Optimum Sediment Transport in Pipes with Non-Deposited Beds

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Abstract

Transportation of solids in pipes has been used in a wide variety of applications, such as transport of coal, slurry, sand and pumping of wood chips, due to its ease of installation, monitoring, maintenance and cheap means of transport. Recently there is a growing interest in using pipelines to transport dredged sediment or deposited silt in reservoirs to suitable dumping sites. The procedure for designing such pipelines to perform under optimum condition has not yet reached the state of maturity as that of clear water pipeline system

Many researchers have focused on defining the self-cleansing or limiting velocity that necessary to maintain non-deposited bed or deposit-free condition. Though there are many equations available they rely on a wide range of parameters and give different results for similar conditions. This partly due to the limited range of tested parameters, complicated form of equations that relate the parameters and absence of standard design procedure such as that for clear water flow in pipes.

In this paper the well-known empirical Durand's equation will be used to derive analytically two equations for the critical velocity and the optimum sediment transport capacity in pipes with non-deposited beds. This velocity exceeds the limiting velocity defined under laboratory condition and thus corrects for the previous under estimation that claimed by many researchers. The critical velocity equation was emerged to be casted in a modified form of a Darcy-Weisbach equation. This will simplify the design and allow the use of the standard clear water flow equations and charts for sediment transport in pipes. The sediment transport capacity can then be easily evaluated via the second equation.

Key words: sediment transport in pipes, critical velocity, limiting velocity, non-settling velocity, minimum head loss, self-cleansing.

1. INTRODUCTION

Reservoir sedimentation is threatening the storage capacity and functioning of many reservoirs around the world. The conventional methods of flushing and sluicing alone have failed to deal with the threat which led researchers to think about alternative solutions. The use of a pipeline to transport sediment entering a reservoir or removing deposited sediment has been a subject of experimentation and investigation by many researchers to explore its potentiality. The system has already been applied successfully in some controlled processes e.g. dredging, transport of coal, slurry, sand and pumping of wood chips as well as wastewater. However the procedure for designing such a pipeline to perform under optimum conditions has not reached the state of maturity as that of a clear water pipeline system.

The problem is that the sediment transport velocity necessary to maintain deposit-free flow does not assume a single value but is dependent on a number of factors such as sediment concentration, grain size, cohesiveness plus all the other factors related to clear water flow.

Further, sediment is transported in a pipe under various different characteristic modes. Broadly these modes are: homogeneous, heterogeneous, and bed load (with deposit). In the homogeneous flow mode, the mixture is almost uniform across the section of the pipe. It is stated by various investigators (Durand (953) and Herbich(2000)) that for homogenous flow, which occurs with fine materials less than 40 microns, the clear water head loss equation can be used provided the properties of the mixture such as its density and viscosity are used.

In the heterogeneous flow regime there exists a concentration profile along the vertical with some bed load movement, but free from any deposits. This mode of transportation is the most encountered one in a non-uniform grain size situation and it is the most often investigated regime.

The bed load or deposit regime is characterised by the presence of deposits, bed load as well as suspension load. The presence of deposits greatly increases the head loss gradient and may lead to blockage of the pipe if there is a sudden increase in the sediment inflow rate.

This paper deals with the design of a pipeline sediment transport system that is expected to be operated under a heterogeneous flow regime. Through the application of the minimum head loss concept the intention is to find a way of expressing the head loss/velocity relationship in the well-established form of the Darcy-Weisbach equation for clear water flow. Use will be made of the well-known and well established sediment transport relations for pipes developed by Durand and his co-workers. The other reason for selecting this equation is that it contains all the necessary parameters related to pipeline design, while the majority of other pipeline sediment transport equations disregard some of the important parameters like the head loss or pipeline friction.

