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Sediment Transport Through Storm water Systems – Gaza City as Case Study

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Dedication

A special dedication is presented to those who conceived the mission of the holy land and presented their virtuous souls to achieve the Palestinian dream.....

To my beloved parents for their endless and generous support

Acknowledgments

Firstly all thanks for my God, Allah for his unlimited blessings and for giving me the strength to complete this study.

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List of Abbreviations

A	Cross sectional area
Ab	Cross sectional area of the deposited bed
Af	Cross sectional area occupied by the water
C_D	Drag coefficient
C_L	Lift coefficient
C	Volumetric sediment concentration
C_y	Volumetric sediment concentration at depth Y
D	Sediment particle diameter
D_{EB}	Equivalent diameter of the cross section occupied by the bed
D_{Ef}	Equivalent diameter of the cross section occupied by the water inside the pipe
D	Pipe diameter
F_D	Drag force
F_L	Lift force
g	Acceleration of gravity
hb	Depth of the deposited bed
i	Hydraulic gradient for flow with deposition
iw	Hydraulic gradient for flow without deposition
K_s	Pipe wall roughness
K_{ss}	Overall roughness parameter
L_s	Length of sediment inside the pipe
n	Manning 's roughness coefficient
P	The perimeter pf the pipe
P_b	Width of the bed interface
P_s	Perimeter of the pipe in contact with the bed
P_w	Perimeter of the pipe in contact with the flow
P*	The power
Q	Volume discharge rate of water
Q_s	Volume discharge rate of sediment
Q_{s*}	Sediment transport rate

R	Hydraulic radius
Re	Reynolds number
Re*	Particle shear Reynolds number = $d V^*/ \nu$
S	Specific gravity
SP	The shear parameter
TP	Transport parameter
V*	Friction velocity
V	Average flow velocity
V_{ss}	Velocity due to the use of K _{ss}
V_r	Threshold velocity
V_o	Velocity of flow at incipient motion
Y	Depth of water in the pipe
W	Settling velocity
f	Friction factor
f	Density of water
f_s	Density of sediment
τ_o	Shear stress

Abstract

This dissertation studies the effect of sediment concentration on hydraulic flow through stormwater sewer system for turbulent uniform flow.

Hydrodynamic forces that exist through storm sewers have been derived using theoretical model.

Experimental works have been done using experiment rig; constructed at the hydraulic laboratory.

The experiment has been designed to perform a uniform flow and high velocity flow

The analysis of the results for high-speed velocity flow through storm sewer gave the following design formula. $T = 0.0561 \times K^{3.54}$

According to proposed equation many relationships have been applied between grain diameter, velocity, diameter, concentration and hydraulic gradient.

Chapter 1

Introduction

1.1 Statement and significance of the problem:

In order to design a stormwater system there are many factors included in the design methods such as drainage area rainfall intensity and a coefficient which reflects the combined effects of surface storage, infiltration and evaporation. In addition of previous factors the sediments characteristics are one of the most important factors that affects the hydraulic flow in the stormwater system.

The sediment transport problem is not as simple as it appears in previous codes of minimum self cleansing velocity, because complete analysis contains a lot of hydraulic parameters as in the following table (1.1).

This study is mainly concerned with sediment transport through pipes, especially storm sewer. The first researcher in this field with a practical approach to solve sediment transport problem in pipes was Durand (1952). His approach is still in use but key researchers such as Condolios & Chapus (1963) and Gibert (1960) have introduced several modifications.

Sediment property	d50 *	Shape	Size distribution	density	
Liquid property	Dynamic viscosity	Density of water	Temperature		
Pipe property	Diameter	Pipe wall roughness Ks			
Flow property	Velocity	Concentration	gravity	Depth of the deposited bed	Hydraulic gradient

* Grain diameter at 50% passing

This study addresses the problem: ***What is the effect of sediment containment on stormwater sewer system?***

This could be solved through understanding the following points:

1. Understanding sediment distribution of contributing upstream disturbed lands;
2. Identifying the design-size particle;
3. Identifying and providing a design discharge value;

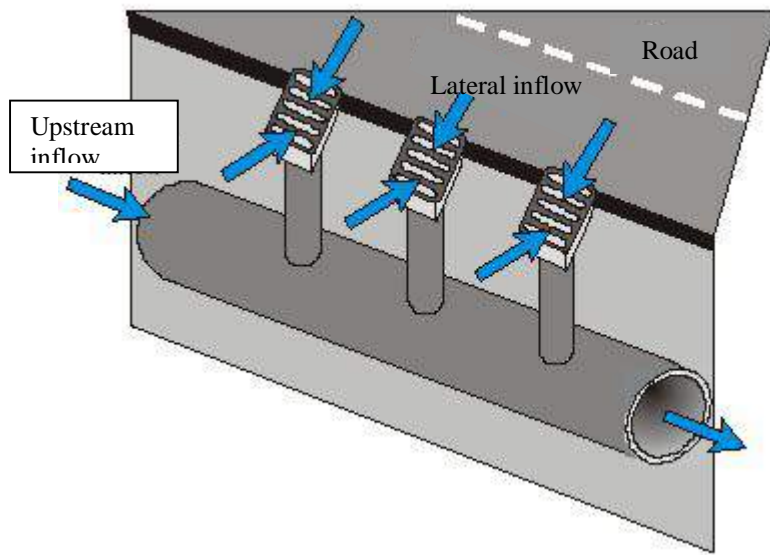


Fig (1.1) Flow of stormwater through sewer⁽¹³⁾

The advantages of the proposed approaches that are they allow consideration of the actual hydraulic conditions and sediment load in the system or part of the system; rather than application of a single self cleansing velocity criterion.

Thus greater flexibility can be achieved in the design approach particularly in systems where heavy sediment loads are encountered.

1.2 Background:

To solve the above problem, extensive investigations have been reported for the last half-century. The studies in sediments transport theories include Durand (1952), Robinson and Graf (1972), Spell (1955), Yufin and Lopasin(1968), Wiedenroth and Kirchner, Hughmark(1961), Sinclair (1962) , Charles(1970), May (1982), Dunard and Condolios (1952), Zandi and Govates(1967), Hayden and Stelson (1968), Condolios and Chapus (1963), Newitt et al . (1955), Rose and Duckworth (1969), Novak and Nalluri (1975), Kuhail (1989) and many others.

The conclusions of these researches are given later in literature review.

1.3 Objectives

The specific objectives addressed in this study are:

- 1) To know the effects of the sediments on the stormwater networks
- 2) To formulize an experimental, yet semi-theoretical method to design stormwater network which takes into consideration the effects of sediments.
- 3) To study the effects of sediments on the hydraulic flow of stormwater and try to formulate a mathematical model.
- 4) To Compare between the design methods of stormwater and the experimental method concluded in this research.

1.4 Limitations and assumptions

This study is limited to the effect of sediments through sewer and the study concentrate on turbulent high velocity flow which mean Reynold number > 4000

1.5 Outline

This dissertation includes eight chapters.

Chapter 1:

Briefly introduces the Statement and significance of the problem, specific objectives addressed in this study and the Limitations and assumptions for the study are indicated in this chapter.

Chapter 2:

Gives data basis for the Gaza city, climate of Gaza Strip, Sources of Stormwater Sediments, Describes the hydraulic factors that affect the flow.

Chapter 3:

This chapter Review previous major achievements of sediment transport through pipes. This survey was divided into two main parts:

1. Flow patterns; which describes the flow regimes in a solid – liquid Conveyance systems.
2. Transport theories.

Chapter 4:

This chapter discusses the Sediment transport in sewer, storm sewer junction hydraulics, sediment transport, the Physical Modeling for that and the self-cleansing velocities.

Chapter 5:

Theoretical Analysis was introduced in this chapter; movement processes of sediments within Storm Sewer, forces due to impact of liquid on particles and the Geometrical Relationships.

Chapter 6:

This chapter describes the sediment used in experiments and the experimental set-up layout of tanks, pipes, pumps, sediment feeder, and sediment trap.

Chapter 7:

Hydraulic parameters of the proposed method, the list of experimental results and Analysis of results. Summary of coefficient for the proposed method. Relationships of the proposed method.

Chapter 8:

Conclusions on this study included the results and proposed method obtained. Recommendations and further research required in the field of sediment transport in storm sewer.

Chapter 2

Sediments sources and characteristics for Gaza City

2.1 Introduction

This chapter gives an idea about the data basis for Gaza City, sources of stormwater sediments and its affect on the hydraulic load on the storm sewer.

In this study the climate effect in Gaza City has been investigated which the runoff flow is depended on the climate and weather conditions.

The climate of Gaza City is rainy in winter and dry in summer, this situation affects on design of storm sewer. The sediments on the surface deposits during dry whether periods on roofs and streets, sediments are transported to and from the area by wind and traffic depositing in sheltered locations with low wind velocities and less traffic, which approximately 50% of the solids ends up in the gutter.

At the initiation of the rain event the sediments deposited on the surface are wetted along with the surface itself. Already, before rainwater actually starts flowing towards the sewer inlet sediment particles on the surface are moved by the impact of individual rain drops. When runoff is established sediment particles may be transported with the flow. Potential sediment movement depends on rain intensity and particle characteristics.

In this study the investigations have been constrained for Gaza City as a case study which is considered an urban area since a lot of storm sewer projects have been constructed in the period which represents a comprehensive survey on the sediment found in gutter has been easily understood.

2.2 Data Basis for the Gaza City⁽¹⁹⁾

2.2.1 Location of Gaza Strip

The Gaza Strip is Located in the eastern coast of the Mediterranean Sea, between Egypt in the south, the occupied territories in the east and north, and Mediterranean Sea from the west.

The area of the strip is about 365 km², with a length of 45 km from the town of Rafah in the south to Beit-Hanun in the north, its width range between 6 to 12 km.

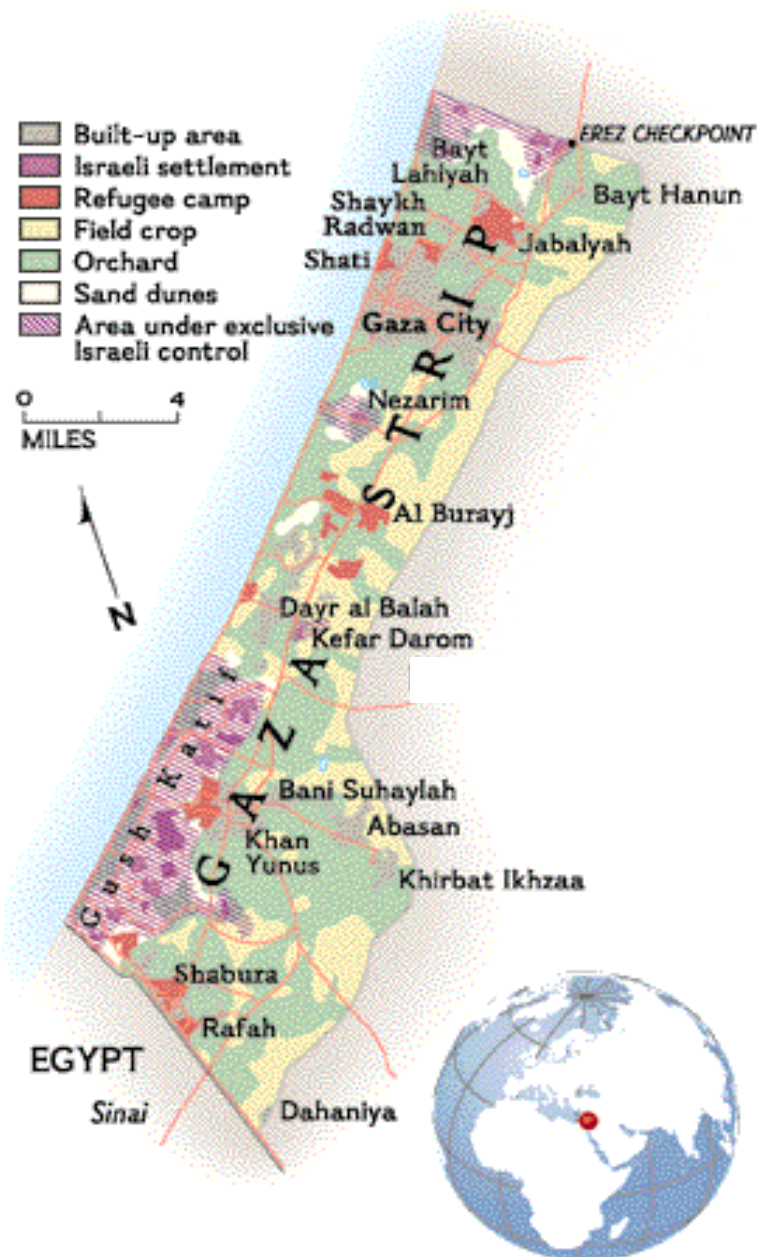


Fig (2.1) The Gaza Strip map⁽¹⁹⁾

2.2.2 Climate of Gaza Strip ⁽¹⁹⁾

The Gaza Strip is located in the transitional zone between the arid desert climate of the Sinai Peninsula and the temperate and the semi- humid Mediterranean climate along the coast. This fact could explain the sharp difference of more than 200 mm/year between Beit Lahia in the north and Rafah in the south in rainfall quantities.

The average daily mean temperature ranges from 25° C in summer to 13 ° C in winter. Average daily maximum temperatures range from 29° C to 17° C and minimum temperatures from 21° C to 9° C in summer and in winter respectively.

The average annual rainfall varies from 450mm/year in the north to 200mm/year in the south. Most of the rainfall occurs in the period from October to March, the rest of the year being completely dry. Regarding to the evaporation, maximum values in order of 140mm/month are quoted for summer, while relatively low evaporation values of around 70mm/month were measured during the months December to January. The daily relative humidity fluctuates between 65 % in the daytime and 85% at night in summer and between 60% and 80% respectively in winter.

This information is used for storm sewer design through many methods such as the rational method which is the simplest of the methods used for storm sewer design . the procedure of this method calculates the flow as the product of rainfall intensity , drainage area, and a coefficient which reflects the combined effects of surface storage, infiltration , and evaporation .

$$Q = CiA$$

Where,

i = rainfall intensity

A = Catchment Area

C = coefficient tend to increase as the rainfall continues

The data of rainfall intensity of Gaza (1981-1999) is shown in the Table (2.1) and Fig (2.2)

	1981	1982	1983	1984	1985	1986	1987	1988	1989	1990	1991
Jan	27.5	72	159	111	5	25	85.5	99	136	127	180
Feb	21	130	120	2.5	85	51.5	33	146.5	52	41.5	66.5
Mar	10	48	65.1	21.5	17.5	2.5	36.5	20	21	86.5	95
Apr	0	0	3.5	3.5	21	48.5	0	6	0	1	0
May	0	0	0	0	0	7	0	0	0	0	0
Oct	0	0	5.3	8.5	9	29	33.5	32	17.5	4	0
Nov	52	124.5	27	29.5	5	316.7	0	22	104	20	230
Dec	2	42.5	21	47.5	81	65.5	0	58.5	37.5	0.5	82.5

	1992	1993	1994	1995	1996	1997	1998	1999	sum	Average
Jan	165	121	81	8.8	138	130	90	99	1859.80	97.88
Feb	135	134	21.2	82	37.7	35.9	21.9	14.9	1232.1	64.85
Mar	15.5	25	52	13	35.7	35.7	19	4.5	624	32.84
Apr	0	0	0	21.5	5.4	0	0	4.5	114.9	6.05
May	8	0.5	0	0	0	0	0	0	15.5	0.82
Oct	0	38.5	16.1	0	40	1.9	7.5	13.5	256.3	13.49
Nov	32.5	14.5	260.4	38.5	1	4.7	2.9	15.6	1300.8	68.46
Dec	146.5	6	179.6	128.9	54.3	113.6	25.1	32.9	1125.4	59.23

Table (2.1) Rainfall pattern in Gaza City (mm)

