid density ratio

$q$ is the discharge per unit width

$S$ is the bed slope

$T$ is the specific weight of water

d is the grain size

c) Einstein's formula

One of the best known formulas is that of Einstein (2,6,8,13) which is based on physical reasoning as well as dimensional and statistical considerations.

His basic observation was that a grain on being dislodged from the bed, travelled a distance $L$ before coming to rest again, and that $L = \lambda d$

where $L$ is the particle travel distance

$\lambda$ is a coefficient
If $p_s$ is the probability that any grain will be dislodged within a given second, and $A_1 d^2$ is the bed area occupied by each grain, then the number of grains dislodged every second will be equal to:

$$\frac{L p_s}{A_1 d^2}$$

Hence if $A_2 d^3$ is the volume of each grain, then the total volume of grains crossing a section every second is equal to:

$$q_s = \frac{L p_s A_2 d^3}{A_1 d^2} = \frac{A_2}{A_1} \lambda d^2 p_s$$  \hspace{1cm} (2.4)

where $A_1$ and $A_2$ are particle shape factors.

To evaluate $p_s$ Einstein postulated that it would be a function of the ratio between the lift force which the flow can exert on the grain and the submerged weight of the grain itself or:

$$\frac{T_0}{A_2 T (S_s - 1) d}$$

which is proportional to the Shields entrainment function $F_s$.

Since $F_s$ is dimensionless and $p_s$ has dimension of $1/t$ it follows that:

$$p_s = \frac{1}{\tau} \mathcal{F}(F_s)$$

Einstein postulated that a suitable time interval would be the time for a grain to fall its own diameter at its fall velocity. Thus:

$$p_s \frac{d}{w} = \mathcal{F}(F_s)$$  \hspace{1cm} (2.5)

Substituting the above in equation 2.4, incorporating the constants $A_1, A_2$, and $\lambda$ in the functional sign, he obtained:

$$\frac{q_s}{\omega d} = \mathcal{F}(F_s)$$  \hspace{1cm} (2.6)

or

$$\phi = \mathcal{F}\left(\frac{1}{\Psi}\right)$$  \hspace{1cm} (2.7)
where $w$, the particle fall velocity, is given by

$$ w = G \sqrt{\frac{g d}{(S_s - 1)}} $$

and

$$ G = \sqrt{\frac{2}{3}} \frac{36 \nu^2}{g d^3 (S_s - 1)} = \sqrt{\frac{36 \nu^2}{g d^3 (S_s - 1)}} $$

(2.8) (2.9)

$\Phi$ is the Einstein bed load function

$\Psi$ is the reciprocal of Shields' entrainment function

$g$ is the acceleration due to gravity

$\nu$ is the kinematic viscosity

The empirical $\Phi - 1/\Psi$ relation is plotted in figure 2 in a form due to Brown (2), who also deduced the equation given for the upper straight line portion of the curve which is:

$$ \Phi = 40 \left( \frac{1}{\Psi} \right)^3 $$

d) Kalinske's formula

Kalinske (2,8) suggested a bed load formula based on the analysis of turbulent fluctuations in velocity at the bed. He stated that the volume rate of sediment movement per unit width must be equal to the product of the particle velocity $v_s$, the particle volume $\frac{\pi d^3}{6}$ and the average number of particles moving per unit bed area.

Thus

$$ q_s = \frac{\pi d^3}{6} v_s p \frac{d}{\pi d^2/4} = \frac{2}{3} v_s p d $$

(2.10)

where $p$ is the coverage factor

The quantities $v_s$ and $p$ remain to be evaluated. The particle velocity $v_s$ is proportional to the shear velocity $\sqrt{\frac{T_o}{\rho}}$ and the coverage factor $p$ is a function of $T_o/T_c$, which is directly proportional to the entrainment function $1/\Psi$ with the result that
\[
\frac{q_s}{\sqrt{To/\rho}} \text{d} = f \left( \frac{1}{\psi} \right) \tag{2.11}
\]

where \( \rho \) is the fluid density

This function is plotted in figure 3 in a form produced by Brown (2) who also showed that the upper portion followed the relation:

\[
\frac{q_s}{\sqrt{To/\rho}} \text{d} = 10 \left( \frac{1}{\psi} \right)^2 = 10 \left( \frac{To}{T (Ss-1) d} \right)^2 \tag{2.12}
\]

### 2.2 Choice of Bed Load Formula

It is often very difficult to make a choice among the bed load formulae developed by DuBoys, Shields, Einstein and Kalinske, for the data used in developing these equations had about one log cycle of scatter. It is generally accepted that the DuBoys' equation is the least satisfactory while the remaining equations give equally acceptable results. The equations as developed are in a chronological order and each succeeding researcher utilized most of the previous experimental data in trying to develop a more general formula. Einstein's and Kalinske's experiments were generally in the dune-type bedform range of flow. These equations are only meant to be used for channels with granular, non-cohesive bed materials.

### 2.3 Liu-Hwang Equations

Liu and Hwang did a study to find a suitable formula to determine more accurately the mean velocity of flow in alluvial channels. They assumed that in general an exponential type of discharge formula would apply and could be written as:

\[
V = C_a R^x S^y \tag{2.13}
\]
where \( C_a \) is an empirical coefficient

\[ x \text{ and } y \text{ are pure numbers} \]

Four figures were developed to evaluate the above constants. Using figure 4, the bed form is evaluated, and then from figure 5 the power of the hydraulic radius is estimated. Figure 6 shows the power of the slope, while figure 7 estimates the discharge coefficient.

2.4 Lacey's Equations

Lacey (10) using the principles of classical physics approached the problems of mobile-boundary hydraulics by an analysis of measurable data derived from canal systems in India.