2. PREVIOUS STUDIES

2.1. General Sediment Transport Relationship

The flow of solid-liquid mixtures through pipes has been investigated by many researchers and they have suggested different equations. Vanoni (1975) carried out a comparative study between different pipe sediment transport formulae and recommended that of Durand. Durand and Co-workers carried out an extensive study of sediment transport in pipes with pipe diameters ranging from 38 mm to 700 mm, sediment concentrations ranging from 50 to 600 g/l, and grain sizes ranging from 20 microns to 100 mm. The Durand (1953) general transport equation reads

$$\phi = k\psi^{m} \tag{1}$$

Where ϕ and ψ are dimensionless parameters given by

$$\phi = \frac{J_m - J}{JC_v} \tag{2}$$

$$\psi = \left[\frac{V^2\sqrt{C_d}}{gd(s-1)}\right] \tag{3}$$

Where J_m and J are the head loss gradient for flow with and without sediment respectively,
V = velocity of flow, C_d = particle drag coefficient, d = pipe diameter, C_v = sediment concentration by volume, s = sediment specific gravity, and g = acceleration due to gravity. The values for the coefficients m and k have been given as -1.5 and 81 respectively

Some other researchers have cited different values for m and k. Zandi and Govatos (1967) analysed a large set of data including that of Durand and have suggested values of m = -1.93 and k = 280 for ψ <10, and values of m = -.354 and k = 6.3 for ψ > 10. Charles (1970) gave values of m = -1.5 and k = 120, but the RHS of Eq. 1 contains (s-1) as an extra term in order to remove the discontinuity between the homogeneous and heterogeneous modes of flow. Recently Hotchkiss and Huang (1995) carried some field tests at lake Atkinson, on the Elkhorn River, in Nebraska using a 152 mm pipe diameter, and they found that their data yielded m = -1.31 and k = 211. The maximum transported concentration was 2% and the sediment has a median diameter of 0.23 mm. However, in general most sediment transport formulae can be expressed in a form similar to that of Eq. 1.

2.2. Limiting Velocity of Sediment Transport

In general, most of the researchers have concentrated their effort in finding an expression for the critical velocity of transport rather than a general sediment transport formula in pipes as that of Eq. 1. Various terms such as critical, self-cleansing, free-deposited, and limiting velocity were used to describe the velocity of flow when no deposit was observed, but the sediment will start to deposit at the bed if the sediment transport rate was slightly increased. Durand (1953) defined the limiting velocity as

$$V_l = Fr_l\sqrt{2gd(s-1)} \tag{4}$$

Where, Fr_l is the Modified Froude Number which varies with the particle diameter and flow concentration. He presented his results graphically for uniform grain size and later Condolios and Chapus (1963) extended it to non-uniform material (**Fig 1**).

Mayerle et al (1991) have summarised most of the previous formulae of critical velocity. Almost in all cases the condition of no deposited sediment was determined by visual observation. There is no doubt

that this criterion can only indicate the safe transporting velocity or the minimum operational velocity for transportation of sediment without subjecting the system to the likelihood of interruption by blockage. However, the question of whether the system under this condition is performing optimally by delivering the maximum sediment discharge is not necessary true. Operation of the system at a velocity which is slightly higher than the observed non-settling velocity may produce safe operation as well as the optimum transport of sediment.
The minimum head loss concept was thought of as answering this question, but it was found that occasionally it underestimates the observed limiting velocity (Eftekharzadeh, 1987). A clarification for such underestimation will be explained and corrected for after the perceived minimum head loss (MHL) concept is dealt with in the following section.

2.3. Previous Application of The Minimum Head Loss Concept

The perceived minimum head loss concept is to find the optimal velocity at which the mixture head loss gradient is minimal. This can be found from any general sediment transport formula like that of Durand. Following the line of analysis of Eftekharzadeh (1987) and Hotchkiss and Huang (1995).The general equation of sediment transport in pipes can be written for any value of the constants k and m as

$$J_m = J + k\left[\frac{V^2\sqrt{c_d}}{gd(s-1)}\right]^m C_v J \qquad (5)$$

The clear water head loss gradient J is given by Darcy-Weisbach formula

$$J = f\frac{V^2}{2gd} \qquad (6)$$

Where f = Darcy-Weisbach friction coefficient
By substituting Eq. 6 into Eq. 5 and letting $dJ_m/dV = 0$ we get

$$V_m = \left[-(m+1)k\right]^{-\frac{1}{2m}}\left[\frac{gd(s-1)}{\sqrt{c_d}}\right]^{\frac{1}{2}} C_v^{-\frac{1}{2m}} \qquad (7)$$

If Durand's values for m and k, i.e. -1.5 and 81 respectively, are substituted in the above equation the velocity V_m will read

$$V_m = 3.43\left[\frac{gd(s-1)}{\sqrt{c_d}}\right]^{\frac{1}{2}} C_v^{\frac{1}{3}} \qquad (8)$$

The above equation was first obtained by Goedde (1978). Eftekharzadeh (1987) compared Eq. 8 with experimental data of the limiting transport velocity (Fig 1). He argued that the above equation is occasionally not reliable as it underestimates the limiting velocity that has been reported in experimental observation. He also commented that the mathematical development of the minimum energy concept might not be complete. In the following a modified mathematical development for the minimum head loss will be introduced.