The Source is from Palestinian Water Authority

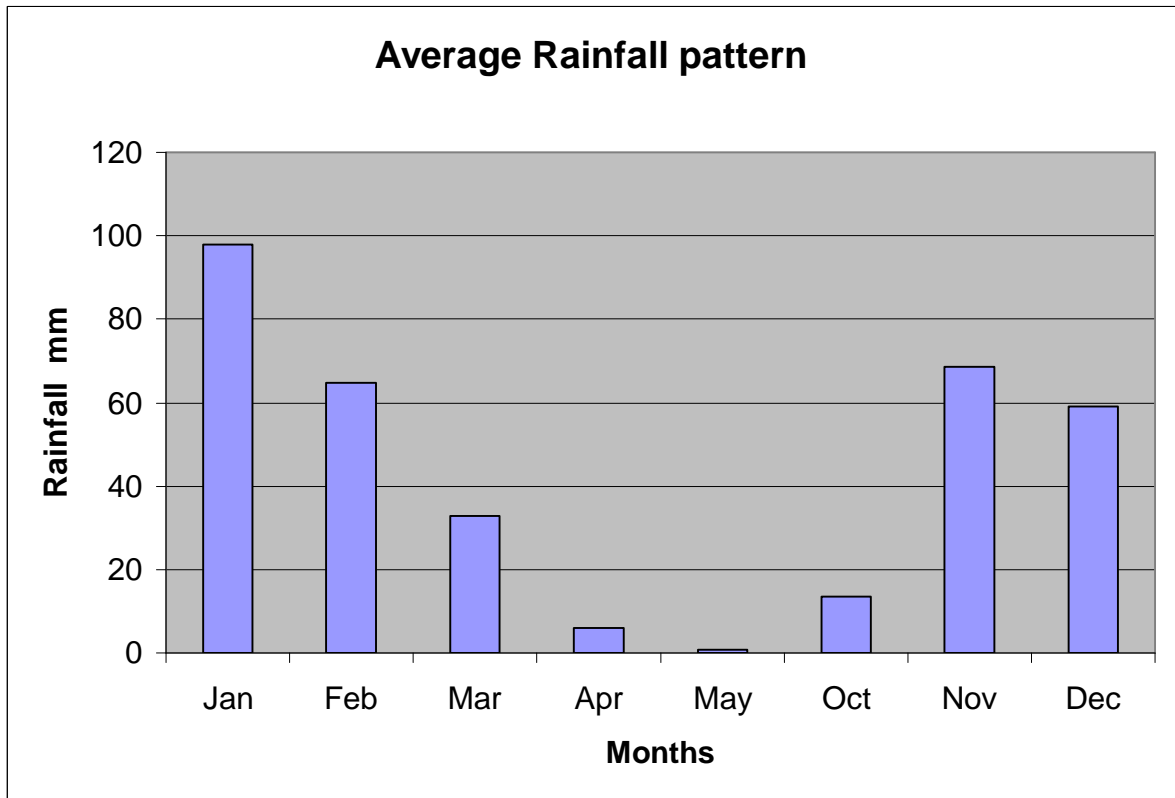


Fig (2.2) Average rainfall pattern for Gaza City (mm)

The Source is from Palestinian Water Authority

2.3 Sources of Stormwater Sediments ^(13,19)

A considerable amount of research effort has been undertaken attempting to identify and quantify sources of stormwater Sediments. This research has identified approximately seven major sources of common stormwater Sediments within storm sewer. Each of these sources is described briefly in the following sections. A number of these sources are illustrated in Fig (2.3)

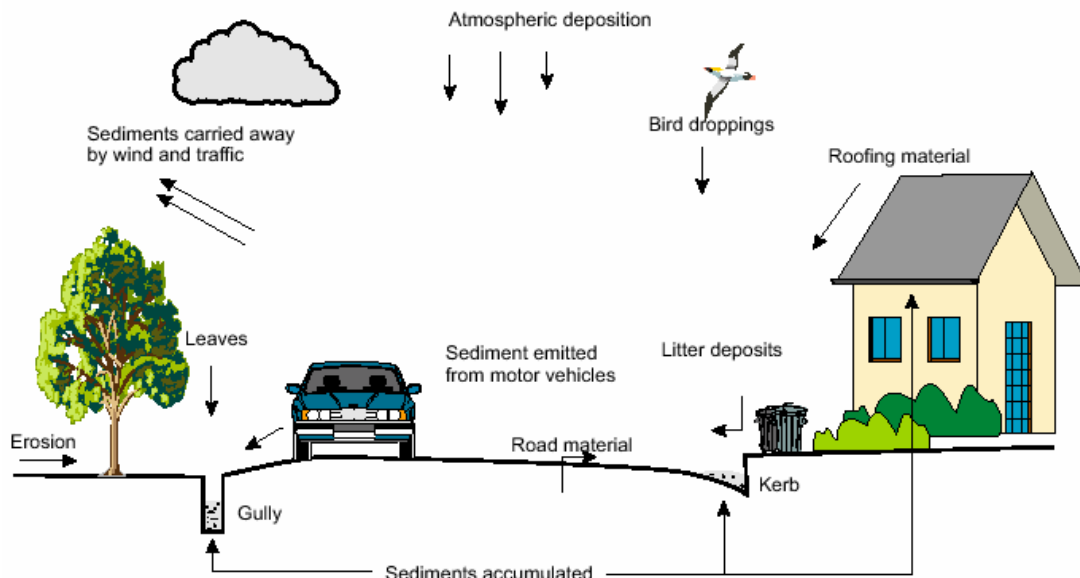


Fig (2.3) Sources of sediments on the catchment surface.

2.3.1 Street Pavement

Components of road surface degradation are common constituents of urban runoff. Studies have indicated that as much as 0.05-0.10 inch of pavement surface is worn away from the roadway each year ⁽¹²⁾. The largest component of street pavement is the aggregate material itself, with additional smaller quantities originating from the asphalt binder, fillers and substances applied to the surface. The amount of material originating from roadway surfaces is dependent upon the age and type of surface, the climate of the area, and the average daily traffic loading.

2.3.2 Motor Vehicles

Motor vehicles contribute a wide variety of materials to runoff flow. Common constituents generated by motor vehicles include fuels, lubricants, and particles from tires or brake lining, exhaust emissions which collect on the roadway surface, corrosion products, and larger broken parts which fall from vehicles during operation.

2.3.3 Vegetation

Waste vegetation matter is an important source of organic and nutrient pollutants in urban stormwater.

Organic matter such as leaves, grass, and other plant materials that fall or become deposited in urban areas may become part of stormwater runoff flows.

2.2.4 Land Surface

Land use within a drainage basin is a primary factor in determining the characteristics of stormwater runoff generated within that basin. The type of ground cover found in the drainage basin as well as the amount of vehicular and pedestrian traffic is a function of land use and will have a direct effect on the quality of stormwater runoff generated within that area.

2.3.5 Litter

Litter consists of various kinds of discarded material such as food containers, packaging material and animal droppings. Many types of man-made litter do not constitute significant sources of pollution, although litter is highly visible and can be aesthetically unpleasing when discharged into a receiving sewer. In some areas, animal droppings have been shown to be a major contributor of both nutrients and bacterial contamination in stormwater runoff.

2.3.6 Atmospheric Fallout

Atmospheric fallout originates as air pollution such as dust and particles from industrial processes, dust emissions from automobiles and planes, and from exposed land. A large portion of the atmospheric fallout settles on the land surface and becomes entrained into the runoff flow during storm events.

2.3.7 Construction Sites

Erosion of soil from land disturbed during construction activities is a highly visible source of suspended matter in stormwater runoff. Soil erosion is a major source of stormwater solids for both urban and suburban areas.

Fig (2.4) shows the surface sediment of sand on the road near the gutter.

Fig (2.5) shows the location and type of surface Catchment.



Fig. (2.4) Surface sediment of sand on the road near the gutter.

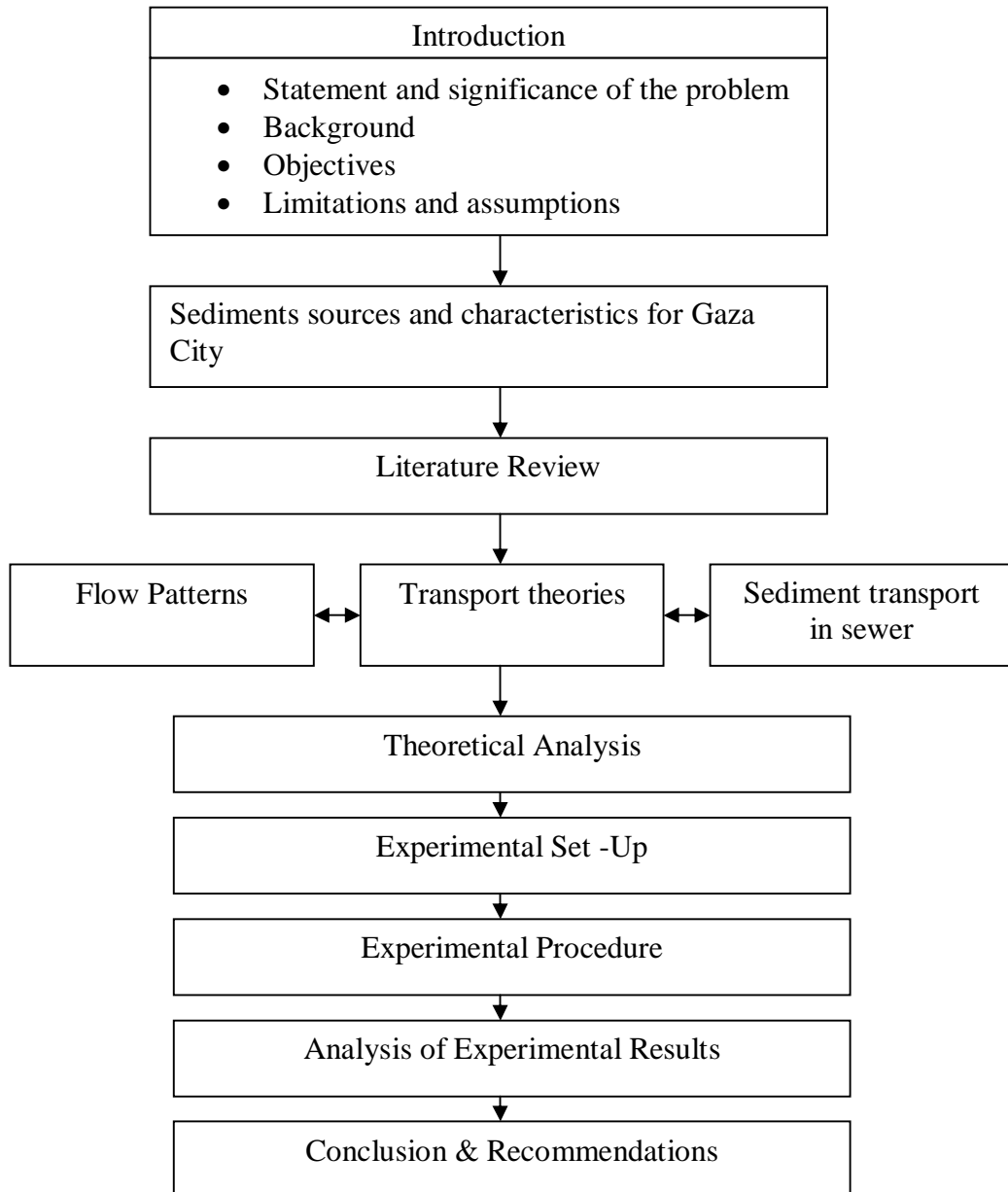


Fig (2.5) Catchment surface.

2.4 The sediments effects on the hydraulic flow ⁽⁷⁾

The most obvious effect of a sediment deposit in the invert of a pipe is the reduction of pipe area available for flow. In a pipe flowing full, unless the sediment was rapidly eroded, such a reduction would increase the flow velocity with corresponding increase in head loss. The other obvious consequence of a sediment bed is an increase in roughness as the sediment deposit is likely to have a greater roughness than the pipe. This further increases the head loss.

In a sewer flowing full, a sediment bed thus increase both the velocity and the level of turbulence assuming a sufficient head is available to support the greater resistance. Hence even with a sediment load near the transport capacity of the flow , a stable equilibrium can be maintained under steady flow conditions. If events such as a decrease in flow or an increase in sediment supply occur, the deposition of the sediments takes place but provided head is available in the system without flooding from the upstream manhole equilibrium will be reestablished. ⁽⁶⁾

Fig (2.6) Methodology Chart for the whole dissertation

Chapter 3

Literature Review (1,5,6,7,12,13,17,18)

3.1. Introduction.

This chapter Review previous major studies of sediment transport through pipes.

3.2 - Flow Patterns: (After Kuhail1989)

There are different patterns to define the flow of granular solids in liquid.

It can be defined as follows:

V1 = the velocity at and above which the mixture flows in the homogeneous pattern

V2 = the velocity at which the mixture flows in the heterogeneous (i.e. asymmetric suspension) but above the velocity **V3** at which solids form a deposit on the bottom of the pipe

V3 = the velocity at which a moving bed of particles exists on the bottom of the pipe. It is above the velocity **V4** which part of the bed in contact with the pipe wall becomes stationary.

V4 = the velocity at which a bed exists, the lower part of which is stationary whilst particles in the upper part move by saltation, but above the velocity at which pipe blockage occurs.

These transition velocities may be related to the pressure gradient –velocity curve as shown in the schematic Fig (3.1). Above **V1** the suspension behaves very like true fluid.

The concept of the transition velocities **V1** to **V4** is only applicable to suspensions of closely sized particles.

From the previous definitions, it is well established that the velocity **V2** coincides with the minimum point that occurs in the “ pressure gradient- mixture velocity “relationship. The velocity **V3** is not clearly reflected in the pressure gradient curve and **V4** is seldom reached in actual measurements.

Fig (3.2) shows the flow regimes for given fluid, sediment and pipe size qualitatively only.

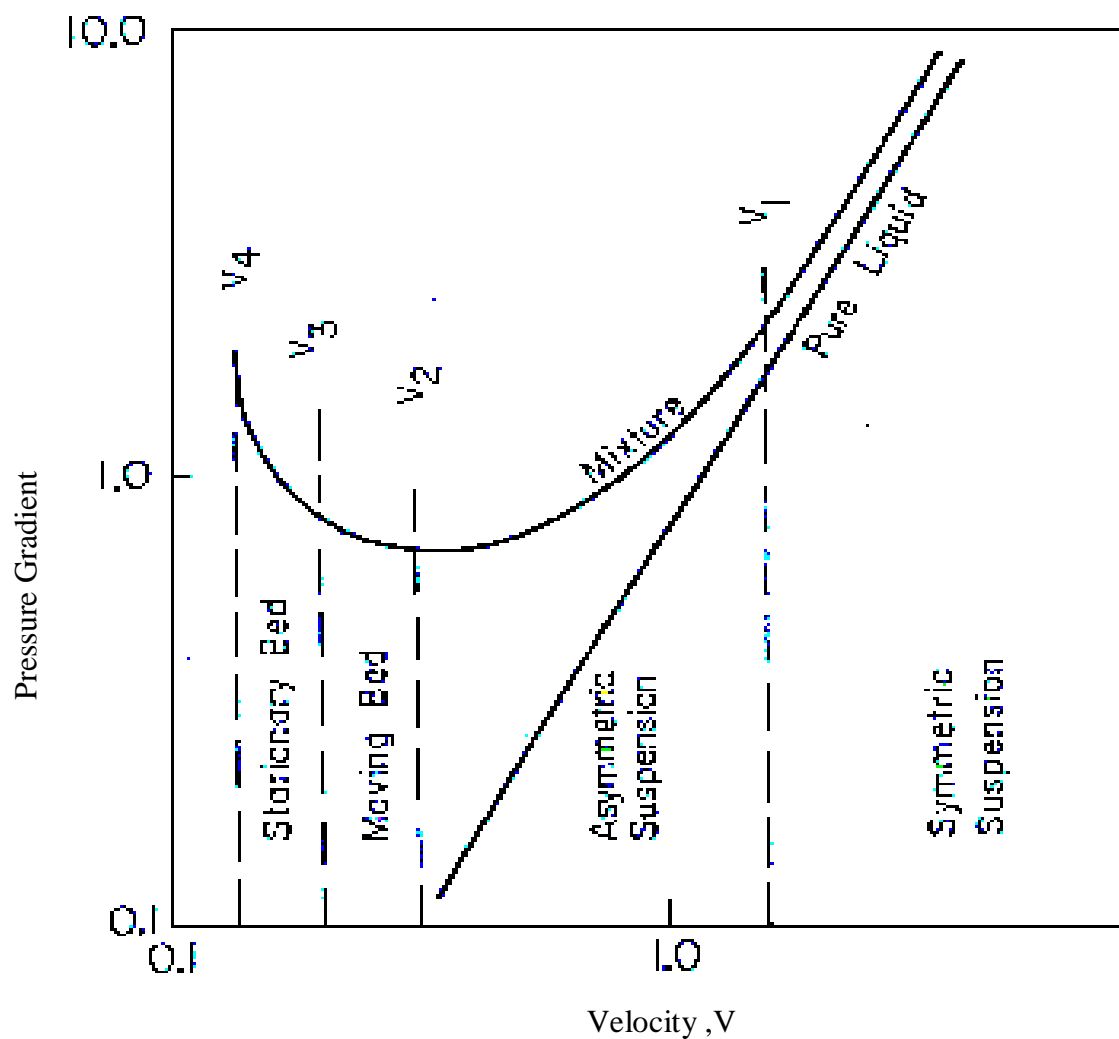


Fig (3.1) Schematic representation of pressure gradient curves

(After Aziz et al 1972)