Lacey first examined the proposition

\[ \frac{P}{R} = \text{function } V \]

obtaining the expression

\[ P = 2.67 Q^{1/2} \tag{2.14} \]

where \( P \) is the wetted perimeter of the flow cross-section

\( R \) is the hydraulic mean radius

\( V \) is the mean velocity

\( Q \) is the volumetric rate of discharge

Lacey also put forward "a very rough qualitative formula" for the silt factor \( f \) as follows:

\[ f = 8 d^{1/2} \tag{2.15} \]

where \( d \) is the grain size in inches

He utilized this equation in deriving the following formulae for slope and area:

\[ S = 0.000547 \varepsilon^{5/3} Q^{-1/6} \tag{2.16} \]
From which it is possible to derive equations for the flow depth and velocity.

\[ A = 1.25 f^{-1/3} Q^{5/6} \quad (2.17) \]

\[ D = 0.473 f^{-1/3} Q^{1/3} \quad (2.18) \]

\[ V = 0.895 f^{1/3} Q^{1/6} \quad (2.19) \]

where \( A \) is the flow area

\( D \) is the mean depth of flow

2.5 Stream Geometry Equations

Leopold and Maddock (12) using the data on streamflow available in the Water Supply Papers of the U.S. Geological Survey, developed a series of equations relating width, depth and velocity to the discharge.

First they examined the characteristics of the channel shape at certain river cross-sections. At a given cross-section the width, depth and velocity all change with the amount of water flowing in the cross-section.

Next they examined these same characteristics in succeeding sections moving downstream along the length of a particular river system.

They found that if the width, depth and velocity are plotted against discharge on logarithmic paper, their relations to discharge are expressed by nearly straight lines, irrespective as to whether the data was for one cross-section or a downstream series.

Thus the relation of discharge to other hydraulic factors in natural rivers can be described by

\[ W = a Q^b \quad (2.20) \]

\[ D = c Q^e \quad (2.21) \]
\[ V = k Q^j \]  \hspace{1cm} (2.22)

where \( W \) is the channel bed width

\( a, c, k, b, e, \) and \( j \) are constants

Since \( Q = \text{area} \times \text{velocity} \)

\[ = W D V = a Q^b \times c Q^e \times k Q^j \]

\[ = a c k Q^{b+e+j} \]  \hspace{1cm} (2.23)

It therefore follows that

\[ b + e + j = 1.0 \]  \hspace{1cm} (2.24)

\[ a c k = 1.0 \]  \hspace{1cm} (2.25)

The average values of the exponents \( b, e, \) and \( j \) were computed for 20 river cross-sections by Leopold and Maddock, representing a large variety of rivers in the Great Plains and Southwest and they were found to be:

\[ b = 0.26 \]

\[ e = 0.40 \]

\[ j = 0.34 \]

In the second study the average value for these exponents were calculated using data from 8 river systems and were found to be:

\[ b = 0.50 \]

\[ e = 0.40 \]

\[ j = 0.10 \]

Since the width and depth for a given discharge vary widely from one cross-section to another the values of \( a, c \) and \( k \) will also vary, and so far the factors governing these variations are yet to be determined.

2.6 Einstein and Barbarossa Equation

Einstein and Barbarossa assumed that the hydraulic radius consisted of two parts; the hydraulic radius \( R' \) due to surface
roughness and the hydraulic radius $R - R'$ due to form roughness.

Doland and Chow have taken this idea and have shown that the function

$$
\Phi \left( \frac{R}{d} \right) = \frac{0.0342}{(R'/R)^{2/3}}
$$

and have developed figure 8 to evaluate $(R'/R)$. Thus Manning's $N$ can be written as

$$
N = \Phi \left( \frac{R}{d} \right) d^{1/6} = \frac{0.0342}{(R'/R)^{2/3}} d^{1/6}
$$

### 2.7 Simons and Richardson Equations

Simons and Richardson developed a number of equations from which Chezy's $C$ can be derived once the bed form is determined. They also developed figure 9 to classify the bed form. For dune beds the equation is of the form

$$
\frac{C}{\sqrt{g}} = 7.4 \log_{10} \left( \frac{D}{d_{35}} \right) \sqrt{\frac{1 - \frac{AR}{S}}{RS}}
$$
CHAPTER 3
DEVELOPMENT OF FORMULAE

3.1 Manning - Strickler equations

In the previous chapter the various equations governing the transport of bed material were discussed. These equations along with Manning's equation were used to describe the flow of the water-sediment mixture.

Manning's equation can be written as

\[ q = \frac{1.486 A R^{2/3} S^{1/2}}{N B} \]

or

\[ q = \frac{1.486 B D^{2/3} S^{1/2}}{N B} \]

where the approximation \( D = R \) is used

\[ q = \frac{1.486 D^{5/3} S^{1/2}}{N} \] \hspace{1cm} (3.1)

Substituting Strickler's empirical relation

\[ N = 0.0342 d^{1/6} \] \hspace{1cm} (3.3)

where \( d \) is the grain size in feet

\[ q = \frac{43.4 D^{5/3} S^{1/2}}{d^{1/6}} \] \hspace{1cm} (3.4)

\[ S = \frac{0.00053 q^2 d^{1/3}}{D^{10/3}} \] \hspace{1cm} (3.5)

where \( N \) is Manning's resistance coefficient

\( B \) is the channel width

\( d \) is the grain size in feet
3.2 Du Boys-Manning-Strickler-tractive Force Equations

\[ q_s = C_s T_o (T_o - T_c) = \frac{0.173 T D S (T D S - T_c)}{d_{mm}^{3/4}} \] (3.6)

Substituting Manning's \( S \) from equation (3.4)

\[ q_s = \frac{0.173 T D}{d_{mm}^{3/4}} \left( \frac{0.00053 q^{1/3}}{D^{10/3}} \right) \left[ T D \left( \frac{0.00053 q^{1/3}}{D^{10/3}} \right) - T_c \right] \] (3.7)

Dividing by Manning's \( q^2 \) from equation (3.4)

\[ q_s = \frac{0.173 T D S (T D S - T_c)}{q^{2/3} d_{mm}^{1/6}} \left( \frac{1/6}{d_{ft}^{1/6}} \right)^2 \left( \frac{43.4 D^{5/3} S^{1/2}}{0.00053 q^{1/3}} \right) \] (3.8)

Substituting \( d = 0.60 \) mm and \( T_c = 0.0004 \) from figure 1

\[ q_s = 0.424 \times 10^{-5} \frac{q^4}{D^{4.67}} - 0.259 \times 10^{-4} \frac{q^2}{D^{2.33}} \] (3.9)

\[ q_s = 0.0648 S - 0.259 \times 10^{-4} \frac{q^2}{D^{1.33}} \] (3.10)

or \( S = \left( \frac{q_s + 0.259 \times 10^{-4} \frac{q^2}{D^{2.33}}}{q_s \frac{2}{q} \frac{1}{D^{2.33}}} \right) \) (3.11)