3. REFORMULATION OF THE MINIMUM HEAD LOSS CONCEPT

To correct for the previous underestimation of the critical transport velocity the whole concept of the minimum head loss was reviewed and reformulated into a new shape. Two sources of possible errors were identified. Since the differentiation is taken with respect to the velocity, the concentration C_v and the coefficient of friction f are no longer constants. Why? Because both C_v and f depend on V. It was also realised by Hotchkiss and Huang (1995) that the error due to concentration can be corrected for by replacing C_v by the sediment discharge rate Q_s since they are linked by

$$C_v = \frac{Q_s}{AV} \qquad (9)$$

A is the cross-sectional area of the pipe. However, Hotchkiss and Huang (1995) followed the same previous approach of Goedde (1978) and Eftekharzadeh (1987) but the resulting equation for Q_s, which in essence is correct, was very cumbersome to handle in design.

Regarding the error due to fixing of the coefficient of friction f, may be neglected at a high Reynolds number, when the turbulence is fully developed. In this range f varies with the relative roughness height (k_s/d) only. However, at transition turbulence f becomes dependent on both relative roughness height (k_s/d) as well as the Reynolds number (R_e = Vd/ν). Eftekharzadeh (1987) commented that the consideration of f as a variable makes the above mathematical development very cumbersome.
Keeping in mind that the main objective is to maximise the sediment discharge Q_s with the available water head or gradient J_m, it seems obvious and more conceivable to approach the solution through nullifying the derivative of the sediment discharge Q_s with respect to the velocity V. By considering this approach it was found that the variation of the coefficient of friction f can easily be accommodated (Siyam 2000). More rewardingly, the critical velocity of transport can be computed from a relationship similar to the well-known Darcy-Weisbach formula for clear water with the use of a modified coefficient of friction which depends only on the value of the Durand's parameter m.

3.1. The Critical Velocity and Optimum Sediment Transport for Fully Developed Turbulence Flow

In this section two equations for computing the critical velocity (V_c) and the sediment transport capacity (C_{vc}) will be derived for the case when the friction coefficient f is considered as a constant.
By substituting Eq. 9 into Eq. 5 we get

$$J_m = \frac{f}{2gd}V^2 + k\left[\frac{\sqrt{c_d}}{gd(s-1)}\right]^m \frac{f}{2gd}\frac{Q_s}{A}V^{(2m+1)} \qquad (10)$$

Letting

$$b_1 = \frac{f}{2gd} \quad \text{and} \quad b_2 = \frac{k}{A}\left[\frac{\sqrt{C_d}}{gd(s-1)}\right]^m \qquad (11, 12)$$

Rearranging Eq. 10; differentiating and letting $dQ_s/dv = 0$; and substituting for b_1 we get

$$J_m = \left(\frac{2m-1}{2m+1}\right) f \frac{V_c^2}{2gd} \qquad (13)$$

This is an important finding which implies that $J_m = \sigma.J$ or $f_m = \sigma.f$. Where f_m is a modified Darcy-Weisbach coefficient of friction which takes the presence of sediment into account, and σ is a new multiplication coefficient which takes into account the presence of sediment on pipe friction and it is found to be a function of Durand's parameter m as given by:

$$\sigma = \frac{(2m-1)}{(2m+1)} \qquad (14)$$

Equation 13 relates the water sediment mixture head loss gradient J_m to the clear head loss gradient J. This implies that the optimal transport velocity of the mixture can be determined by the normal use of clear water equations or charts.
Now by substituting Eq. 13 into Eq. 10 and reintroducing the optimum sediment concentration C_{vc} we get.

$$C_{vc} = -\frac{2}{k(2m+1)}\left[\frac{V_c^2\sqrt{C_d}}{gd(s-1)}\right]^{-m} \qquad (15)$$