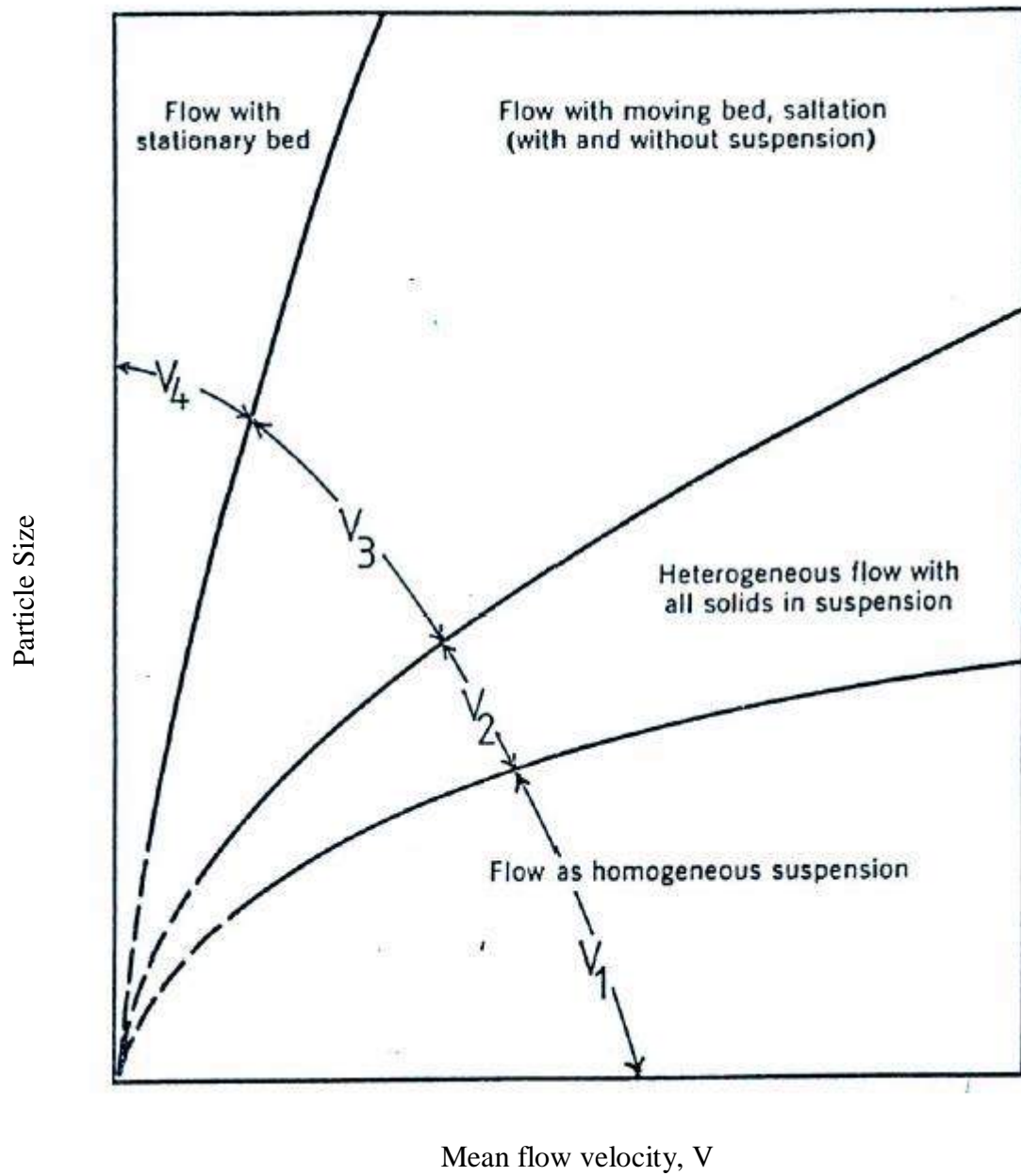


Fig (3.2) Flow regimes for given fluid, sediment and pipe size, qualitatively only
(After ASCE 1975)

3.3- Transport theories:

The movement of the sediment in pipes can be explained in two ways:

- 1- With velocity equations ; considering the impact of liquid on the particles
- 2- With shear equations or hydraulic gradient equations; considering the fractional drag of the flow on the particles.

In this part of the literature survey, the theories and approaches for the heterogeneous flow and the moving and stationary bed flow will be dealt with. In each case the theories will be divided into two parts;

- 1- Due to velocity.
- 2- Due to shear stress or hydraulic gradient.

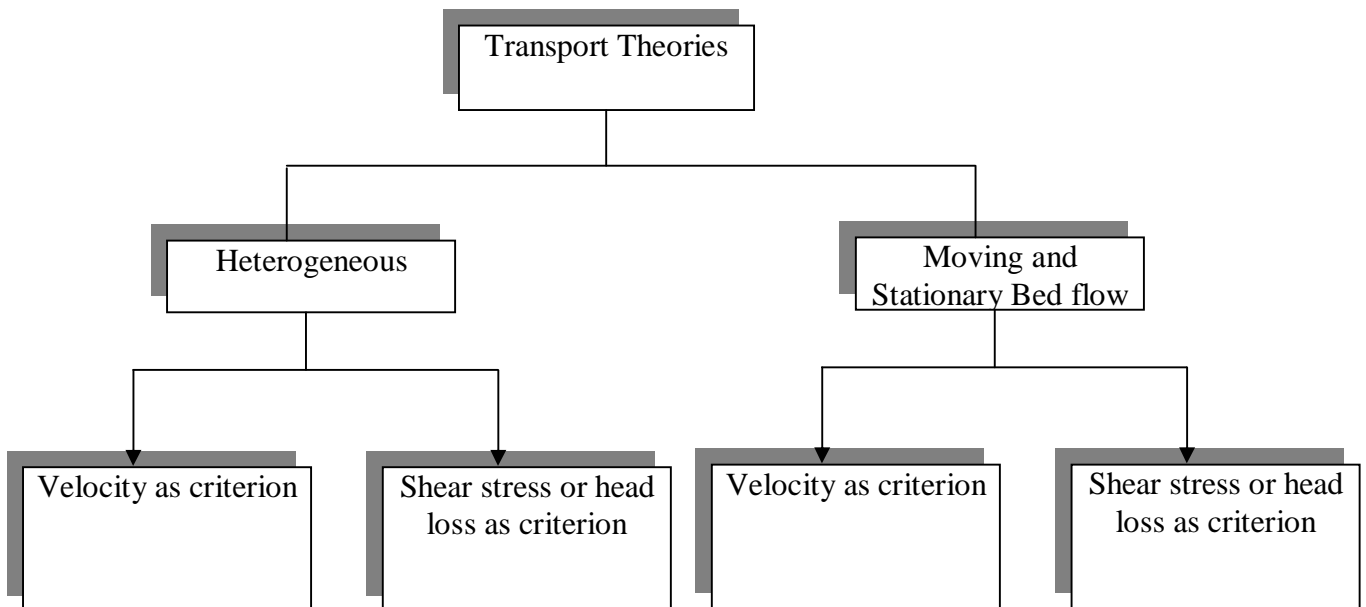


Fig (3.3) sediment transport theories in pipes

Table (3.1) Transport theories for Heterogeneous Velocity as criterion
As sited in (Kuhail 1989) with modifications.

Transport Theories		
3.2.1 Heterogeneous		
3.2.1.1 Velocity as criterion		
1	Durand (1953)	
	$V_2 = Fr[2gD(s-1)^{1/2}]$	<p><i>Durand (1953)</i> was the first to attempt to define the conditions for liquid – Solid systems beyond which a deposit on the pipe invert would be encountered. For the “ limit deposit velocity V_2 he proposed the empirical relationship $V_2 = Fr[2gD(s-1)^{1/2}]$ where Fr is a dimensionless factor influenced by particle size and concentration as given in Figure(3.4). Durand’s correlation was based on data covering pipe diameter from 40mm to 700mm for sand and coal particles with concentrations to about 15% by volume.</p>
2	Robinson and Graf (1972)	
	$Fr = \frac{0.928C^{0.105}d^{0.056}}{(1 - \tan\phi)}$ $V_2 = 17V_o$ $V_2 = 17\left[\frac{4gd(s-1)}{3C_D}\right]^{1/2}$	<p><i>Robinson and Graf (1972)</i> determine the limit of deposition at concentrations between 0.1% and 7%. Where ϕ =the angle of the pipe to the horizontal (+ve for an upwords-sloping pipes)</p>
3	Newitt et al (1955)	
	$V_2 = 17V_o$ $V_2 = 17\left[\frac{4gd(s-1)}{3C_D}\right]^{1/2}$	<p>The semitheoretical analysis of Newitt et al (1955) resulted in the following equation for V_2 ;</p> $V_2 = 17V_o$ <p>where V_o = is the velocity at which the drag forces just balances the gravitational force in turbulent flow</p>

4	Spell (1955)	
	$V_2^2 = 0.0251 \left[\frac{DV_2}{\nu} \right]^{0.775} gd(s-1)$ $V_2 = 54.4C_D^{0.815} D^{0.633} V_o^{1.63}$	<p>Spell (1955) using an empirical approach based on dimensional analysis presented a equation for the minimum velocity which is also equivalent to V2</p> $V_2^2 = 0.0251 \left[\frac{DV_2}{\nu} \right]^{0.775} gd(s-1)$ <p>which may be arranged and combined with Newton's equation for Vo to give</p> $V_2 = 54.4C_D^{0.815} D^{0.633} V_o^{1.63}$
5	Yufin and Lopasin	
	$V_2 = 8.3D^{1/3} C^{-1/24}$	<p><i>Yufin and Lopasin</i> have derived the following equation for sand and gravel particles of diameter greater than 0.05mm.</p> $V_2 = 8.3D^{1/3} C^{-1/24}$
6	Wiedenroth and Kirchner	
	$V_2 = 0.6(gD)^{0.5} [V_o / gd]^{0.25}$	<p><i>Wiedenroth and Kirchner</i> deduced a relationship between the Froude numbers for the pipe flow and the particles from which the following equation for V2 was derived;</p> $V_2 = 0.6(gD)^{0.5} [V_o / gd]^{0.25}$
7	Hughmark(1961)	
	$\frac{V_2}{\sqrt{gD}} = f[C(s-1)F_{rD}]$	<p><i>Hughmark(1961)</i> has Proposed a correlation for V2 in the form $\frac{V_2}{\sqrt{gD}} = f[C(s-1)F_{rD}]$ where F_{rD} is a correlating factor given by Figure (3.5)</p>
8	Zandi and Govates (1967)	
	$N1 = \frac{V_2^2 \sqrt{C_D}}{cgD(s-1)}$ $V_2 = \frac{40CgD(s-1)}{\sqrt{C_D}}$	<p><i>Zandi and Govates (1967)</i> Proposed a flow pattern index</p> <p>After appraising some 1452 data points. $N1 = \frac{V_2^2 \sqrt{C_D}}{cgD(s-1)}$</p> <p>and indicated that when $N1 \leq 40$ the flow would be by saltation. This is equivalent to $V_2 = \frac{40CgD(s-1)}{\sqrt{C_D}}$</p> <p>It would appear that N1 is good criterion of V2 but that other factors are required to accommodate a variety of particle sizes.</p>

9	Sinclair(1962)	
	$\frac{(V_2)_{\max}^2}{gd_{85}(s-1)^{0.80}} = 650$	<p><i>Sinclair (1962)</i> has carried out what is the most comprehensive experimental investigation of the limit deposit velocity, V_2 for liquid –solid mixtures in small pipes (12.5, 18.8 and 25mm). he tested closely sized coal <i>Sinclair</i> elected to correlate $(V_2)_{\max}$ in terms of a modified Froude number and the diameter ratio d_{85}/D where d_{85} is the particle diameter such that 85% by weight of the particles are less than d_{85} . His correlation is given in Figure (3.6) and Figure (3.7). <i>Sinclair</i> shows that for d_{85}/D less than 0.001, the limit velocity becomes independent of d/D. His equation is</p> $\frac{(V_2)_{\max}^2}{gd_{85}(s-1)^{0.80}} = 650$
10	Charles(1970)	
	$V_2 = \frac{4.8C^{1/3}[gD(s-1)]^{0.5}}{C_D^{0.25}(1+C(s-1))^{1/3}}$	<p><i>Charles (1970)</i> Proposed pressure gradient equation</p> $V_2 = \frac{4.8C^{1/3}[gD(s-1)]^{0.5}}{C_D^{0.25}(1+C(s-1))^{1/3}}$ <p>This equation has been recommended as a guideline for critical velocity prediction. For non-uniform sediment, C_D is recommended to be evaluated corresponding to the largest particle present in the size distribution.</p>
11	May (1982)	
	$Vr = 0.61(d/R)^{-0.27}\sqrt{gd(s-1)}$ $C = 0.0205\left[\frac{D}{A}\right]^2\left[\frac{d}{R}\right]^{0.6}\left[\frac{V_2^2}{gD(s-1)}\right]^{3/2}\left[1 - \frac{Vr}{V_2}\right]^4$	<p><i>May (1982)</i> reviewed previous researches and theories on pipeline transport and developed several new concepts. May's theory of pipeline transport with little or no deposition is based on the hydrodynamics of particle movement and dimensional analysis. the final equation relating the limiting velocity for deposition V_2 To the transport rate expressed as Volumetric concentration "C"</p> $C = 0.0205\left[\frac{D}{A}\right]^2\left[\frac{d}{R}\right]^{0.6}\left[\frac{V_2^2}{gD(s-1)}\right]^{3/2}\left[1 - \frac{Vr}{V_2}\right]^4$

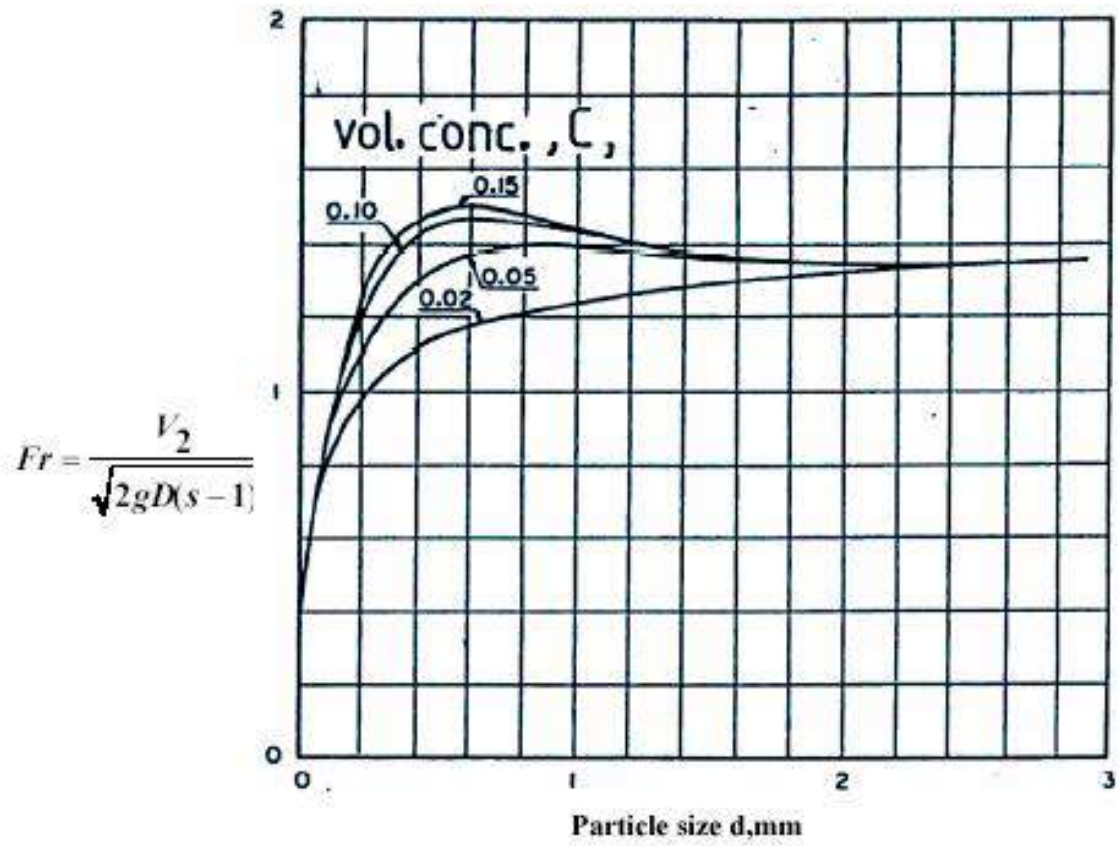


Fig (3.4) Durand's correlation for minimum transport velocity
(After Durand 1953)