Substituting \( d = 0.20 \) mm and \( T_c = 0.0003 \)

\[ q_s = 0.465 \times 10^{-5} \frac{q^4}{D^{4.67}} - 0.638 \times 10^{-5} \frac{q^2}{D^{2.33}} \] (3.12)

\[ S = \left( \frac{q_s + 0.307 \times 10^{-4} \frac{q^2}{D^{2.33}}}{q_s \frac{2}{q} \frac{1}{D^{2.33}}} \right) \) (3.13)
3.3 Shields-Manning - Strickler- Tractive Force Equations

\[ \frac{q_S S_s}{q S} = 10 \left( \frac{T_D - T_c}{S_s - 1} \right) \]  

(3.14)

\[ \frac{q_S}{q S} = \frac{10 S (T D S - T_c)}{S_s T (S_s - 1) d_{ft}} \]  

(3.15)

Substituting Manning's S from equation (3.4)

\[ q_S = \frac{10 q}{S_s T (S_s - 1) d_{ft}} \left( \frac{0.0053 q d_{ft}^{1/3}}{d^{10/3}} \right)^{1/3} \left( \frac{T_D 0.0053 q d_{ft}^{1/3}}{d^{10/3}} - T_c \right) \]  

(3.16)

Dividing by Manning's q2 from equation (3.3)

\[ q_S = \frac{10 q}{S_s T (S_s - 1) d_{ft}} \left( \frac{d_{ft}^{1/6}}{43.4 D^{5/3} S^{1/2}} \right)^2 \]  

(3.17)

Substituting d = 0.60 mm and Tc = 0.001 from figure 1

\[ q_s = 0.5 \times 10^{-5} \frac{q^5}{5.67} - 0.76 \times 10^{-5} \frac{q^3}{D^{3.33}} \]  

(3.18)

\[ \frac{q_S}{q} = \frac{0.76 S}{D^{2.33}} - 0.76 \times 10^{-5} \frac{q}{D^{3.33}} \]  

(3.19)

or

\[ S = \left( \frac{q_S}{q} + \frac{0.76 \times 10^{-5}}{D^{3.33}} \right) \frac{D^{2.33}}{0.076} \]  

(3.20)

Substituting d = 0.20 mm and Tc = 0.0006

\[ q_s = 0.72 \times 10^{-5} \frac{q^5}{5.67} - 0.95 \times 10^{-5} \frac{q^3}{D^{3.33}} \]  

(3.21)

\[ \frac{q_S}{q} = \frac{0.95 \times 10^{-5}}{D^{3.33}} \frac{D^{2.33}}{1.58} \]  

(3.22)
3.4 Einstein-Manning-Strickler-Tractive Force Equations

\[ \frac{q_{s}}{w} = \frac{q_{s}}{G} \sqrt{g} \frac{d}{(S_{s} - 1)} \frac{D^{3}}{(S_{s} - 1)^{3}} \]  

Substituting Manning's \( S \) from equation (3.4)

\[ q_{s} = G \sqrt{g} \frac{d}{(S_{s} - 1)} \frac{D^{3}}{(S_{s} - 1)^{3}} \left( \frac{0.0053 q^{2} d^{1/3}}{D^{10/3}} \right)^{3} \]

\[ q_{s} = 0.929 \times 10^{-8} \frac{G d^{6}}{d^{1/2} D^{7}} \]  

Dividing by Manning's \( q \) from equation (3.3)

\[ \frac{q_{s}}{q} = G \sqrt{g} \frac{d}{(S_{s} - 1)} \frac{D^{3}}{(S_{s} - 1)^{3}} \frac{1/6}{S^{1/2}} \]

or \( S = 0.854 \left( \frac{1}{G} \right)^{0.4} \left( \frac{q_{s}}{q} \right)^{0.4} \left( \frac{d}{D} \right)^{0.533} \)

3.5 Kalinske-Manning-Strickler-Tractive Force Equations

\[ q_{s} = 10 \left( \frac{T_{0}}{T (S_{s} - 1)} \right)^{2} \]

\[ q_{s} = 10 \frac{T_{0}^{5/2}}{T^{1/2} (S_{s} - 1)^{2}} \frac{D^{5/2}}{S^{5/2}} \]

Substituting Manning's \( S \) from equation 3.4

\[ q_{s} = 10 \frac{T_{0}^{1/2} D^{5/2}}{(1.95)^{1/2} (S_{s} - 1)^{2}} \frac{0.0053 q^{2} d^{1/3}}{D^{10/3}} \]

\[ = 0.13 \times 10^{-6} \frac{q^{5}}{d^{0.167} D^{5.83}} \]
Dividing by Manning's $q$ from equation (3.3)

\[
\frac{q_{s}}{q} = \frac{10 T^2 d^{5/2} S^{5/2}}{\sqrt{\rho} (s_{s} - 1)^2 d \left( \frac{d^{1/6}}{43.4 D^{5/3} S^{1/2}} \right)} \tag{3.32}
\]

or

\[
S = 1.45 \left( \frac{q_{s}}{q} \right)^{0.5} \left( \frac{d}{D} \right)^{0.42} \tag{3.33}
\]

### 3.6 Liu-Hwang Equations

In sections 3.2 to 3.5 various equations were developed using the Manning-Strickler equations. These equations were originally deduced for uniform flow conditions with a plane bed. An alternative form of equation may be one developed using the fact that there was resistance due to the bed form. Thus equations were developed using the Liu-Hwang method. Using figure 4 it was found that the experiments all lay in the range of dune to transitional, and knowing that the experiments gave a dune type profile, it was decided to use the line representing dunes in interpreting the figures.