If Durand's values for k and m are used then Eq. 13 and Eq. 15 will respectively become:

$$J_m = 2f\frac{V_c^2}{2gd} \qquad (16)$$

$$C_{vc} = \frac{1}{81}\left[\frac{V_c^2\sqrt{C_d}}{gd(s-1)}\right]^{1.5} \qquad (17)$$

To ease comparison with that of Eq. 8, Eq. 17 can be rewritten as to calculate the critical velocity V_c and not the capacity of transport.

$$Vc = 4.33\left[\frac{gd(s-1)}{\sqrt{c_d}}\right]^{\frac{1}{2}} C_{vc}^{\frac{1}{3}} \tag{18}$$

3.2. The Critical Velocity and Optimum Sediment Transport for Transition Turbulence Flow

With the previous approach of redefining the minimum head loss concept, it has been possible to take the variation of the coefficient of friction f into account when the flow is in the transition turbulence. Detailed derivation of sediment transport capacity and the critical velocity when the flow is in transition turbulence can be found in Siyam (2000). The modified Darcy-Weisbach for transition turbulence can also be written in a similar form as of **Eq. 13**

$$J_m = \left[\frac{(2m-1)f}{(2m+1)f + V_c f'}\right]. f\,\frac{V_c^2}{2gd} \tag{19}$$

Or

$$\sigma = \left[\frac{(2m-1)f}{(2m+1)f + V_c f'}\right] \tag{20}$$

At the same time the corresponding sediment transport capacity equation reads

$$C_{vc} = \frac{\delta}{k}\left[\frac{V_c^2\sqrt{C_d}}{gd(s-1)}\right]^{-m} \tag{21}$$

Where

$$\delta = \left[\frac{-2f - V_c f'}{(2m+1)f + V_c f'}\right] \tag{22}$$

4. DISCUSSION AND ASSESSMENT OF THE NEW EQUATIONS FOR OPTIMUM SIDEMENT TRANSPORT IN PIPES WITH NON-DEPOSITED BEDS

Simple comparison between Eq. 18 and Eq. 8 reveals that V_c is always greater than V_m by a factor of 1.26. This comes as a desirable effect and corrects for the previous underestimation of the critical velocity which is attributed to the wrong assumption of constant C_v. **Fig (2)** shows the computed Modified Froude Number obtained by rearranging Eq. 18 and plotted for various sediment sizes and different concentrations. The general trend compared very well with that of **Fig (1)** and the computed Modified Froude Number is always greater than the observed values.