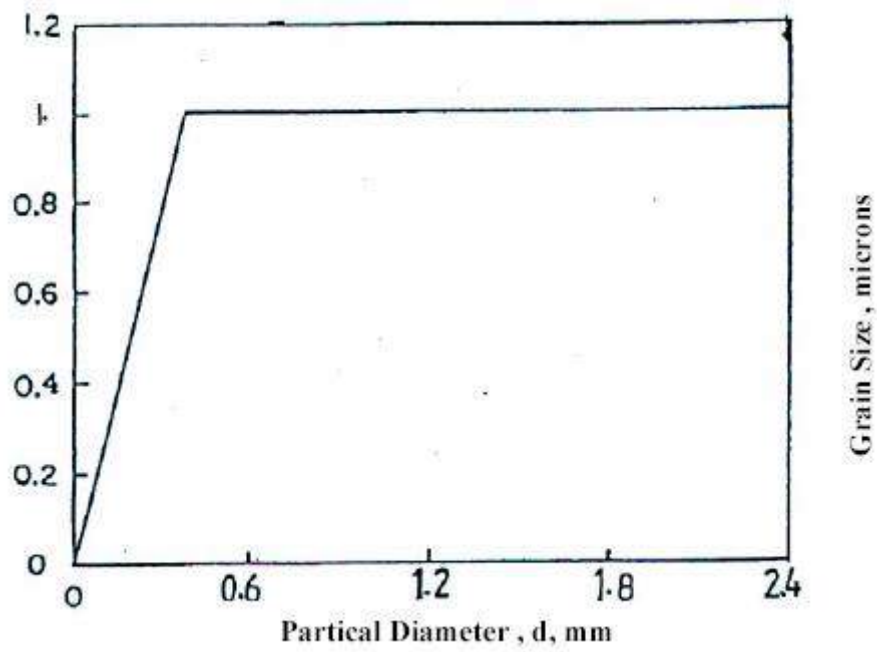
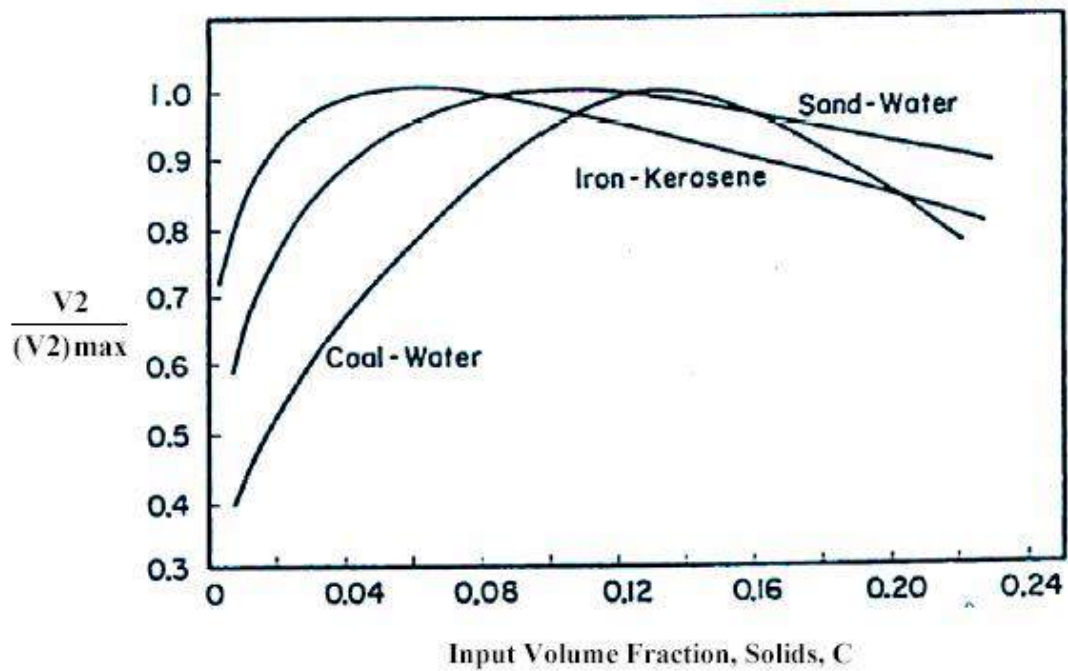


Fig (3.5) Hughmark's correlation Factor, FrD

Fig (3.6) Minimum Transport Velocity Relationship
(After SINCLAIR 1962)

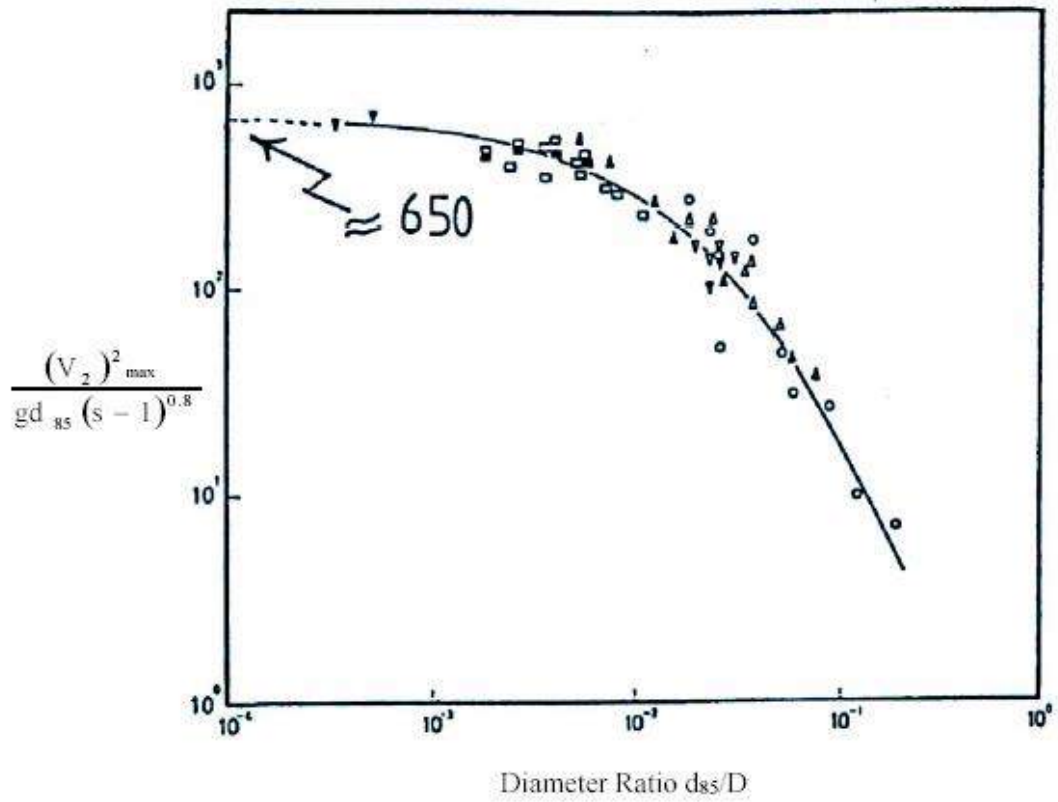


Fig (3.7) Minimum Transport Velocity Correlation
(After SINCLAIR 1962)

Table (3.2) Transport theories for Heterogeneous Shear stress or head loss as criterion

As sited in (Kuhail 1989) with modifications

3.2.1.2 Shear stress or head loss as criterion	
12	Dunard and Condolios (1952)
$f^* = K_D \left[\frac{V}{\sqrt{gD}} \right]^3 \left[\frac{1}{C_D} \right]^{3/2}$ <p>where</p> $C_D = \frac{4gd(s-1)}{3w^2}$ <p>and $K_D = 176$</p> $f^* = K_D \left[\frac{V}{\sqrt{gD}} \right]^3 \left[\frac{\sqrt{gd(s-1)}}{w^2} \right]^{-3/2}$	<p><i>Dunard and Condolios (1952)</i> have been carried out extensive tests with sands, gravel and coal ranging in particle size from 0.2mm to 25mm in pipelines with D=40mm To 580mm and C from 2 to 23%.</p> <p>Fig (3.8) is one of their plots of experimental data. They concluded that the data is described by</p> $f^* = K_D \left[\frac{V}{\sqrt{gD}} \right]^3 \left[\frac{1}{C_D} \right]^{3/2}$
13	Zandi and Govates(1967)
$N1 = \frac{V2^2 \sqrt{C_D}}{CgD(s-1)}$	<p><i>Zandi and Govates(1967)</i> collected and correlated most of the available data (2549 data points) for horizontal flow of water-solid systems, excluding those of which they thought a moving bed had been presented and those of concentration less than 5%, to develop two equations to describe the remaining data their exclusion of suspected moving bed data was based on their criterion for V2</p> $N1 = \frac{V2^2 \sqrt{C_D}}{CgD(s-1)}$
14	Hayden and Stelson (1968)
$\frac{i - i_w}{C * i_w} = 100 \left[\frac{gD(s-1) Vr}{V_2^2 \sqrt{gd(s-1)}} \right]^{1.3}$	<p><i>Hayden and Stelson (1968)</i> Proposed modification of Durand's equation to the form</p> $\frac{i - i_w}{C * i_w} = 100 \left[\frac{gD(s-1) Vr}{V_2^2 \sqrt{gd(s-1)}} \right]^{1.3}$ <p>They supported this form with their data on fine and coarse gravel in 25 and 50mm pipes. See Fig(3.9)</p>

15	Condolios and Chapus (1963)	<p><i>Condolios and Chapus (1963)</i> Reported that the Durand and Condolios correlation applies to materials having a wide range of particle sizes provided that a weighted drag coefficient is used. They define such a coefficient as</p> $\sqrt{C_{DM}} = \sum m_n \sqrt{C_{DN}}$ <p>where m_n = mass fraction of solid having a drag coefficient “C_{Dn}”</p>
16	Newitt et al. (1955)	<p><i>Newitt et al. (1955)</i> has correlation based on their own data on the flow in a 25 mm pipe of various sizes, fine coal, and sands. All traveling in water suspension with the transition velocity V_2. they obtained;</p> $\frac{i - i_w}{C * i_w} = B_N \frac{gDV_0(s-1)}{V_2^3}$ $\frac{i - i_w}{C * i_w} = 1100 \frac{gDV_0(s-1)}{V_2^3}$ <p>based on data taken only for small pipe</p> $d_m = \sum_n d_n m_n$ <p>The final equation is;</p> $\frac{i - i_w}{C * i_w} = 1100 \frac{gDV_0(s-1)}{V_2^3}$ <p>This equation was based on data taken only for small pipes.</p> <p>Newitt et al Observed that where the solids cover a range of particle sizes, the finer particles tend to increase the carrying capacity of the mixture for larger particles. They used the weighted parameter in their correlation;</p> $d_m = \sum_n d_n m_n$ <p>m_n = mass fraction of solids of diameter d_n.</p>
17	Rose and Duckworth (1969)	<p><i>Rose and Duckworth (1969)</i> in connection with their detailed study on the hydraulic gradient for the flow of solids suspended in liquids presented an empirical relationship for the minimum transport velocity. they define velocity as that ‘ at which the particles settle out of suspension and thereby tend to block the pipe’ which can be interpreted as “V_2” their equation is</p> $V_2 = 3.2V_0 \operatorname{Re} \left[\frac{D}{d} \right]^{0.6} i^{-0.7} \left[\frac{V_2^2}{gD} \right]^{0.25}$ $V_2 = 13.6 \left[\frac{gd(s-1)}{C_D} \right] \left[\frac{1}{gD} \right]^{1/2} i^{-1.7} \operatorname{Re}^{0.4} \left[\frac{D}{d} \right]^{1.2}$ <p>Which , if we substitute for V_0 from Newton’s law and rearrange the equation lead to ;</p> $V_2 = 13.6 \left[\frac{gd(s-1)}{C_D} \right] \left[\frac{1}{gD} \right]^{1/2} i^{-1.7} \operatorname{Re}^{0.4} \left[\frac{D}{d} \right]^{1.2}$

18	Novak and Nalluri (1975)	
	$f = 11.6y^{-2.04}$ $f = \frac{CV_2R}{\sqrt{gd^3(s-1)}}$ $y = \frac{(s-1)d}{iR}$ $i = (s-1) \left[\frac{d}{R} \right]^{1/3} \left[\frac{CR^{1/6}}{11.6n\sqrt{g}} \right]$ $V_2 = \frac{(s-1)^{3/2} CR^{5/3} d^{1/2}}{11.6n^4 \sqrt{g}}$ $\frac{V_2}{[8gR(s-1)]^{1/2}} = 0.632 f^{-0.662} C^{0.325} (d/R)^{0.175}$	<p><i>Novak and Nalluri (1975)</i> used their experimentally determined transport function for no deposition;</p> $f = 11.6y^{-2.04}$ <p>Novak and Nalluri explained combining with Manning equation how direct solution was obtainable for no deposition case;</p> $i = (s-1) \left[\frac{d}{R} \right]^{1/3} \left[\frac{CR^{1/6}}{11.6n\sqrt{g}} \right]$ <p>it can be shown from Novak et al transport function that ;</p> $V_2 = \frac{(s-1)^{3/2} CR^{5/3} d^{1/2}}{11.6n^4 \sqrt{g}}$ <p>Novak and Nalluri rearranged the previous equation to get ;</p> $\frac{V_2}{[8gR(s-1)]^{1/2}} = 0.632 f^{-0.662} C^{0.325} (d/R)^{0.175}$
19	Bonapace (1981)	
	$i \log_{10} \left[\frac{15d_m}{Ks + 6.5v(gDi)^{-1/2}} \right] = 0.0488(s-1) \left[\frac{d_m}{d} \right]$	<p><i>Bonapace (1981)</i> analyzed theoretically the lift force acting on individual particles at the threshold of movement by incorporating the Colebrook-White resistance equation .</p> <p>The following equation was obtained for the energy gradient “i” of the flow needed to produce movement of particles in the pipe flowing full;</p> $i \log_{10} \left[\frac{15d_m}{Ks + 6.5v(gDi)^{-1/2}} \right] = 0.0488(s-1) \left[\frac{d_m}{d} \right]$ <p>where dm is the maximum particle size</p>
20	Macke (1983)	
	$Q_s^* = Q_s g(s-1)w^{3/2}$ <p>For $Q_s^* \geq 2.0 \times 10^{-4}$</p> $Q_s^* = 1.64 \times 10^{-4} t_o^3$ <p>Macke recommended a safety factor of</p> $t_o = 1N/m^2 \text{ for the region } Q_s^* < 2.0 \times 10^{-4}$ $1.64 \times 10^{-4} t_o^3 = Q_s f g(s-1)w^{1.5} \text{ for } Q_s^* < 2.0 \times 10^{-4}$ $V_2 = 1.98 f^{-0.6} w^{0.3} [(s-1)AC]^{0.2}$ <p>where Q_s^* is the sediment transport rate</p>	<p><i>Macke (1983)</i> introduced a theory, which converts the energy in resisting the frictional force to turbulent fluctuations in the flow. His experiments were carried out to determine the limit of deposition for fine and medium sand in pipes flowing full and part full. The theory assumes the turbulent maintains the sediment particle in suspension. The data used to obtain a best-fit equation is taken Macke’s own experimental data in pipes, flowing full and part full.</p>