It was found that

\[
V = 22 R^{0.585} S^{0.336} \quad \text{for } d = 0.60 \text{ mm} \tag{3.34}
\]

\[
V = 11 R^{0.430} S^{0.305} \quad \text{for } d = 0.20 \text{ mm} \tag{3.35}
\]

or

\[
q = 22 R^{1.585} S^{0.336} \quad \text{for } d = 0.60 \text{ mm} \tag{3.36}
\]

\[
q = 11 R^{1.430} S^{0.305} \quad \text{for } d = 0.20 \text{ mm} \tag{3.37}
\]

or

\[
S = 0.0001 \frac{q^{2.98}}{D^{4.72}} \quad \text{for } d = 0.60 \text{ mm} \tag{3.38}
\]

\[
S = 0.000358 \frac{q^{3.28}}{D^{4.68}} \quad \text{for } d = 0.20 \text{ mm} \tag{3.39}
\]
3.7 DuBoys-Liu-Hwang-Tractive-Force-Equations

\[ q_s = \frac{.176}{d_{\text{mm}}^{3/4}} T D S (T D S - T_c) \quad (3.40) \]

\[ q_s = 1005 D^2 S^2 - .00645 D S \quad \text{for } d=.60 \text{ mm} \quad (3.41) \]

\[ q_s = 2280 D^2 S^2 - .011 D S \quad \text{for } d=.20 \text{ mm} \quad (3.42) \]

Substituting \( S \) from Liu-Hwang equation

\[ q_s = .1005 \times 10^{-4} \frac{q^{5.96}}{D^{7.54}} - .645 \times 10^{-5} \frac{q^{2.98}}{D^{3.72}} \quad \text{for } d=.60 \text{ mm} \quad (3.43) \]

\[ q_s = .3380 \times 10^{-3} \frac{q^{6.56}}{D^{7.36}} - .424 \times 10^{-5} \frac{q^{3.28}}{D^{3.68}} \quad \text{for } d=.20 \text{ mm} \quad (3.44) \]

Divide by \( S \) from Liu-Hwang equations and solve for \( S \)

\[ S = \left( \frac{q_s}{q} \right)^{2.72} \left( \frac{.489 \times 10^{-6}}{D^{3.72}} \right) \quad \text{for } d=.60 \text{ mm} \quad (3.45) \]

\[ S = \left( \frac{q_s}{q} \right)^{2.68} \left( \frac{.550 \times 10^{-5}}{D^{3.68}} \right) \quad \text{for } d=.20 \text{ mm} \quad (3.46) \]

3.8 Shields-Liu-Hwang-Tractive-Force-Equations

\[ \frac{q_s S_s}{q S} = 10 \left( \frac{T_0 - T_c}{T (S_s - 1) d} \right) \quad (3.47) \]

\[ q_s = 1162 D S^2 - .001862 S \quad \text{for } d=.60 \text{ mm} \quad (3.48) \]

\[ q_s = 3430 D S^2 - .0033 S \quad \text{for } d=.20 \text{ mm} \quad (3.49) \]
Substituting $S$ from the Liu-Hwang equations

$$q_s = 1162 \times 10^{-4} \frac{q^6}{D^{8.54}} - 1862 \times 10^{-6} \frac{q^3}{D^{4.72}}$$
for $d = .60 \text{ mm}$ \hspace{1cm} (3.50)

$$q_s = 5080 \times 10^{-3} \frac{q^7}{D^{8.36}} - 1270 \times 10^{-5} \frac{q^4}{D^{4.68}}$$
for $d = .20 \text{ mm}$ \hspace{1cm} (3.51)

Dividing by $S$ from the Liu-Hwang equations and solving for $S$

$$S = \left( \frac{q_s}{q} + \frac{1.162 \times 10^{-6}}{D^{4.72}} \right) \frac{D^{3.72}}{.1162}$$
for $d = .60 \text{ mm}$ \hspace{1cm} (3.52)

$$S = \left( \frac{q_s}{q} + \frac{.73 \times 10^{-6}}{D^{4.68}} \right) \frac{D^{3.68}}{.757}$$
for $d = .20 \text{ mm}$ \hspace{1cm} (3.53)

3.9 Einstein-Liu-Hwang-Tractive Force- Equations

$$q_s = 40 \left( \frac{T_o}{T (S_s - 1)} \right)^3$$
\hspace{1cm} (3.54)

$$q_s = 539000 \ D^3 \ S^3$$
for $d = .60 \text{ mm}$ \hspace{1cm} (3.55)

$$q_s = 198000 \ D^3 \ S^3$$
for $d = .20 \text{ mm}$ \hspace{1cm} (3.56)

Substituting $S$ from the Liu-Hwang equation

$$q_s = .539 \times 10^{-6} \frac{q^8.94}{D^{11.16}}$$
for $d = .60 \text{ mm}$ \hspace{1cm} (3.57)

$$q_s = .113 \times 10^{-4} \frac{q^9.84}{D^{11.04}}$$
for $d = .20 \text{ mm}$ \hspace{1cm} (3.58)
Dividing by $S^2$ from Liu-Hwang equation and solving for $S$

\[
S = \frac{186.7 \, q_s \, D}{q} \quad \text{for } d = .60 \text{ mm} \quad (3.59)
\]

\[
S = \frac{34.07 \, q_s \, D}{q} \quad \text{for } d = .20 \text{ mm} \quad (3.60)
\]

3.10 Kalinske-Liu-Hwang-Tractive Force- Equations

\[
\frac{q_s}{\sqrt{T_0/\rho}} = 10 \left( \frac{T_0}{T \,(S_s-1) \, d} \right)^2 \quad (3.61)
\]

\[
q_s = 10580 \, D^{5/2} \, S^{5/2} \quad \text{for } d = .60 \text{ mm} \quad (3.62)
\]

\[
q_s = 31200 \, D^{5/2} \, S^{5/2} \quad \text{for } d = .20 \text{ mm} \quad (3.63)
\]

Substituting $S$ from the Liu-Hwang equations

\[
q_s = 1.058 \times 10^{-5} \, \frac{q^{7.54}}{d^{9.30}} \quad \text{for } d = .60 \text{ mm} \quad (3.64)
\]

\[
q_s = 9.1 \times 10^{-4} \, \frac{q^{6.21}}{d^{9.21}} \quad \text{for } d = .20 \text{ mm} \quad (3.65)
\]