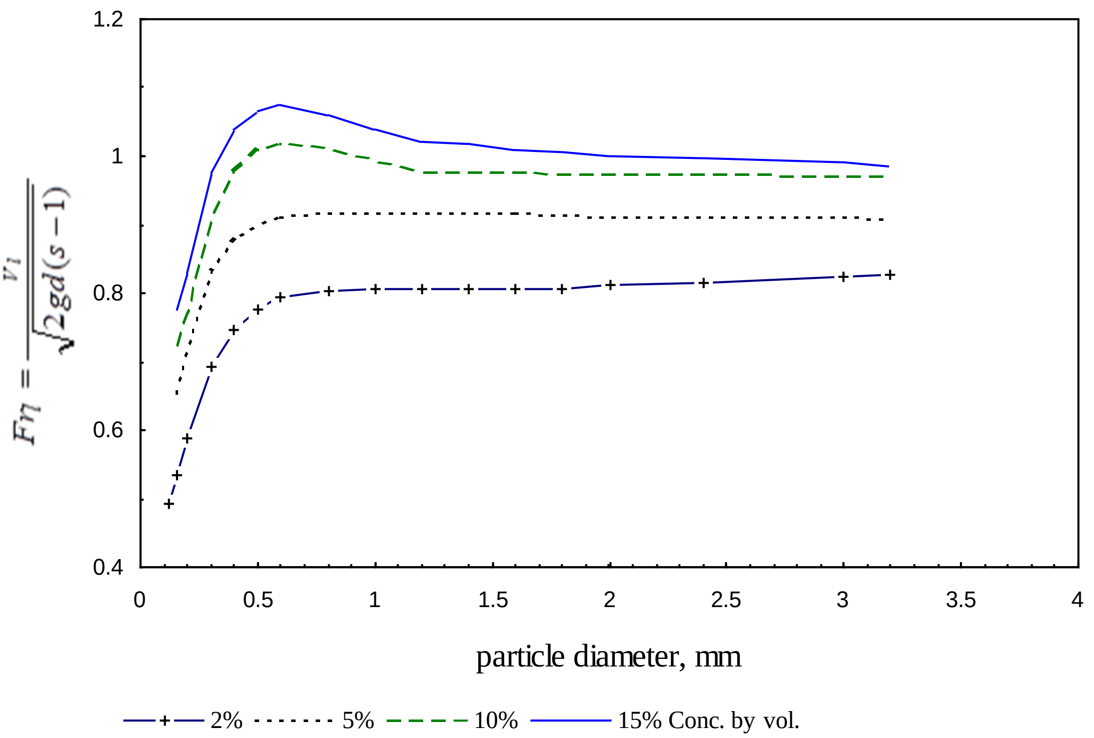

Fig (1): Durand's Number after Condolios and Chapaus (1963)

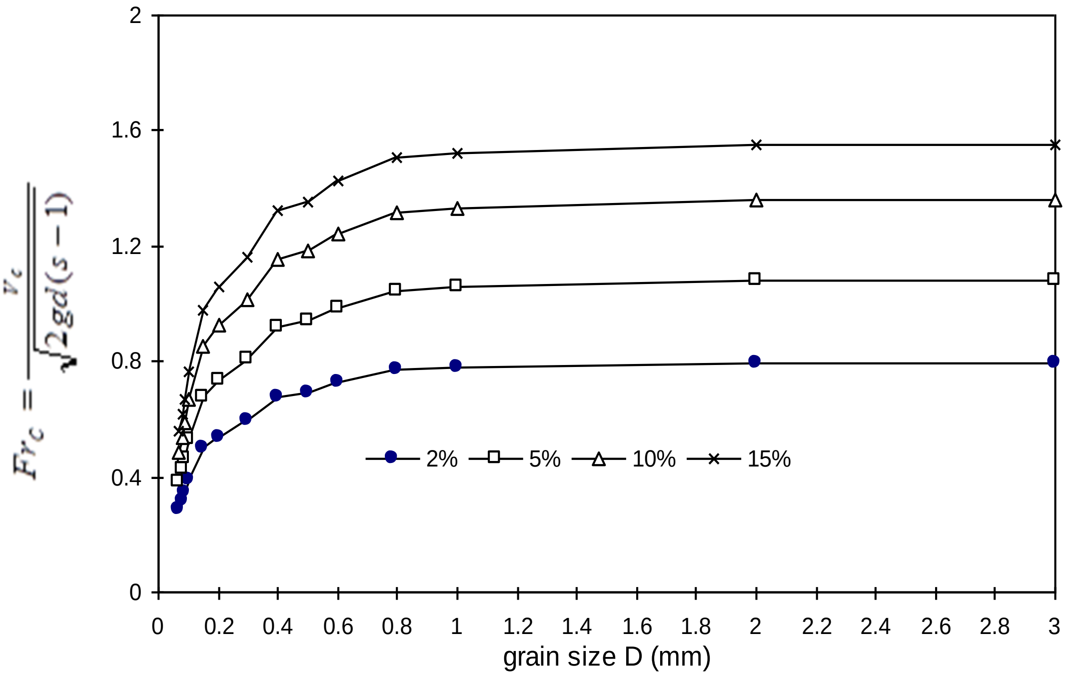

Fig (2): Computed Critical Durand Number Versus Sediment Grain Diameter with Concentration as Parameter

Fig (3) shows a typical head loss relationship presented by Durand. It needs mentioning that, when a stationary bed develops, the head loss curve at lower velocities was reported to fall again (Acaroglu and Graf 1969). The practical interest is in the velocity at which no bed deposits exist and, if different, the velocity at which the transport rate is optimum. At first it appears that the velocity which corresponds to the lowest point in the head loss curve may give an indication to the most favourable operating condition.