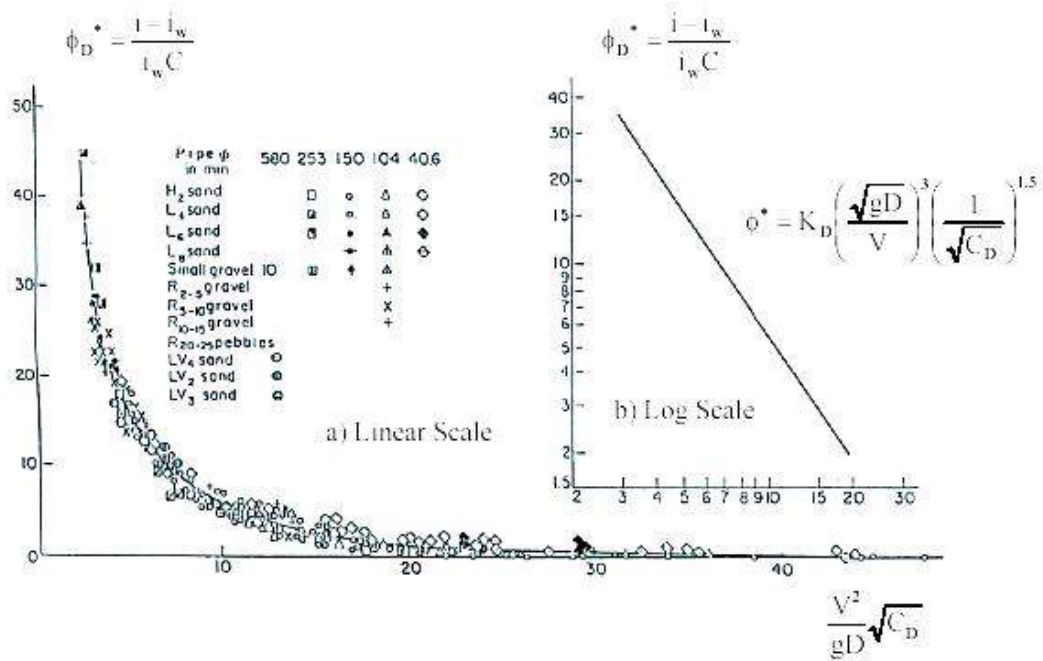


Fig (3.8) Head losses in pipes with non-deposit flow regimes
(After DURAND 1952)

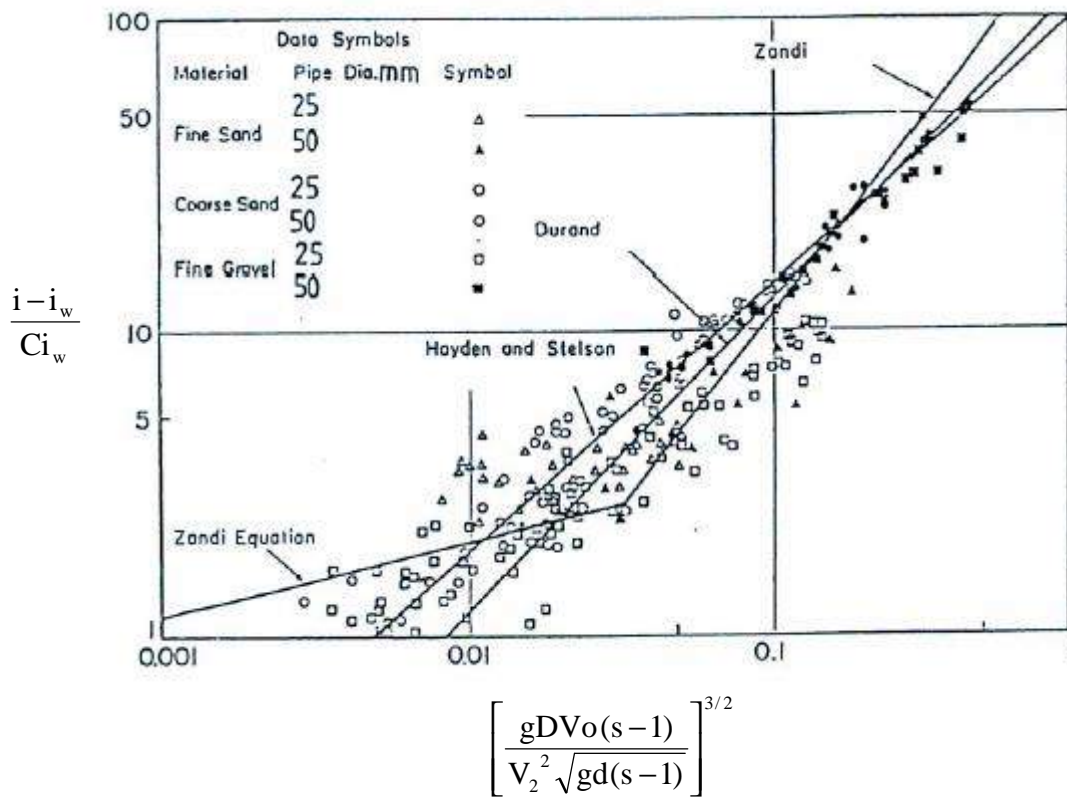
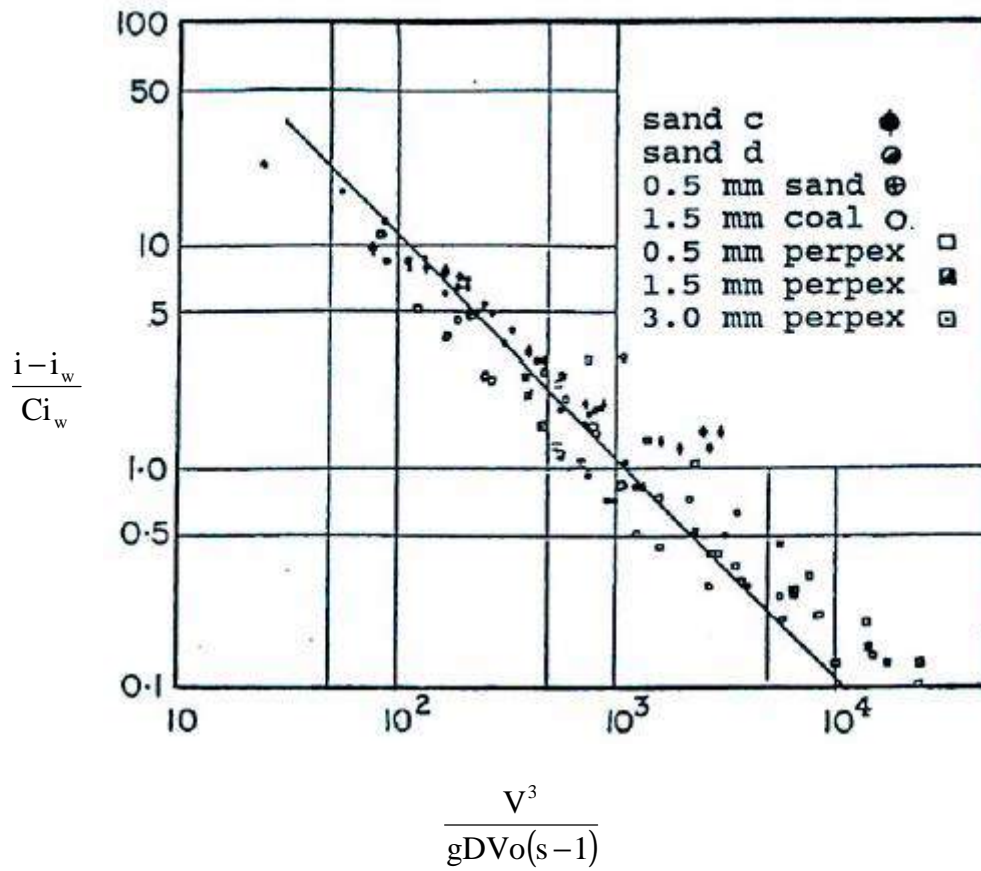


Fig (3.9) Comparison of the predictions for the pressure Gradi
(After HAYDEN 1968)



Fig(3.10) Test of Newitt et al . Equation.
(After NEWITT et al 1955)

Table (3.3) Transport theories for Moving and Stationary Bed flow, Flow velocity as criterion. As cited in (Kuhail 1989) with modifications.

3.2.2 Moving and Stationary Bed flow	
3.2.2.1 Flow velocity as criterion	
21	Laursen (1956)
$L = \left[\frac{VA}{D^{2.5} C^{1/3} \sqrt{g(s-1)}} \right]$	<p><i>Laursen (1956)</i> found that the flow velocity and the blockage ratio (A_b/A) depends on the dimensionless “L”</p> $L = \left[\frac{VA}{D^{2.5} C^{1/3} \sqrt{g(s-1)}} \right]$ <p>Laursen combined the dimensionless parameter “L” and the blockage ratio as shown in Fig(3.11)</p>
22	Condolios and Chapus (1963)
$\frac{V}{\sqrt{gD_{EF}}} = Z$ $\frac{V}{\sqrt{D_{EF}g}} = \frac{V_2}{\sqrt{Dg}}$	<p><i>Condolios and Chapus (1963)</i> report on the basis of actual measurements with water-solid mixtures above the bed “V” adjust it self so that ;</p> $\frac{V}{\sqrt{gD_{EF}}} = Z$ <p>Where D_{EF} is the equivalent diameter of the conduit above the deposited bed and Z is a function only of the solid concentration and particle size. They state that the previous equation also applies at the condition of first deposition of solids, namely when $D_{EF} = D$ and $V=V_2$ indicating that ;</p> $\frac{V}{\sqrt{D_{EF}g}} = \frac{V_2}{\sqrt{Dg}}$
23	Wicks (1968)
$V = \frac{V_2 A}{(A - A_b)}$ $f' = f(s'')$ $f' = \frac{f^3 dV^4}{(fs - f)gm^2}$ $s'' = \left[\frac{d}{D} \right]^{1/3} \left[\frac{DV}{v} \right]$ $f' = 0.1s''^2 \text{ for } s'' < 40$ $f' = 100s''^{1.5} \text{ for } s'' < 400$	<p><i>Wicks (1968)</i> found that if V is calculated from</p> $V = \frac{V_2 A}{(A - A_b)}$ <p>Where A_b is the area of the deposited bed, the ratio $\frac{V}{\sqrt{gD_{EF}}}$ is not constant and varies as much as 65% from the value corresponding with $\frac{V_2}{\sqrt{Dg}}$ as reported by Condolios and Chapus for similar mixture.</p>

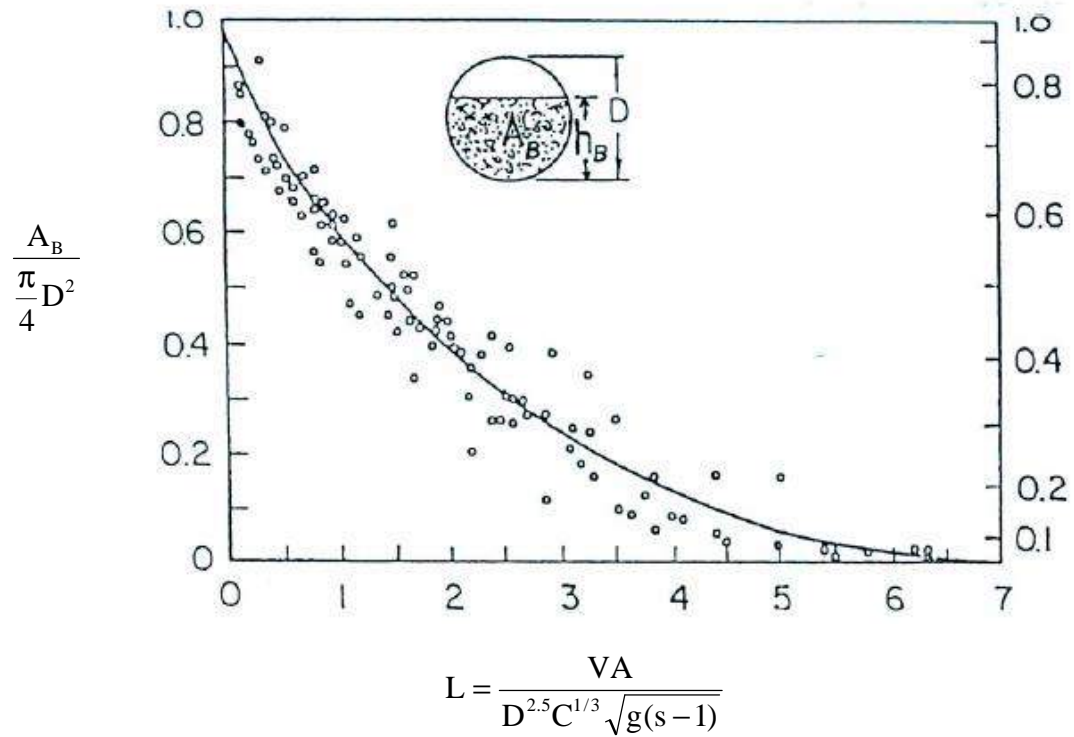


Fig (3.11) Blockage due to sediment deposition for uniform Sand graded Sands, and Slag
(After LAURSEN 1956)

Table (3.4) Transport theories for Moving and Stationary Bed flow, Shear stress or head loss as criterion

3.2.2.2 Shear stress or head loss as criterion	
24	Gibert and Condolios(1960)
$\frac{i - i_w}{Ci_w} = K_D \left[\frac{v^2 \sqrt{C_D}}{g * 4 * R(s-1)} \right]^{-1.5}$ $i_w = \frac{fV^2}{8gR}$ $f = f'' \left[\left(\frac{V4R}{v} \right) \frac{Ks}{R} \right]$ $f = f' \left(\frac{V4R}{v} \right)$	<p>Gibert and Condolios discussed the possibility of extending the Durand and Condolios relationship (1953) to the case of deposited bed. They reported that the top of the deposit was approximately flat and the deposit height was measured.</p> <p>The hydraulic radius R which replaces the pipe diameter) is the hydraulic radius of the free-flow cross sectional area. With this in mind, Durand et al . becomes;</p> $\frac{i - i_w}{Ci_w} = K_D \left[\frac{v^2 \sqrt{C_D}}{g * 4 * R(s-1)} \right]^{-1.5}$
25	Carven (1952)
$i = E(Qs / d^{2.5} \sqrt{g}, (s-1), h_b / D, Ks / D, Re)$	<p>Carven (1952) studied the problem by the application of dimensional procedures and by determination of the functional relationship. It was proposed that the following relation is sufficient to describe the sediment transport problem ;</p> $i = E(Qs / d^{2.5} \sqrt{g}, (s-1), h_b / D, Ks / D, Re)$ <p>However only a relatively small amount of data was used to establish the function, and thus remains of limited use.</p>
26	Graf and Acarglu (1968)
$\Phi = E(f')$ $\Phi = \frac{CVR}{\sqrt{gd^3(s-1)}}$ $\Phi' = \frac{(s-1)d}{iR}$ $\frac{V}{8gR(s-1)} = 0.732 f^{-0.624} C^{0.248} (d/R)^{0.252}$	<p>Graf and Acarglu (1968) have studied the hydraulic transport of solids in presence of a bed in open channels, rectangular conduits, and pipes. By consideration of the hydrodynamics forces acting on the particles , they show that the shear intensity parameter ϕ' must be a function of the transport parameter Φ . See Fig(3.12)</p> $\Phi = E(f')$ <p>where</p> $\Phi = \frac{CVR}{\sqrt{gd^3(s-1)}}$ $\Phi' = \frac{(s-1)d}{iR}$ <p>The relationship between the shear intensity parameter and the transport parameter :</p> $\frac{V}{8gR(s-1)} = 0.732 f^{-0.624} C^{0.248} (d/R)^{0.252}$