Dividing by $S^{3/2}$ from Liu-Hwang and solving for $S$

\[
S = \frac{94.5 \, q_s \, D^{4.58}}{q} \quad \text{for } d = .60 \text{ mm} \quad (3.66)
\]

\[
S = 4.22 \, q_s \, D^{4.52} \quad \text{for } d = .20 \text{ mm} \quad (3.67)
\]
3.11 Einstein-Barberossa-Equation

From experiment 7-4.32
\[ \frac{d_{35}}{R S} = 1.975 \]
\[ \left( \frac{R}{d_{65}} \right)^{1/3} = 3.76 \]

From figure 8 \( R'/R = 0.35 \)

\[ N = \frac{0.0342}{d^{1/6}} = 0.0242 \]
\[ \left( \frac{0.35}{2/3} \right) \]

\[ V = 61.5 R^{2/3} S^{1/2} \quad (3.68) \]

3.12 Simons and Richardson Equation

From experiment 7-4.32

\[ S \text{ energy} = 0.008 \]
\[ V = 0.5609 \text{ ft/sec} \]
\[ R = 0.114 \text{ ft} \]

Stream Power = TRS V = 0.317

From figure 9 the experiment is in the dune range, thus use equation

\[ \frac{C}{\sqrt{g}} = 7.4 \log_{10} \left( \frac{D}{d_{35}} \right) \sqrt{1 - \frac{\Delta R S}{R S}} \]

\[ \frac{C}{\sqrt{32.2}} = 7.4 \log_{10} \left( \frac{0.1258}{0.7 * 0.03937/12} \right) \sqrt{1 - 0.00055} \]

\[ C = 45 \]

\[ V = 45 R^{1/2} S^{1/2} \quad (3.69) \]
CHAPTER 4

EXPERIMENTAL APPARATUS AND PROCEDURE

4.1 The flume

The experiments were carried out in a model reservoir built in a recirculating flume 15 feet long, 5 feet wide and 4 feet deep as shown in figure 11. The flume is of welded steel construction with a plexiglass sidewall for observations. The size of the flume allows a wide selection in lengths, widths and depths for the models built inside it.

4.2 Selection of reservoir shape

In studying a hydraulics problem on reservoir sedimentation the configuration of the reservoir must be defined. Van't Hul (15) has proposed a system of classifying reservoirs by using a plot of the reservoir capacity versus depth curve. The points when plotted usually fall in a straight line on log-log paper, the equation of which is in the form:

\[ H = KC^n \]

where \( H \) is the reservoir depth measured from the bottom.

\( C \) is the reservoir capacity.

\( K \) is a proportionality constant.

\( n \) is the slope of the line.

Using "m", the reciprocal of the slope "n", four classes of reservoirs can be described. As can be seen in figure 14, the gorge type reservoir has a small rate of increase in capacity with depth, whereas, the lake type has a large rate of increase of capacity with depth. It must be remembered that the depth is
measured as the distance from the bottom.

<table>
<thead>
<tr>
<th>&quot;m&quot;</th>
<th>Reservoir Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 - 1.5</td>
<td>Gorge</td>
</tr>
<tr>
<td>1.5 - 2.5</td>
<td>Hill</td>
</tr>
<tr>
<td>2.5 - 3.5</td>
<td>Flood Plain......foothill</td>
</tr>
<tr>
<td>3.5 - 4.5</td>
<td>Lake</td>
</tr>
</tbody>
</table>

For the present study it was decided to use a gorge type. The tapered inlet section was used to simulate a stream expanding into the reservoir and the tapered outlet section was used since most dams are built at the narrowest part of the river.

4.3 The model

The model is of 3/4" plywood construction resting on 2"*4" supports. The slope of the model was set at 0.01. The model consisted of a one foot wide entrance channel, 2 feet long, which expanded into a two and one half foot wide test section, 10 feet long which then contracted to an outflow area one foot wide as shown in figure 12. The depth of water in the model was controlled using a horizontal, sharp crested, overflow weir at the outlet section.

Flow enters the model through a horizontal tube type flow straightener which effectively distributes the water evenly across the entrance channel.

To achieve a uniform bed load rate it was necessary to add the sand upstream of the test section at a constant rate throughout the experiment. This was achieved by installing a large storage hopper above the entrance channel and letting the sand flow out the bottom of this through a small orifice. It was found that by using
various size orifices it was possible to set nearly any sand flow rate desired. Separate calibration runs were made for each orifice and sand size and it was found that the sand flow rate was constant until the 6 cubic foot hopper was nearly empty, so that by periodically topping up the hopper it was possible to run any length of experiment. The sand had to be dried between each use and care was taken not to overheat it in the drying ovens as cracking of the sand grains could have happened. Continuous checking of the size distribution proved that this was not occurring. The sand flow out of the orifice was spread over the one foot width of the entrance channel by having the sand stream strike and spray off the end of a brass stud. Even though this method sounds very crude it was very effective in distributing the sand evenly.

4.4 Bed material

The two separate sizes of sand used in this experiment were Ottawa Flint Shot with a mean diameter of .60 mm and Ottawa Silica 90 with a mean grain size of .20 mm. These sands were chosen since they are practically all one grain size which can be seen from figure 13. It was found that there was no selective sorting of the sand during the tests due to this narrow size distribution.

4.5 Laboratory procedure

The laboratory procedure followed for each run involved recirculating a given discharge through the model while adding a constant sediment load upstream of the test section. The experiment was continued until the reservoir filled with sand and equilibrium conditions were established, with sand being carried through the section and over the spillway at the same rate as it was being...
In the model it was possible to set any flow, sediment discharge or initial storage capacity desired. The flow dynamics then would build up a sand bed, the width of the model, until a depth and slope had been established that would transport the entire sediment load.

The time required to establish equilibrium conditions varied with the flow, sediment load and initial storage capacity. Some runs with lower flows and concentrations required 12 to 14 hours to achieve equilibrium, whereas, at higher sediment loads equilibrium was established in 5 to 6 hours.

4.6 Data Obtained

a) Water surface slope

Water surface elevations were measured at three points along the length of the model using precision Ott electric point gauges, which measured to .001 of a foot. The maximum and minimum elevations were averaged, and the slope calculated by dividing the horizontal distance between point gauges into the difference in their mean water levels.

b) Bed slope

The bed slope was determined using two methods. First the active bed elevations were measured during each run using the same method as that for water surface and the average slope calculated.