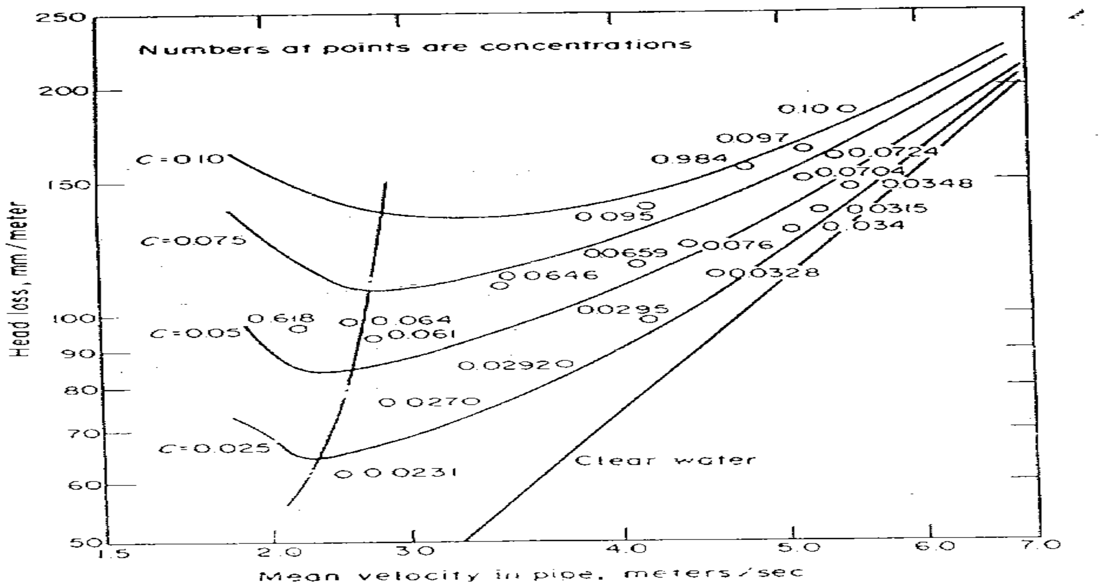

Fig (3): Effect of Concentration of Suspensions on Head Loss in Pipes. (Coarse sand D = 2.04 mm), after Durand (1953)

To check the validity of the above point, the head loss versus velocity curves for C_v = 2.5%, 5%, and 10% were generated by solving Durand's equation for pipe diameter = 0.5 m, and sediment size = 0.4 mm. The result is shown in Fig 4 together with the computed velocities V_m (Goedde 1978 form of optimum velocity, V_c (corrected optimum velocity), and the laboratory reported Limiting velocity V_L taken from Fig.1. For the three concentrations shown V_m is less than V_L and V_c, while V_c equals V_L at 2.5% and exceeds V_L at 5% and 10% C_v curves. On the other hand, it seems clearly that V_m marks the minimum head loss in all curves. This clarifies the previous researcher's remarks as derivation of Goedde (1978) was based on assumption of constant C_v.