27	Wilson (1970)	
	$\frac{i}{f} \left[\frac{q^* - \sin q^* \cos q^*}{4} + \frac{A_b}{D} \left[\sin q^* - \frac{r_s \tan f}{\tan f} \right] \right] = r_s (s-1) Es (\sin q - q^* \cos q^*) x \left[\frac{g}{2g_c} \right]$	
	<p>Wilson (1970) has made a force balance analysis for the condition of incipient motion of a bed of granular solids in pipe, corresponding with V3.</p> <p>θ^* = one half the angles subtended at the pipe center by the surface of the bed of solids , radians. r_s = Coefficient of static friction for the solids against the pipe wall. \emptyset = angle of repose of solid particles Es = volume fraction of solids in the bed.</p>	
28	Urcikan (1984)	
	$\frac{1}{\sqrt{f}} = -2 \log_{10} \left[a1 + \left[\frac{2.51}{Re \sqrt{f}} \right] + \left[\frac{Ks}{3.71D} \right] \right]$ $a1 = \frac{510(f_s C)0.666}{Re - 5000^{1.25}} - \frac{140Ks}{0.05 + D}$ <p>f_s is the density of the sediment in Kh/m^3</p>	<p><i>Urcikan (1984)</i> describe head loss measurements obtained from a test rig built at a sewage treatment works. Five sizes of PVC pipe were studied under pipe full conditions using wastewater and sediments removed from combined sewers. A modification to the Colebrook-White equation was proposed to describe the effect of sediment on head loss. The equation applies for sediments with mean weighted diameter between 0.9mm and 1.2mm</p>
29	Ackers and White (1984)	
	$V = -\sqrt{32gRi} \log \left[\frac{Kss}{14.8} + \frac{1.255v}{R\sqrt{32gRi}} \right]$ $Kss = \frac{P_w x Ks + P_b x d_{50}}{P_w + P_b}$ $G_{gr} = c' \left[\frac{F_{gr}}{A_c - 1} \right]^m$ $F_{gr} = \left[\frac{CD}{d} \right] x \left[\frac{V_*}{V} \right]^n$ $= \frac{(V^{1-n} V^{n*})}{\left[\sqrt{gd(s-1)} \left[\log(10D/d) \sqrt{32} \right] \right]^{1-n}}$ $D_{gr} = \left[\frac{g(s-1)}{V^2} \right]$ $\log c' = 2.86 \log D_{gr} - (\log D_{gr})^2 - 3.53$	<p><i>Ackers and White (1984)</i> combined their equation with Colebrook –White equation to make use of their formula for designing pipes with deposited bed.</p> <p>where G_{gr} = a transport function F_{gr} = a mobility number c' = an empirical coefficient A_c = initial motion condition (empirical)</p>

30	Ab Ghani (1993)	
	$C_v = 0.355 \left(\frac{W_b}{y_o} \right)^{1.12} \left(\frac{D}{d_{50}} \right) I_c^{1.94} \left(\frac{V^2}{g(s-1)D} \right)^{3.12}$ $I_c = 0.0014 C_v^{-0.04} \left(\frac{W_b}{y_o} \right)^{0.34} \left(\frac{R}{d_{50}} \right)^{0.24} D_{gr}^{0.54}$ $D_{gr} = d_{50} \left(\frac{g(S_s - 1)}{v^2} \right)$	<p>Where</p> <p>C_v = volumetric sediment concentration W_b = sediment bed width g = acceleration due to gravity y_o = flow depth above sediment bed D = pipe diameter d_{50} = mean sediment size V = mean velocity R = hydraulic radius D_{gr} = Sediment parameter d_{50} = mean sediment size S_s = specific gravity of sediment v = kinematic viscosity of water</p>
31	May (1993)	
	$C_v = \Omega \left(\frac{D^2}{A} \right) \left(\frac{y}{D} \right)^{0.6} \left(\frac{I_g V_L^2}{8gf(s-1)D} \right)^{1.5}$ <p>Where Ω depend on the particle mobility G_s</p> $G_s = \left(\frac{y}{D} \right)^{0.2} \left(\frac{I_g V_L^2}{8gf(s-1)D_{50}} \right)^{0.5}$ <p> $G_s \leq 0.15$ $\Omega = 0$ $0.15 < G_s \leq 0.55$ $\Omega = 8.25 G_s - 1.24$ $0.55 < G_s \leq 0.9$ $\Omega = 1.78 G_s - 2.32$ </p> <p>λ_g is derived from</p> $\frac{1}{\sqrt{I_g}} = -2 \log_{10} \left(\frac{d_{50}}{12R} + \frac{0.6275v}{VR\sqrt{I_g}} \right)$	<p>Where A = area of flow y = flow depth λ_g = friction factor V_L = limiting flow velocity without deposition f = friction coefficient between pipe and sediment</p> <p>The factor λ_g is a friction factor corresponding to the grain shear stress and is derived from the Colebrook-White equation. This factor takes into account the sediment characteristics.</p>

32	Kuhail (1996)	
	$\frac{C \cdot d_{50} V^4}{n^2 \cdot (s-1) g} = a \left[\frac{i^{0.5} R_E^{1.5}}{n} \left(\frac{d_{50}}{D} \right)^{2/3} \text{Log} \left(\frac{14.8 R_E}{K_{SS}} \right) \right]^b$ <p> $a = 0.01$ $b = 2.789$ $K_{SS} = \frac{P_w K_s + P_b d_{50}}{P_w + P_b}$ </p>	<p>Using d_{50} as representative of the sediment.</p> <p>K_{SS} = total roughness</p> <p>K_s = Surface roughness of Pipe</p> <p>P_w = wet perimeter of pipe</p> <p>P_b = width of sediment</p>
33	Kuhail (1989)	
	$\frac{C R^{4/3} \sqrt{(s-1)}}{V K_{SS}^{1/3} \sqrt{gd}} = A \left[\frac{V_*^2}{(s-1)gd} \right]^B$	<p>1. Using d_{50} as representative of the sediment.</p> <p>a. Deposited bed $TP_1 = 0.0447385 \times SP^{2.336}$</p> <p>b. Without deposited bed $TP_2 = 0.0152039 \times SP^{1.24}$</p> <p>2. Using d_{50}/AM as representative of the sediment.</p> <p>a. Deposited bed $TPK_1 = 0.434991 \times 10^{-2} \times SPK^{1.775}$</p> <p>b. Without deposited bed $TPK_2 = 0.359454 \times 10^{-2} \times SP^{1.125}$</p>

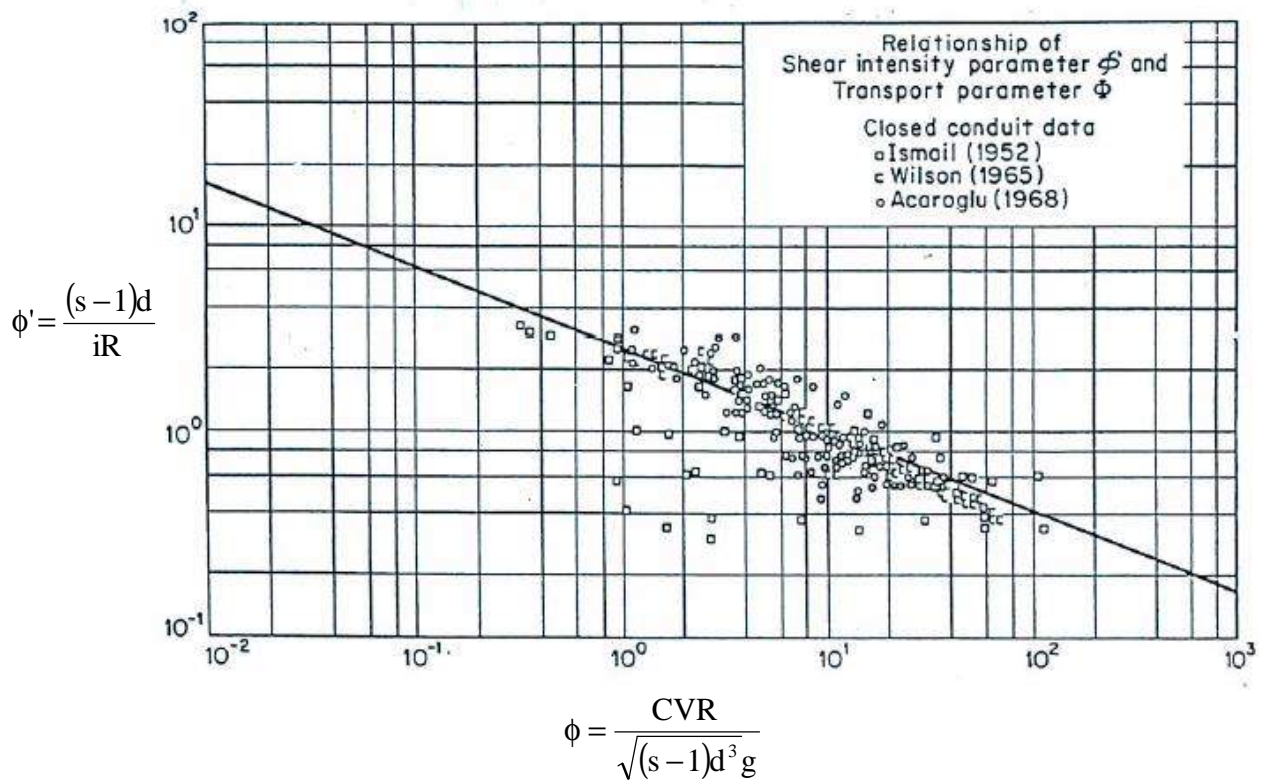


Fig (3.12) Graphical representation of Graf et al Equation
 (After GRAF et al. 1968)

Chapter 4

Sediments transport in sewer

4.1 Introduction

The processes of sediment transport in sewer system is complex. The understanding of the sediment transport phenomena has advanced considerably during the last decades. A considerable amount of experience has been transferred from sediment transport in rivers. The ultimate success to apply the theories from this field has not been obtained. In sewers; the hydraulic conditions may change rapidly both in time

and space which of course will change the conditions for sediment transport as well. Another important difference is the influence of the cross sectional shape and cross section changes in the sewer system. This induces turbulence and local possibilities for sedimentation. Particle types , sizes and distributions are restively well definable in rivers but in sewer a number of different types of particles occur. These particle dose not necessarily behave in the same manner when eroded , transported or deposited.

Atmospheric precipitation of particles during the rain event, wash off of sediments from the surface, build-up and erosion and transport of sediment through the gullies, transport of sediments during dry weather, build-up of deposits in the sewer during the dry weather period, transport and erosion of sediments during rain events in suspension or near the bed including first flush etc, dose not have simply definable timescales as the processes depends on both the hydraulic conditions, time and the composition of the sediments themselves.

4.2 Storm Sewer Junction Hydraulics and Sediment Transport ⁽¹⁴⁾

4.2.1 Introduction

Stormwater can carry substantial loadings of suspended solids into storm sewer systems. A potentially important issue associated with sediment transport through sewers is “shock loading.” This occurs when periods of low flow leave significant sediment deposits in sewers and manholes. A large event can then provide the energy to scour the deposited material and transport far more loading to a receiving waterbody than is generated from the land surface during the event. This research will present the results of significant physical modeling research on energy losses associated with manholes and discuss how these losses might effect sediment transport. Additionally, a 3-D hydrodynamic model was applied to various manholes configurations to qualitatively assess circulation tendencies and sediment movement within manholes.

4.2.2 Physical Modeling of Storm Sewer Junction Hydraulics

While head losses can not be directly correlated to sediment transport potential, these losses are caused by the high turbulence which exists in a manhole in high flow conditions. The water churns and bubbles violently (secondary motion), even under steady-state inflow conditions. This high turbulence has the potential to suspend particles, which had previously settled to the manhole floor. So even if the horizontal velocity in the junction is relatively small (remember that water elevations in a manhole can be far higher that the crowns of inflow and outflow pipes), particles may be continuously resuspended and available for outflow transport.

The hydraulic analysis through a manhole focuses on the calculation of the energy loss from the inflow pipes to the outflow pipe. Several determining factors affect the computation of the energy loss coefficient in the HGL methodology. These include :

- the manhole size relative to the outlet pipe diameter,
- the depth of flow in the manhole,

- the amount of discharge,
- the inflow pipe angle,
- the plunge height, the relative pipe diameter,
- the floor configuration.

A lab study was performed in an attempt to isolate these different factors. A test matrix was developed to examine the seemingly endless number of physical configurations. Hundreds of runs were conducted under a wide variety of flow conditions. Thousands of data points were collected and analyzed. The hydraulic gradeline for the flow was measured and from that, the energy gradeline was calculated. The energy loss through the manholes was calculated and the results were used to create the empirical equations. Needless to say, the results tended to exhibit a lot of scatter.⁽¹⁴⁾

The hydraulic gradeline across manholes methodology starts with equation (4.1) which describes the energy loss for an inflow pipe:

$$\Delta E = (C_1 C_2 C_3 + C_4) w \frac{V_o^2}{2g} \quad \text{Equation (4.1)}$$

Where:

ΔE = Energy loss for an inflow pipe, m.

C_1 = Coefficient related to relative manhole size.

C_2 = Coefficient related to water depth in the manhole.

C_3 = Coefficient related to lateral flow, lateral angle, and plunging flow.

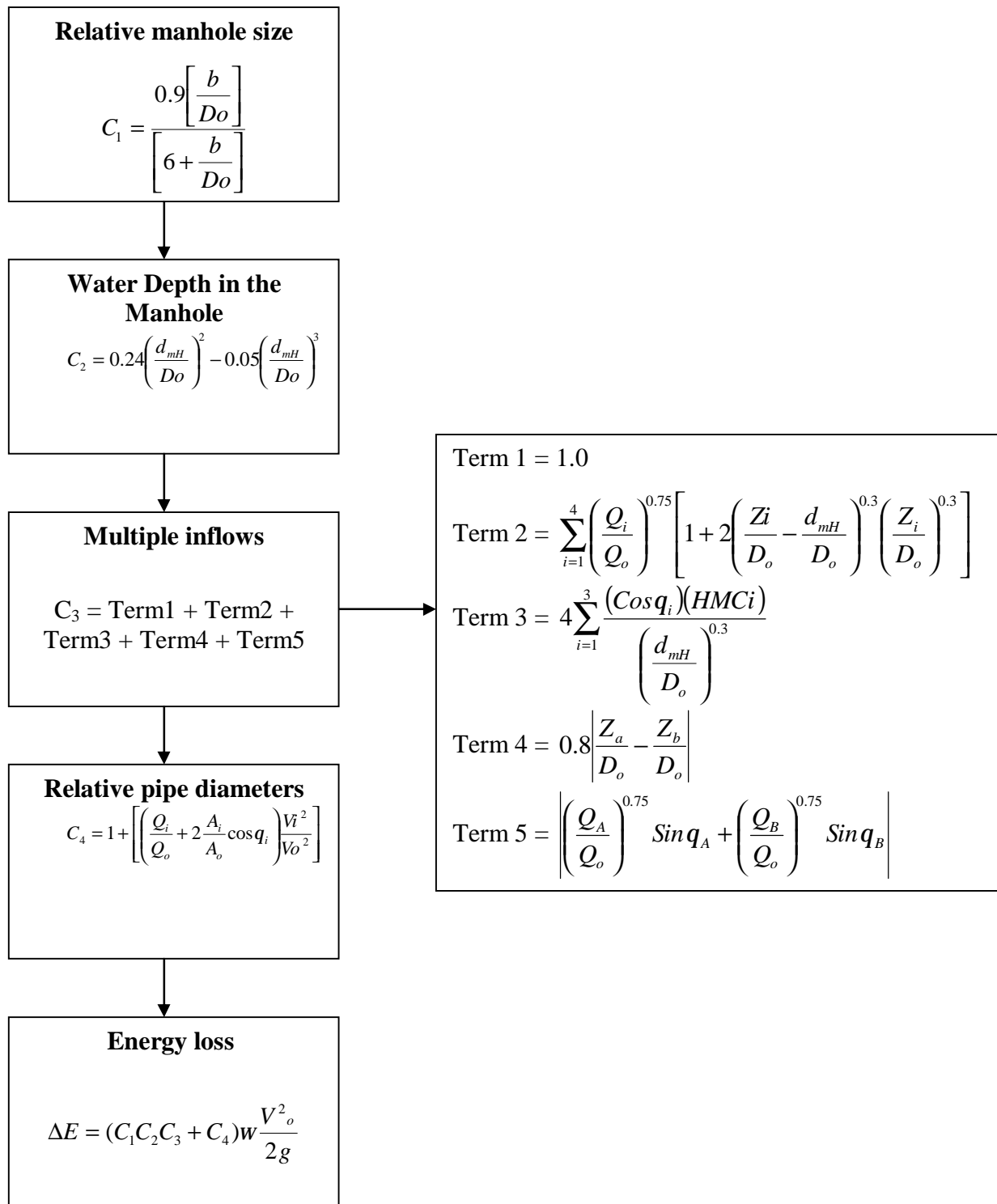
C_4 = Coefficient related to relative pipe diameters.

w = Correction factor for benching.