In the second method the bed elevations were measured after the flow had been stopped and the flume carefully
drained, so as not to disturb the bed configurations. These measurements were made every four inches along the flume using an Ott point gauge which read directly to .01 of a inch. These elevations were plotted on graph paper and for that portion where a constant slope had been established an average slope was determined using the method of least squares.

c) Discharge

To measure the discharge a calibrated, sharp-crested $15^\circ$ V notch weir was installed in the return channel between the two sections of the sump. It was possible to measure flows from 20 to 222 G.P.M.(U.S.) to within an accuracy of 1 G.P.M.

The water discharge was measured four or five times each run. The average discharge for a run is the mean of these measurements.

The V notch was calibrated using a venturi meter for the higher flows and a catch tank and stop watch for the very low flows.

d) Depth

The flow depth was calculated by subtracting the average bed level from the average water surface level at each of the three measuring points and then averaging these 3 readings. It was necessary to modify this procedure to take into account that the sand had not been carried into the reach of all the point gauges until near the end of each run.
**e) Velocity**

The velocity was calculated using the continuity equation or in other words by dividing the flow by the cross-sectional area.

**4.7 Bed configuration**

Centerline profiles and contour maps were made of the bed deposits after each run. The contours were laid out during the draining of the flume by stopping at each inch drop in water level and placing a string around the water edge. This is shown in Photo 2.

The centerline profile was measured by taking bed elevations every four inches along the flume using a precision point gauge.

In addition to measuring the bed configuration a series of photographs of the bed were taken from above the flume and through the plexiglass side wall, showing the progression of the bed form into the reservoir with time.
CHAPTER 5

ANALYSIS AND DISCUSSION

5.1 Bed load equations

The first investigation carried out was to see if the equations of Du Boys, Shields, Einstein and Kalinske, when used along with the well known Manning's equation could describe the sediment transport capacity.

Figures 15 to 18 show how close each equation came in predicting the actual bed load. As can be seen there is a considerable amount of scatter, but this is all well within the factor 10 which is common in sediment transport problems. At first glance it is quite obvious that Du Boys' equation does not describe the action very well at all for it comes up with negative transport rates which are physically impossible. To show which equation best fits the data an analysis was done on the errors, with error being defined as $|q_s \text{ experiment} - q_s \text{ equation}|$. As can be seen from figure 19, Du Boys' has the greatest error while Einstein and Kalinske essentially have the smallest error. Thus the equations of Einstein and Kalinske offer the best solution even though the mean error is in the same order of magnitude as the experimental bed load.

5.2 Slope equations

Figures 20 through 23 show how well the equations developed describe the slope of the bed form. Again there is a considerable amount of scatter but this time points lie both above and below the $45^\circ$ line showing that there is no bias in the equations to either overestimate or underestimate the slopes. Again the errors, $|S \text{ exp} - S \text{ equation}|$, were examined to see which equation best described
the experiments and as can be seen from figure 24 the Shield's equation has the largest mean error and the largest variance of this error; therefore this is the poorest equation. In the remaining three equations there is only a marginal improvement of one equation over the next but Kalinske's equation is again the best due to it being more consistent as shown by it having a smaller standard deviation.

5.3 Liu-Hwang equations

As can be seen in figures 25 to 28 the Qs values as found by the Liu-Hwang equations are much greater than those found from the Manning-Strickler equations. The values for S given by this method as shown in figures 29 to 32 are very much smaller than those given by the Manning-Strickler equations or even those found in the experiments.

It is obvious that by increasing the resistance by including some form resistance tends to reduce the energy slopes required to transport a given bed load or conversely it tends to increase the sediment carrying capacity of a channel for a given slope. However, from this study it seems that the equations developed from the Liu-Hwang graphs greatly overestimate the bedform resistance and thus give incorrect answers.

5.4 Other Form Resistance Equations

The equations of Einstein and Barbarossa as well as those of Simons and Richardson as developed in sections 3.11 and 3.12, do not change the basic equations as much as those of Liu and Hwang and thus will have a lesser effect on the Qs and S values.
Again the order of magnitude of the change is much too great and gives incorrect final results.

5.5 Stream Geometry Equations

The next investigation carried out was to see how the data obtained compared to the regime equations of Lacey and the stream geometry equations of Leopold and Maddock. As can be seen in table 1, the experiments were only conducted at two discharges, thus only general trends in the relationships of the parameters versus discharge can be made. The values of the variables at each discharge were averaged and the equations were computed as follows:

\[ P = a Q^b \]  
\[ 2.673 = a (0.08733)^b \]
\[ 2.721 = a (0.17645)^b \]
\[ \frac{2.673}{2.721} = \frac{0.08733}{0.17645} \]
\[ 0.983 = 0.4945^b \]
\[ b = 0.0244 \]
\[ 2.721 = a (0.17645)^{0.0244} \]
\[ a = 2.74 \]
Therefore for the reservoir experiments

\[ P = 2.74 \, Q^{0.0244} \quad (5.1) \]

Lacey's comparable equation is

\[ P = 2.67 \, Q^{0.5} \quad (2.13) \]

On comparing the two equations the point to note is that the powers of the equation are quite different. This is easily understood if one realizes that in the present study the reservoir was full of water at the start of each run and this water level and discharge remained constant throughout the experiment. There was no action that would cause berms or sides to build up since the water cannot move the sand grains higher than its own surface, thus the width was forced to stay at 2.5 feet. The constant 2.74 even though it is very close to that of Lacey's could have had any other value if widths had been chosen other than 2.5 feet.