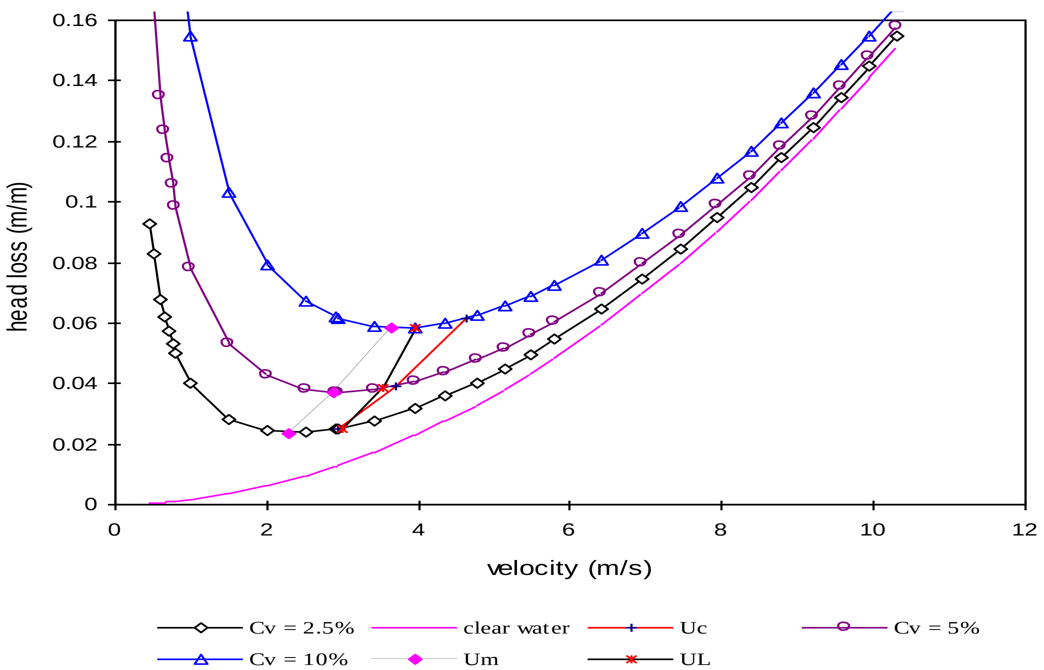

Fig (4): Head Loss Curves and Critical Velocities V_m, V_L, and V_c

Equation 19 is the general modified Darcy-Weisbach equation because it can be reduced to Eq. 13 when Darcy's coefficient f is considered as a constant. The same can be said for the multiplying

factor σ. The presence of the first derivative of the friction coefficient f' does not possess any problem since values of V and f are available from the previous step of iteration.
For the sediment transport capacity in transition turbulence, Eq. 21 also can be regarded as the general equation for determining the optimum sediment transport since it reduces to that of Equation 15 for the case when flow is turbulent. In quantifying the parameter δ the last iteration value of f' which is used for determining the critical velocity can be used.
As pointed out earlier, the observed limiting velocity indicates the minimum safe operation condition, but not necessarily the most economic operation condition. To make judgement from the economic point of view the sediment transport rate per unit head loss has been taken to serve as an economic index. In mathematical notation the term transport index Q_j is defined as:

$$Q_J = \frac{Q_s}{J_m} \tag{23}$$

In other words the transport index quantifies the volume of sediment transported per second per unit head available. In **Fig (5)** both head loss and transport index curves were drawn for the cases of constant C_v and constant Q_s. In the latter case, the maximum Q_j corresponds to the minimum head loss at the same velocity. This velocity was depicted by the modified minimum head loss concept as the critical velocity under which the operation condition is economically favourable since it maximises the transport rate of sediment per unit head loss gradient.