$\frac{V_o^2}{2g}$ = Velocity head, m/s².

Schematic diagram of energy loss calculations shown in fig (4.1)

Fig (4.1) Schematic diagram of energy loss calculations



4.2.3 Relative manhole size

The larger the manhole is relative to the outlet pipe, the greater the space for the flow to expand and dissipate the velocity head. Similarly, the greater the expansion into the manhole, the greater the energy losses in contracting to leave through the outlet pipe. The modeling indicates that for b/D_o , (manhole diameter/outlet pipe diameter), values up to 4.0, the coefficient related to manhole size, C_1 , is calculated with the following equation:

$$C_1 = \frac{0.9 \left[\frac{b}{D_o} \right]}{\left[6 + \frac{b}{D_o} \right]} \quad \text{Equation (4.2)}$$

Where:

b = Manhole diameter, m.

D_o = Outflow pipe diameter, m.

Once the manhole diameter is four times the outlet pipe diameter, or larger, the manhole is “large” and the coefficient C_1 is assumed to be a constant equal to 0.36.

4.2.4 Water Depth in the Manhole

The coefficient, C_2 , which is related to manhole water depth, increases rapidly with relative water depth, d_{mH}/D_o , up to 2.0. The rate of the increase slows when d_{mH}/D_o reaches approximately 3.0. This type of curve can be expressed as a third order polynomial. Equation (4.3) applies for $d_{mH}/D_o \neq 3.0$. When d_{mH}/D_o is greater than 3.0, C_2 is equal to 0.82. The following equation was found to fit the data reasonably well:

$$C_2 = 0.24 \left(\frac{d_{mH}}{D_o} \right)^2 - 0.05 \left(\frac{d_{mH}}{D_o} \right)^3 \quad \text{Equation (4.3)}$$

Where:

d_{mH} = Depth in the manhole relative to the outlet pipe invert, m.

4.2.5 Multiple inflows

The coefficient related to multiple inflows, C_3 , is the most complex term in the composite energy loss coefficient equation. The effect of lateral flows on the energy loss was studied with respect to three parameters: flow rate, connecting angle of the inflow pipe, and elevation of the inflow pipe. The following equation can be used to calculate the coefficient C_3 .

$$C_3 = \text{Term1} + \text{Term2} + \text{Term3} + \text{Term4} + \text{Term5} \quad \text{Equation (4.4)}$$

$$\text{Term 1} = 1.0$$

$$\text{Term 2} = \sum_{i=1}^4 \left(\frac{Q_i}{Q_o} \right)^{0.75} \left[1 + 2 \left(\frac{Z_i}{D_o} - \frac{d_{mH}}{D_o} \right)^{0.3} \left(\frac{Z_i}{D_o} \right)^{0.3} \right]$$

$$\text{Term 3} = 4 \sum_{i=1}^3 \frac{(\cos q_i)(HMC_i)}{\left(\frac{d_{mH}}{D_o} \right)^{0.3}}$$

$$\text{Term 4} = 0.8 \left| \frac{Z_a}{D_o} - \frac{Z_b}{D_o} \right|$$

$$\text{Term 5} = \left| \left(\frac{Q_A}{Q_o} \right)^{0.75} \sin q_A + \left(\frac{Q_B}{Q_o} \right)^{0.75} \sin q_B \right|$$

$$HMC_i = \left[0.85 - \left(\frac{Z_i}{D_o} \right) \left(\frac{Q_i}{Q_o} \right)^{0.75} \right] \quad \text{Equation (4.5)}$$

Where:

Q_0 = Total discharge in the outlet pipe, m³/s.

Q_1, Q_2, Q_3 = Pipe discharge in inflow pipes 1, 2, and 3, m³/s.

Q_4 = Discharge into manhole from the inlet, m³/s.

Z_1, Z_2, Z_3	= Invert elevation of inflow pipes 1, 2, and 3 relative to the outlet
Z_4	= Elevation of the inlet relative to the outlet pipe invert, m.
D_0	= Outlet pipe diameter, m.
b	= Manhole diameter, m.
d_{mH}	= Depth in the manhole relative to the outlet pipe invert, m.
q_1, q_2, q_3	= Angle between the outlet main and inflow pipes 1, 2, and 3, degrees.
HMC_i	= Horizontal momentum check for pipe i.
Q_A, Q_B	= Pipe discharges for the pair of inflow pipes that produce the largest value for term 4, m ³ /s.
Z_a, Z_b	= Invert elevation, relative to outlet pipe invert, for the inflow pipes that produce the largest value for term 4, m.

All angles are represented between 0 and 360 degrees for this equation and are measured clockwise from the outlet pipe. For a simple two-pipe system with no plunging flow, C_3 is assumed to be equal to 1.0.

The **second term** in equation (4.4) captures the energy losses from plunging inflows. This term reflects the fact that flows plunging from greater heights result in greater turbulence and, therefore, higher energy losses. As shown by the summation in the second term, the computation is valid for one to three inflow pipes and plunging flow from the inlet.

The **third term** reflects the effects the angle (with respect to the outflow pipe) has on energy losses. If the horizontal momentum check (HMC_i) is less than 0, the flow is falling from a height such that the horizontal momentum is assumed to be negligible and term 3 is set to 0.

The fourth and fifth terms deal with lateral flows and what effect, if any, they have on the overall energy losses.

4.2.6 Relative pipe diameters

Although no experiments were performed with different pipe diameters in this study, a correction for such a case is required in the hydraulic gradeline analysis. Equation (4.6) was theoretically derived based on conservation of momentum and is proposed for this purpose. Each loss is unique to each inflow pipe and does not affect any other inflow pipes.

$$C_4 = 1 + \left[\left(\frac{Q_i}{Q_o} + 2 \frac{A_i}{A_o} \cos \theta_i \right) \frac{V_i^2}{V_o^2} \right] \quad \text{Equation (4.6)}$$

Where:

A_i, A_o = Cross-sectional area of inflow and outflow pipes, m².

θ_i = Angle between outflow pipe and inflow pipe i, degrees.

If θ_i for any pipe is less than 90 degrees or greater than 270 degrees, ($\cos \theta_i$) is replaced with 0 in equation (4.6). This sets the maximum exit loss to be the incoming velocity head.

Application of above equations is shown in Appendix A

4.3 Self-Cleansing Velocities ^(13,14)

Sediment can reduce the capacity of a stormwater pipe over time. In some installations, it may render the pipe useless until the system can be cleaned. This is an expensive, time-consuming undertaking, so preventive measures should be taken during design.

Sedimentation is of great concern in storm sewer application, because large, heavy grit may be present. To minimize potential problems, flow should be maintained at a minimum, or self-cleansing, velocity.

Flow velocity can be increased by either increasing the slope of the pipe or by using a smaller diameter. Modifying either the slope or pipe size requires careful consideration of site factors and flow needs. However, by using a corrugated polyethylene pipe with a smoother interior (a lower Manning's "n"), a smaller diameter pipe can often be selected in lieu of alternative pipe materials without adversely affecting capacities or modifying the slope of the line.

The potential for settling is determined by the specific gravity and diameter of the particle and flow velocity. The formula for self-cleansing velocity is shown below.

$$V_{sc} = \frac{R^{1/6} [B(sg - 1)Dg]^{1/2}}{n}$$

Where:

V_{SC} = minimum self-cleansing velocity in a full-flow condition (m/s)

R = hydraulic radius (m)

B = constant equal to 0.04 for clean granular particles or 0.8 for cohesive material, unitless

sg = specific gravity of the soil particle

D_g = particle diameter (m)

Minimum required flow for different sediment transport criteria is shown in Fig (4.2)

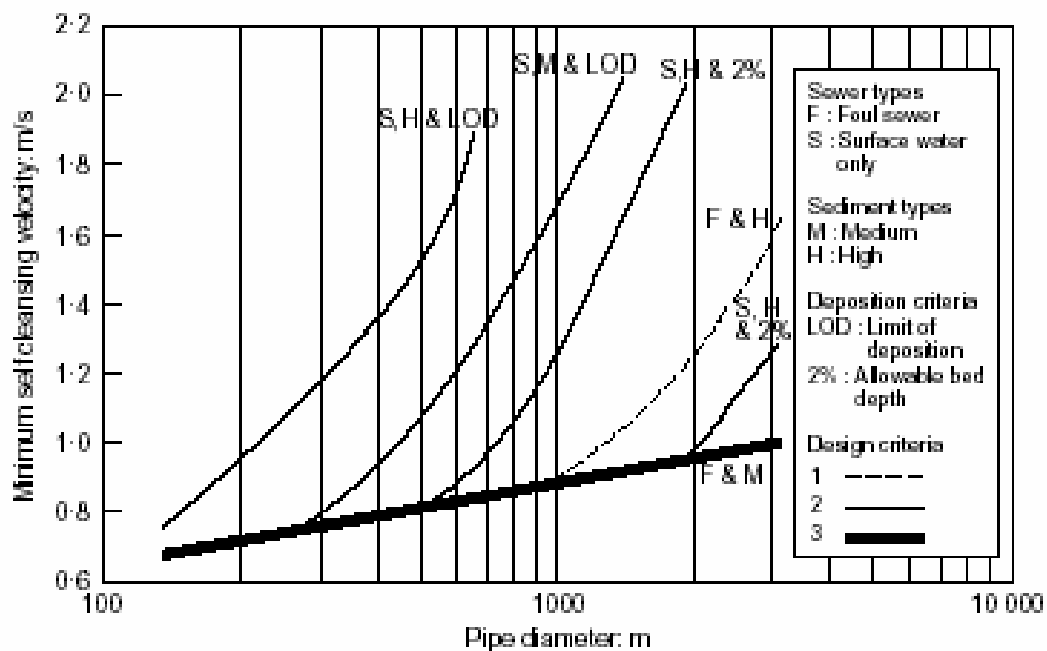


Fig (4.2) Minimum required flow for different sediment transport criteria

(After Ashly 1999)⁽¹⁾

Chapter 5

Theoretical Analysis

5.1 Movement Processes of sediments within Storm Sewer

After (Kuhail 1989)⁽⁶⁾

The movement of sediment comprises three phases:

1. **Entrainment**, which covers the process of initial movement of sediment and its pick up into the flow. Fig (5.1) illustrates the relation, sediment size and critical velocity of erosion.
2. **Transport**, which is divided in to four flow pattern: Stationary bed, Bed load transport, suspended load (heterogeneous flow) and wash load (homogeneous flow).
3. **Deposition**, which occurs when a flow can no longer transport the sediment. This may be because the flow is reducing or because it has less energy.

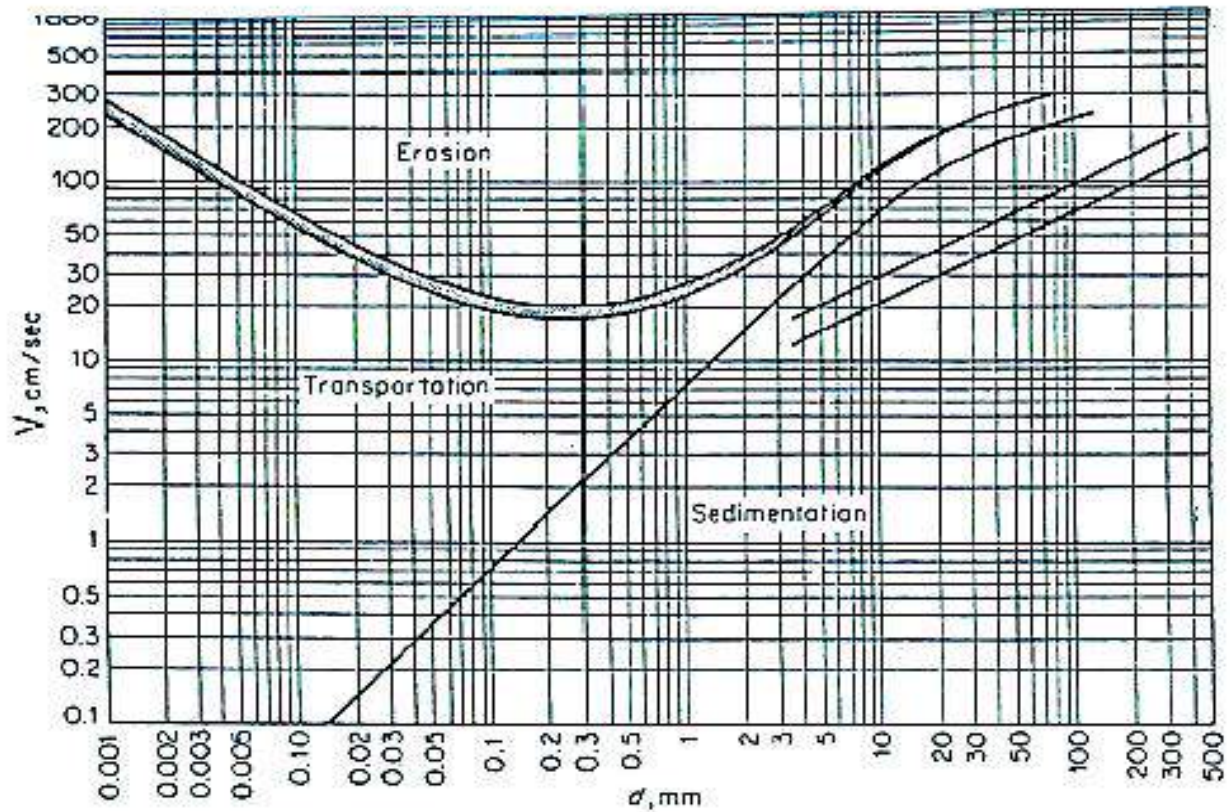


Fig (5.1) Erosion –Deposition criteria for uniform particles

(After HJULSTORM 1953)

5.2 Forces due to Impact of liquid on particles

For general pipe systems; the condition of incipient motion for an assembly of cohesionless, loose and solid particles is described in term of the forces acting on particle by Fig (5.2)

$$\tan(\theta) = \frac{F_p}{F_n} \quad (5.1)$$

Where;

F_p = the resultant forces in the direction parallel to the angle of repose of submerged material.

F_n = the resultant forces in the direction normal to the angle of repose of submerged material.

θ = the angle of repose of submerged material.

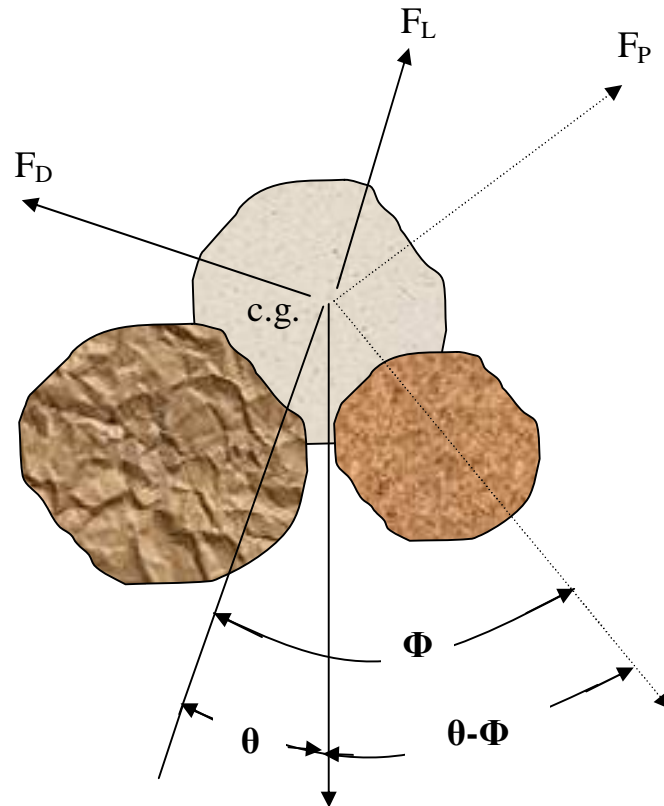


Fig (5.2) Forces on sediment grain in bed of sloping pipe.