Leopold and Maddock used the width rather than the wetted perimeter and developed the equation:

\[ W = a \, Q^{0.26} \quad \text{for a given cross-section} \quad (2.18) \]

\[ W = a \, Q^{0.50} \quad \text{for all cross-sections in a basin} \]

From our studies

\[ W = a \, Q^{b} \]

\[ = 2.5 \, Q^{0.0} \]

\[ W = 2.5 \, \text{feet} \quad (5.2) \]

Lacey also developed an equation for mean slope which was

\[ S = 0.000547 \, Q^{5/3} \, Q^{-1/6} \quad (2.14) \]
or \( S = 0.0056 Q^{-1/6} \)  \hspace{1cm} (5.3)

From the present studies

\[
S = a Q^b \quad \text{(as shown in figure 35)}
\]

\[
0.00561 = a \left(0.08733\right)^b \\
0.01026 = a \left(0.17645\right)^b \\
b = 0.86 \\
a = 0.00456
\]

\[
S = 0.00456 Q^{0.86} \quad \text{(5.4)}
\]

This shows quite readily that in this case the slope is adjusting itself to transport the bed load since the width was unable to adjust itself.

5.6 Depth equations

This phenomenon becomes even more apparent when we compare the depth equations of Lacey, Leopold and Maddock and those developed from the experimental data.

\[
D = i Q^j \quad \text{(as shown in figure 36)}
\]

\[
0.086 = i \left(0.08733\right)^j \\
0.110 = i \left(0.17645\right)^j \\
j = 0.35 \\
i = 0.202
\]

\[
D = 0.202 Q^{0.35} \quad \text{(5.5)}
\]

Lacey gives

\[
D = \frac{0.473 Q^{1/3}}{f^{1/3}} = 0.442 Q^{0.33} \quad \text{(5.6)}
\]
Leopold and Maddock give

\[ D = i Q^{0.40} \]  

(2.19)

All of the above equations show a very good agreement, showing that the depth of flow attained is essentially the same in our case of having the width constrained as those of Leopold and Maddock. Showing that the major adjustment for the transport of bed load, for changing flows, is in the bed slope as shown in figure 37.

5.7 Other factors

The other factors that have been traditionally investigated are area and velocity. In this study our area equation has the same exponent as the depth equation since the width was independent of Q, and area = width * depth.

The velocity equation was found to be

\[ V = 2.02 Q^{0.66} \]  

(as shown in figure 38)  

(5.7)

which bears little resemblance to Lacey's equation of

\[ V = 0.956 Q^{1/6} \]  

(5.8)

This is to be expected, however, since our relationship of cross-sectional areas were different it is only natural that the relationship for velocity would be different since \[ Q = A \times V \].

5.8 Energy Slopes

As can be seen from figure 33 the energy slopes for many of the experiments are closely related to the bed slope. Points lying outside the dotted line, which shows the limit of the measurement error, can be suspected to have some abnormal error in the measurement.
5.9 Sediment Deposits

From figures 39 to 62 it is apparent that the bed form quickly adjusts itself to the full width of the reservoir. If the flow builds up a deposit on one side of the channel the flow will be deflected to the opposite side of the channel. Then the sediment will begin to build up in this region. By successive actions of this type the deposit builds up across the entire width.
CONCLUSIONS

From this study the following conclusions can be drawn.

6.1 As discussed in sections 5.1 to 5.4 the Einstein and Kalinske equations are superior to the Du Boys and Shields equations in estimating the bed load and slope for these channels.

6.2 As discussed in section 5.5 the slope of these channels will be a function of the flow through the reservoir.

6.3 As discussed in section 5.6 the flow depth follows the same relationships for channels formed by building up bed deposits in backwater as those formed by streams cut into uplifted regions.

6.4 As discussed in section 5.9, channels built up in backwater will tend to become the full width of the reservoir.
FIGURE 1. ALLOWABLE TRACTIVE FORCES IN NONCOHESIVE BED MATERIAL
FIGURE 2. EINSTEIN BED LOAD FUNCTION

\[ \Phi = 40 \left( \frac{1}{\Psi} \right)^3 \]

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\[ \frac{q_s}{\sqrt{d}} = 10 \left( \frac{1}{\Psi} \right)^2 \]

**FIGURE 3. KALINSKE BED LOAD FUNCTION**

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FIGURE 4 CLASSIFICATION OF BED CONFIGURATION

FIGURE 5. X VS d
FIGURE 6. Y VS d

FIGURE 7. Ca VS d

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FIGURE 8. VALUES OF \( \frac{R'}{R} \)
FIGURE 9. CLASSIFICATION OF BED CONFIGURATION

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FIGURE 10. ΔRS VS RS FOR DUNE TYPE BED

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FIGURE 11. DETAILS OF FLUME
FIGURE 12 DETAILS OF MODEL
FIGURE 13. GRAIN SIZE DISTRIBUTION

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FIGURE 14. RESERVOIR SHAPES
FIGURE 15 $Q_s$-DUBOYS VS $Q_s$-EXPERIMENTAL
FIGURE 16 $Q_s$-SHIELDS VS $Q_s$-EXPERIMENTAL

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FIGURE 18. $Q_s$-KALINSKE VS $Q_s$-EXPERIMENTAL

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FIGURE 19. DISTRIBUTION OF ERRORS IN Qs EQUATIONS

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FIGURE 20. S-DUBOYS VS S-EXPERIMENTAL

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FIGURE 21. S-SHIELDS VS S-EXPERIMENTAL

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FIGURE 22 S-EINSTEIN VS S-EXPERIMENTAL
FIGURE 23 S-KALINSKE VS S-EXPERIMENTAL

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FIGURE 24. DISTRIBUTION OF ERRORS IN S EQUATIONS
FIGURE 25. Qs-DUBOYS VS Qs-EXPERIMENTAL

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FIGURE 26. $Q_s$-SHIELDS VS $Q_s$-EXPERIMENTAL

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FIGURE 27. Qs-EINSTEIN VS Qs-EXPERIMENTAL

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FIGURE 28 Qs-KALINSKE VS Qs-EXPERIMENTAL

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FIGURE 29. S-DUBOYS VS S-EXPERIMENTAL

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FIGURE 32 S-KALINSKE VS S-EXPERIMENTAL

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FIGURE 33. BED SLOPE VS ENERGY SLOPE