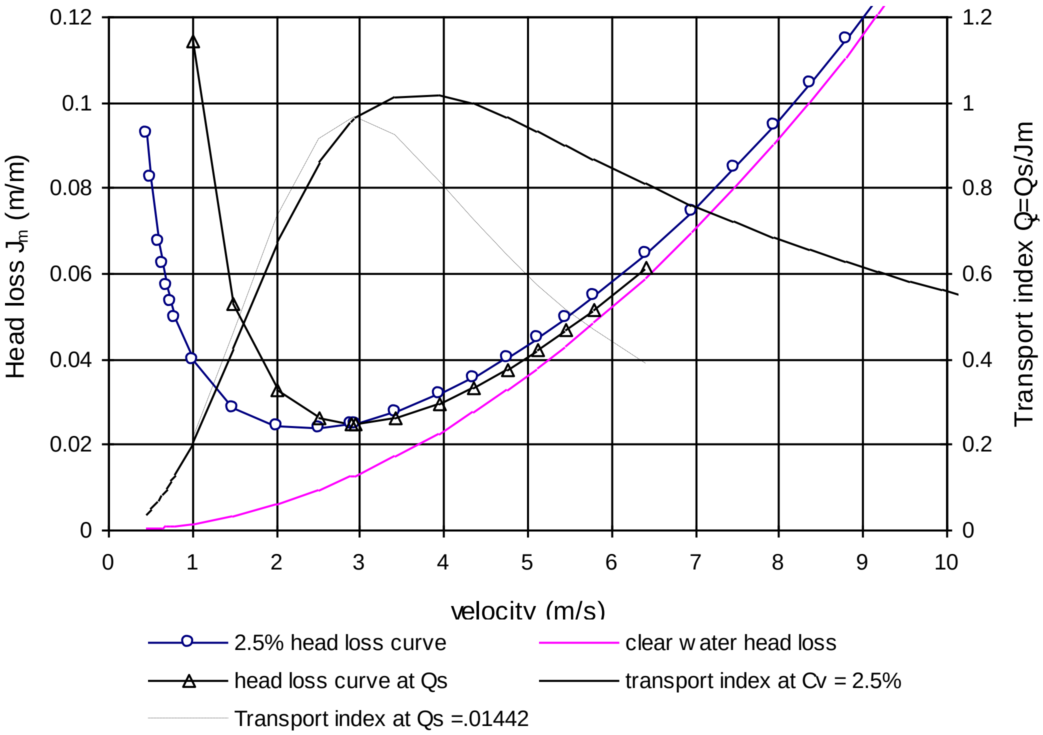

Fig (5): Head loss and transport index curves at constant C_v and Q_J

5. PROSPECT OF APPLICATION IN THE NILE BASIN

Currently hydropower generation in Roseires reservoir in Sudan is experiencing great difficulties from sediment ingress as well as occasional blockage of intakes due high rate of sedimentation. Annual removal of huge volumes of deposited silt by dredging is inevitable process. In a recent study conducted by UNESCO Chair in Water Resources of Omdurman Islamic University in Sudan and Development Research and Technological Planning Centre of Cairo University in Egypt (2015), sediment extraction using dredgers has also been recommended to preserve the capacity of Lake Nasser/Nubia at selected reaches. In such applications, the feasibility of the whole project will rely squarely on the optimal operation of the pipeline/pump system that maximizes the transport of sediment per unit of provided energy or head loss gradient.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions

1. In all previous application of the minimum head loss concept to Durand's equation, only one equation was presented where both the velocity and volumetric sediment concentration are linked to each other (Eq. 7 or Eq.8). Hence the iteration process is inevitable. The previously overlooked equation which would have eliminated this iteration was reproduced (Eq. 19).
2. The minimum head loss concept has been reformulated and applied in a new form that resulted in the desired effect and corrected for the previous underestimation of the optimum velocity of transport of sediment in pipes under non-deposited bed conditions.
3. In this paper two equations were formulated, namely Eq. 13 and Eq. 15 for turbulent flow regime and Eq. 19 and Eq. 21 for transition flow regime. In respective order, the earlier is the modified Darcy-Weisbach equation for evaluating the critical velocity of transport in pipes under non-deposited bed conditions. The latter is for determining the optimum sediment transport using the calculated critical velocity of transport.
4. This study has characterized the two parameters m and k of Durand (1953). The parameter m is turned out to reflect the effect of transported sediment on pipe friction, while the parameter k is the sediment transportability factor.
5. It was found that the critical velocity of transport that corresponds to the optimum sediment transport in pipes under non-deposited bed always returns a value higher than the observed limiting velocity.
6. To make the system operate economically, i.e. maximise sediment transport and minimise water discharge, it has been proved that via assessing the sediment transport index Q_j the critical velocity Vc, as given by the modified Darcy-Weisbach equation, is the most appropriate velocity of transport in pipes under non-deposited bed.
7. This research work paved the way for standardization of procedure for designing sediment transport in pipelines under non-deposited beds as it linked the steps of design to the well-known equations or charts for clear water flow in pipes.

6.2. Recommendations

1. Further research work is needed to determine the range of the two Durand's parameters m and k and to extend the use of equations derived here to sediment transport in pipes under limited deposit.
2. Further research work is recommended on area of designing mechanical devices that could agitate the deposited sediment in reservoirs to the required concentrations.
3. There is a need for development of standard procedure and design charts for design of pipelines that optimally transport sediment under all flow regime conditions.

7. SYMBOLS

A = Cross-section area of pipe.
Cv = Sediment Volumetric concentration.
d = Pipe diameter
g = Acceleration due to gravity
V = flow velocity
J = Clear water head loss gradient
J_m = water sediment mixture head loss gradient
m = Durand's Parameter
Q = clear water discharge
Q_j = Sediment transport Index.
δ = coefficient
C_d = sediment drag coefficient
D = grain size of sediment
Fr = Modified Froude Number
f' = first derivative of f
V_m = self-cleansing velocity
V_l = Limiting velocity
V_c = Optimum or critical velocity of flow
k = Durand's Parameter
Qs = Sediment flow discharge
σ = Multiplication factor
s = specific gravity of sediment.
f = Darcy-Weisbach coefficient of friction for clear water flow in pipe
f_m = Modified Darcy-Weisbach coefficient for water sediment mixture flow in pipe
C_{vc} = Optimum concentration of sediment transport.

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