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P = 2.74 Q^{0.244}

FIGURE 34. WETTED PERIMETER VS DISCHARGE

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$S = 0.00456 \ Q^{.86}$

**FIGURE 35. BED SLOPE VS DISCHARGE**

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FIGURE 36. FLOW DEPTH VS DISCHARGE

\[ D = 0.202 Q^{0.35} \]
FIGURE 37. FLOW AREA VS DISCHARGE

A = 0.54 Q^{0.346}

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EXPERIMENT 1

TIME 1:30
FLOW 198 G.P.M.
CONC 2040 P.P.M.
ELEV 6 5/8"
SAND-FLINT SHOT

FIGURE 40

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EXPERIMENT 1
TIME 3:00
FLOW 198 G.P.M.
CONC 2040 P.P.M.
ELEV $6\frac{6}{8}$
SAND-FLINT SHOT

FIGURE 41

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EXPERIMENT 1
TIME 4:00
FLOW 198 G.P.M.
CONC 2040 PPM.
ELEV 6' 6-3/8"
SAND-FLINT SHOT

FIGURE 42

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EXPERIMENT 1
TIME 6:00
FLOW 198 G.P.M.
CONC 2040 P.P.M.
ELEV 6\(\frac{5}{8}\)
SAND-FLINT SHOT

FIGURE 43
EXPERIMENT 2
TIME 1:00
FLOW 198 G.P.M.
CONC 1000 P.P.M.
ELEV 6\frac{5}{8}"
SAND-FLINT SHOT

FIGURE 44
EXPERIMENT 2
TIME 3:00
FLOW 198 G.P.M.
CONC 1000 P.P.M.
ELEV 6 5/8
SAND-FLINT SHOT

FIGURE 45

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EXPERIMENT 2
TIME 8:00
FLOW 198 G.P.M.
CONC 1000 PPM.
ELEV 6\frac{5}{8}"
SAND-FLINT SHOT

FIGURE 47

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EXPERIMENT 2
TIME 11:43
FLOW 198 G.P.M.
CONC 1000 PPM.
ELEV 6 5/8
SAND-FLINT SHOT

FIGURE 48
PLAN PROFILE

EXPERIMENT 3
TIME 4:22
FLOW 98 G.P.M.
CONC 2020 P.P.M.
ELEV 6 5/8
SAND-SHELL 90

FIGURE 49

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EXPERIMENT 3
TIME 9:21
FLOW 98 G.P.M
CONC 2020 PPM.
ELEV 6'5\text{\textonehalf}"
SAND-SHELL 90

FIGURE 50
EXPERIMENT 3
TIME 14:00
FLOW 98 G.P.M.
CONC 2020 PPM.
ELEV 6 5/8
SAND-SHELL 90

FIGURE 5
EXPERIMENT 4
TIME 4:14
FLOW 98 G.P.M.
CONC 2030 P.P.M.
ELEV 6 5/8"
SAND-FLINT SHOT

FIGURE 52

85

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EXPERIMENT 4
TIME 9:28
FLOW 98 G.P.M.
CONC 2030 PPM.
ELEV 6 5/8" SAND-FLINT SHOT

FIGURE 53
EXPERIMENT 4
TIME 14:23
FLOW 98 G.P.M.
CONC 2030 PPM.
ELEV 6 \( \frac{1}{8} \)
SAND-FLINT SHOT

FIGURE 54
EXPERIMENT 5
TIME 1:46
FLOW 98 G.P.M.
CONC 2030 PPM.
ELEV 4"
SAND-FLINT SHOT

FIGURE 55
EXPERIMENT 5
TIME 4:32
FLOW 98 G.P.M.
CONC 2030 P.P.M.
ELEV 4"
SAND-FLINT SHOT

FIGURE 56

89

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EXPERIMENT 5
TIME 7:41
FLOW 98 G.P.M.
CONC 2030 P.P.M.
ELEV 4"
SAND-FLINT SHOT

FIGURE 57

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EXPERIMENT 6
TIME 9:28
FLOW 98 G.P.M.
CONC 2030 P.P.M.
ELEV 6\(\frac{5}{8}\)"
SAND-FLINT SHOT

FIGURE 58

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EXPERIMENT 6
TIME 16:00
FLOW 30 G.P.M.
CONC 0 P.P.M.
ELEV 5 5/8"
SAND-FLINT SHOT

FIGURE 59

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EXPERIMENT 7
TIME 1:10
FLOW 198 G.P.M.
CONC 2040 PPM.
ELEV 5\frac{3}{8}"
SAND-FLINT SHOT
EXPERIMENT 7
TIME 2:46
FLOW 198 G.P.M.
CONC 2040 PPM
ELEV 5 3/8"
SAND-FLINT SHOT

FIGURE 61

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EXPERIMENT 7
TIME 4:32
FLOW 198 G.P.M.
CONC 2040 P.P.M.
ELEV 5 3/8
SAND-FLINT SHOT
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|               | 1-4.00        | .01754     | .00720        | .00278        | .00650        | .00745        | -             | -             | -             |
|               | 1-6.00        | .01144     | .00733        | .00313        | .00620        | .00718        | -             | -             | -             |
|               | 2-1.00        | -          | -             | -             | -             | -             | -             | -             | -             |
|               | 2-3.00        | -          | -             | -             | -             | -             | -             | -             | -             |
|               | 2-6.00        | .00151     | .00561        | .00172        | .00521        | .00549        | -             | -             | -             |
|               | 2-8.00        | .01427     | .00563        | .00172        | .00542        | .00551        | -             | -             | -             |
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|               | 3-4.22        | -          | .00677        | .00568        | .00498        | .00519        | -             | -             | -             |
|               | 3-9.21        | .00779     | .00685        | .00639        | .00478        | .00503        | -             | -             | -             |
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### TABLE 5

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### TABLE 6

(by Manning-Strickler)  

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BUILD UP OF BED DEPOSITS AT CONSTANT DISCHARGE AND UNIFORM BED LOAD FOR EXPERIMENT 7.

photo 1

photo 2

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3- Brune, Gunnar M., "Trap Efficiency of Reservoirs", Transactions, American Geophysical Union, Vol 34 No 3 June 1953.


9- "Interim Report on Distribution of Sediment in Reservoirs", by the Bureau of Reclamation, 1954.


Born on May 13, 1943, in Windsor, Ontario, Canada.

Graduated from J.L. Forester Collegiate Institute, Windsor, Ontario.

Graduated from the University of Windsor, with a Bachelor of Applied Science in Civil Engineering.

Accepted as a candidate of the Master of Applied Science degree at the University of Windsor.