5.3 Derivation of Equations

for the stage when the resultant force F_p is in the direction of F_D and F_n will be in the direction of P_L the equilibrium equations can be written as;

$$F_p = F_D \quad (5.2)$$

$$F_n = w' - F_L \quad (5.3)$$

$$\tan(\theta) = \frac{F_D}{(w' - F_L)} \quad (5.4)$$

Re-arranging equation (5.4)

$$F_D = w' \tan(\theta) - F_L \tan(\theta) \quad (5.5)$$

where F_D is the drag force and equal to

$$F_D = \frac{C_D B_1 d^2 f U_b^2}{2} \quad (5.6)$$

F_L is the lift force and equal to

$$F_L = \frac{C_L B_2 d^2 U_b^2}{2} \quad (5.7)$$

u_b the fluid velocity at the bottom of the pipe

C_D, C_L are drag and lift coefficient respectively

B_1, B_2 are factors (function of the particle shape factors)

W' the submerged weight of the particle

$$W' = B_3 (f_s - f) g d^3 \quad (5.8)$$

B_3 is another factor, (function of the particle shape factors)

There are a relation between the Drag force and lift force by Chepil (1959)

$$F_L = B_5 F_D \quad B_5 = 0.85 \quad (5.9)$$

Apply eq. (5.9) in eq. (5.4)

$$F_D = W' \tan(\theta) - B_5 F_D \tan(\theta)$$

$$\text{So } W' \tan(\theta) = F_D (1 - B_5 \tan(\theta))$$

By taking

$$a_1 = \frac{(1 - B_5 \tan(\theta))}{\tan(\theta)}$$

the previous equation can be re-written as ;

$$W' = a_1 F_D \quad (5.10)$$

By combining eq. (5.10), (5.6) and (5.8)

$$\frac{a_1 C_D B_1 d^2 f U_b^2}{2} = B_3 (f_s - f) g d^3 \quad (5.11)$$

but

$$C_D = f_1 \left(\frac{dU_b}{v}, B_4 \right) \quad (5.12)$$

Where; B_4 is a factor, function of the particle shape factor.

$$C_D = f_2 (Re^*, B_4) \quad (5.13)$$

According to Einstein et al (1955) and Newitt (1955)

$$u_b = v^* f_3 (Re^*, y/ks, C) \quad (5.14)$$

where;

Re^* is the particle shear Reynolds number

$$Re^* = \frac{dv^*}{v}$$

$$v^* = \sqrt{gRi}$$

y = is the depth of the flow

ks = is the equivalent roughness of pipe

In case of deposited bed in pipe

$$u_b = v^* f_3 (Re^*, D_{Ef} / k_{ss}, C)$$

where ;

C = sediment concentration

D_{Ef} is the equivalent diameter of the flow area inside the pipe

k_{ss} is the equivalent roughness of the pipe roughness and the bed roughness.

Substituting equation (5.15) into eq(5.11)

$$0.5a_1 f_2(\text{Re}^*, B_4) B_1 d^2 f [v^* f_3(\text{Re}^*, D_{\text{Ef}} / K_{\text{ss}}, C)]^2 = B_3 (f_s - f) g d^3 \quad (5.17)$$

Re arranging eq (5.17)

$$\frac{v^{2*}}{(s-1)gd} = \frac{B_3}{(0.5a_1 B_1)} f_4(\text{Re}^*, D_{\text{Ef}} / K_{\text{ss}}, C, B_4) \quad (5.18)$$

$$\frac{v^{2*}}{(s-1)gd} = f_5(\text{Re}^*, D_{\text{Ef}} / K_{\text{ss}}, C) \quad (5.19)$$

The left hand side was called the shear parameter It is dimensionless and can be used as sediment transport criterion denoted as SP

$$SP = \frac{v^{2*}}{(s-1)gd} \quad (5.20)$$

This parameter has been used to describe the hydrodynamic forces in open channel and closed conduit by many researchers.

Some of this researcher are :

1. Shield (1936)

$$\begin{aligned} \frac{\tau_0}{[(s_s - s)d]} &= \frac{fgRi}{[(f_s - f)gd]} \\ &= \frac{V_*^2}{[(s-1)gd]} \\ &= SP \end{aligned}$$

2. Einstien –Brown (1942 ,1950)

$$\frac{(s_s - s)d}{\tau_0} = \frac{1}{SP}$$

3. Acaroglu et al. (1968)

$$\frac{(s-1)d}{iR} = \frac{(s-1)gd}{V_*^2}$$
$$= \frac{1}{SP}$$

4. Novak & Nalluri (1975)

$$\frac{(s-1)gd}{V_*^2} = \frac{1}{SP}$$

So it was decided to use SP as transport criterion through this analysis.

Work rate concept:

The available power P is given by:

$$P = \gamma QH \quad (5.21)$$

where

H = the available power at any particular section

Q = the discharge of fluid, owing to continuity $Q=VA$

The power per unit length per unit perimeter P is

$$P^* = \frac{\gamma V A_i}{P}$$

Taking $A/P = R$ and $g\rho Rr_i = \tau_0$

$$P^* = \tau_0 V \quad (5.21)$$

The power used for the transport of sediment makes a change in the available value of the shearing stress, so τ_0 becomes $F\tau_0$, and the power used becomes

$$P^*_{\text{used}} = \tau_0 VF \quad (5.22)$$

it is believed that the factor F has a functional relationship with shear parameter "SP"

where

$$F = f(\text{SP}) \quad (5.23)$$

the rate of work to transport the sediment is given by:

$$W^* = Gs \frac{(s-1)}{s} \tan \phi \quad (5.24)$$

Gs is the sediment load (weight in air per unit perimeter per unit time)

Acaroglu (1968) stated that the actual work rate is given by:

$$\frac{W^*}{W^+} = \frac{V}{w} \quad (5.25)$$

Where;

W^* is the actual work rate used in transport of sediment

W is the settling velocity

So the actual work rate becomes

$$W^* = \frac{Gs(s-1) \tan \phi}{s} \frac{V}{w} \quad (5.26)$$

The used power for transport = the actual rate used

$$P^*_{\text{used}} = W^* \quad (5.27)$$

$$\tau_0 Vf(SP) = \frac{Gs(s-1) \tan \phi}{s} \frac{V}{w} \quad (5.28)$$

but

$$Gs = C s \rho g V R \quad (5.29)$$

It is believed that any variation in $\tan \phi$ will appear in SP so it can be omitted and equation becomes

$$CgVR(s-1) = V_*^2 wf(SP) \quad (5.30)$$

If the settling velocity w is substituted by its equivalent

$$w = \sqrt{\frac{4(s-1)gd}{3C_D}} \quad (5.31)$$

so the equation becomes

$$CgVR(s-1) = V_*^2 f(SP) \sqrt{[(s-1)gd]} \sqrt{\frac{4}{3C_D}} \quad (5.31a)$$

Since the shear parameter includes the various particle coefficients such as C_D

$$CgVR(s-1) = V_*^2 \sqrt{[(s-1)gd]} f_6(SP) \quad (5.32)$$

By introducing Manning's formula for metric system

$$V = \frac{1}{n} R^{2/3} i^{1/2}. \quad (5.33)$$

or

$$V = \frac{1}{n} R^{1/6} \sqrt{Ri} \quad (5.34)$$

Multiplying both sides by (\sqrt{g}) gives

$$V_* = \frac{Vn\sqrt{g}}{R^{1/6}} \quad (5.35)$$

Webber (1971) quoted a relation between the manning coefficient "n" and the equivalent roughness of the pipe "Ks".

Webber's relationship is in the form

$$n = \frac{Ks^{1/6}}{m_1} \quad (5.36)$$

Where

$m_1 =$ constant and has the dimension of $(L^{1/2}T^{-1})$

$K_s =$ roughness of the pipe which can be replaced by its general form “ K_{ss} ”

$$n = \frac{K_{ss}^{1/6}}{m_1} \quad (5.37)$$

Combining equations (5.37) and (5.35)

$$V_* = \frac{V\sqrt{g}K_{ss}^{1/6}}{m_1R^{1/6}} \quad (5.38)$$

Substituting equation (5.38) in to (5.32) gives;

$$C_gVR(s-1) = \left[\frac{V\sqrt{g}K_{ss}^{1/6}}{m_1R^{1/6}} \right]^2 \sqrt{(s-1)gd}(f_6)(SP)$$

which can be re-written in the form

$$C_gR(s-1) = \left[\frac{VgK_{ss}^{1/3}}{m_1^2R^{1/3}} \right] \sqrt{[(s-1)gd]}(f_6)(SP)$$

Re-arranging the previous equation leads to

$$\frac{m_1^2CR^{3/4}\sqrt{(s-1)}}{[VK_{ss}^{1/3}\sqrt{gd}]} = f_6(SP) \quad (5.39)$$

or

$$\frac{m_1^2CR^{3/4}\sqrt{(s-1)}}{[VK_{ss}^{1/3}\sqrt{gd}]} = A_1(SP)^B$$

The value of m_1 is constant so if omitted from the previous equation its value will appear in the factors taken from regression analysis of the experimental data .

$$A = \frac{A_1}{m_1^2}$$

Substituting the equivalent for the shear parameter in to the previous equation gives

$$\frac{CR^{4/3}\sqrt{(s-1)}}{VK_{ss}^{1/3}\sqrt{gd}} = A \left[\frac{V_*^2}{(s-1)gd} \right]^B \quad (5.40)$$

5.4 Geometrical Relationships:

The relationships shown in (5.41 , 5.42) has been established as a mathematical relationships by (Kuhail 1989).

$$h_b / D = 0.786(Ab / A)^3 - 1.184(Ab / A)^2 + 1.385(Ab / A) + 0.0035 \quad (5.41)$$

$$Ab / A = -0.86(hb / D)^3 + 1.286(hb / D)^2 + 0.579(hb / D) - 0.0015 \quad (5.42)$$

5.5 Hydraulic parameters calculations:

Hydraulic Radius

$$R_b = D_{Eb} / 4 \quad \text{for area occupied by the sediment} \quad (5.43)$$

$$R = D_{Ef} / 4 \quad \text{for area occupied by the fluid} \quad (5.44)$$

5.6 Volumetric concentration:

$$C = Q_s / Q \quad (5.45)$$

5.7 Cross sectional area of the bed

$$A_b = (W_s / f_s) / L_s \quad (5.46)$$

A_b = Cross sectional area of the bed

W_s = weight of the sediment fed in to the pipe

L_s = the length of the sediment inside the pipe

5.8 Hydraulic Gradient “i”

The hydraulic gradient in the shear parameter “SP” is a function of the roughness of the pipe.

$$i = \frac{fV^2}{2gD} \quad (5.47)$$

$$f = f(V4R / v, K_s / R)$$

for one direction pipe (Kuhail1996)

$$\frac{C.d_{50}V^4}{n^2.(s-1)g} = a \left[\frac{i^{0.5} R_E^{1.5}}{n} \left(\frac{d_{50}}{D} \right)^{2/3} \text{Log} \left(\frac{14.8 R_E}{K_{ss}} \right) \right]^b \quad (5.48)$$

$$a = 0.01$$

$$b = 2.789$$

$$C = Q_s/Q$$

Q_s = Sediment transport rate

Q = water transport rate

V = Velocity

n = Viscosity

s = Specific gravity of sediment

g = gravity

i = gradient

D = Pipe diameter

d_{50} = grain diameter at 50 Passing

$$K_{ss} = \frac{P_w K_s + P_b d_{50}}{P_w + P_b} \quad (5.49)$$

K_{ss} = total roughness

K_s = Surface roughness of Pipe

P_w = wet perimeter of pipe

P_b = width of sediment

The using of K_{ss} instead of K_s in Colebrook White Equation for roughness pipe:

$$\frac{V - V_{ss}}{V_{ss}} = 9.5\%$$

V = the experimental velocity

V_{ss} = calculated velocity using K_{ss} instead of K_s

$$= f(i, R, K_{ss}, v)$$

$$i = 3.52 \frac{C^{0.7} V^{2.9} v^{0.6} D^{1.3}}{Z R_E^{2.8} d_{50}^{0.6} (s-1)^{0.7}} \quad (5.50)$$

Chapter 6

Experimental Set-Up

6.1 Introduction :

This chapter describes the sediment used in experiments and the experimental set-up layout of tanks, pipes , pumps, sediment feeder, and sediment trap.

The experiment has been designed to perform a uniform flow and high velocity flow so the length of pipe is designed to be suitable to Produce the uniform flow and to give the chance for the mixture to reach equilibrium stage before leaving the pipe to the outlet tank.

6.2 Sediment type used in experiment.

The type of sediment used in experiment is sand which has a sieve analysis as shown in Table(6.1) and Fig (6.1) .

Sand has been used in experiments due to following reasons:

- § Many investigations have been performed for the gutter in Gaza City since the sand was the predominant sediment type in gutter,
- § Also the hydraulic model has been more efficiently when using Sand according to pipe diameter , flow and velocity required in experiments.

According to the sieve analysis of sand in soil Lab of Islamic University of Gaza, the d50 has been found to be 0.3 mm .

Also the specific gravity tests was performed for the sample which was found 2.65 Fig (6.2) Shows the Sand sample used in the experiment .

Passing % (Sand)	Sample Wt.= 2000 g Wt. of Retained	Sieve Size (mm)
0.725	1985.5	0.075
2.68	1946.4	0.15
49.625	1007.5	0.3
87.815	243.7	0.425
99.58	8.4	0.6
99.775	4.5	1.18
99.855	2.9	2.36
100	0	4.75

Table (6.1) Sieve analysis of Sand

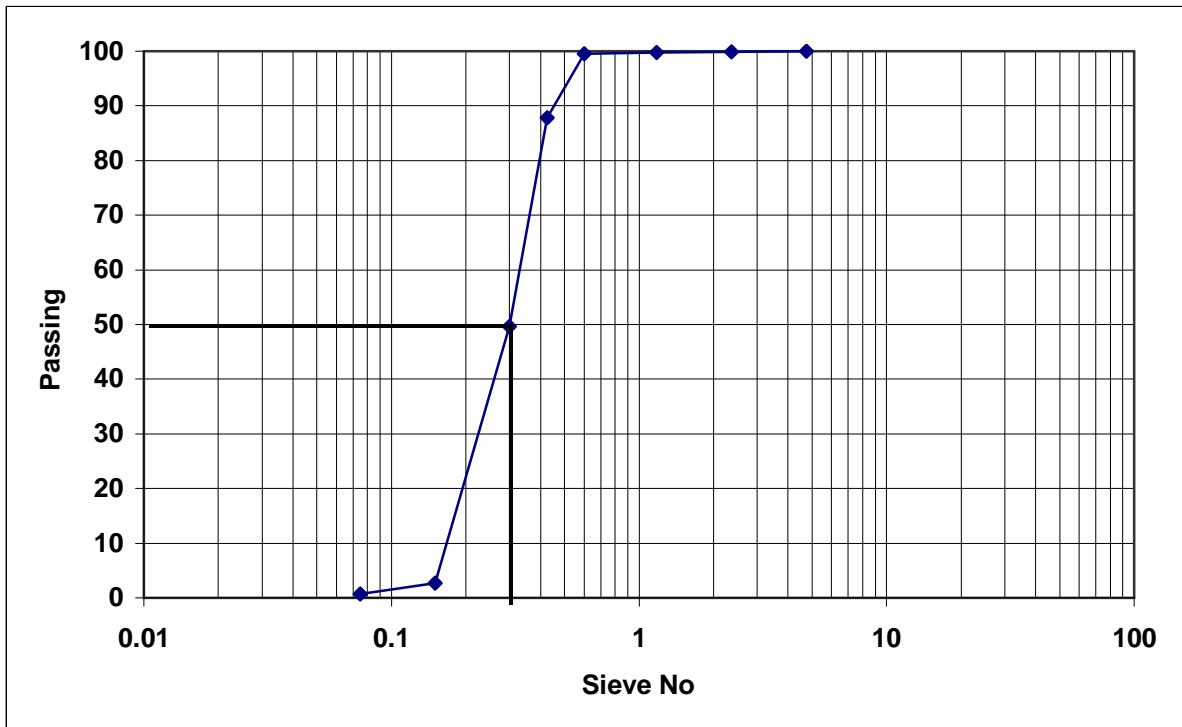


Fig (6.1) the sieve analysis of Sand

Fig (6.2) the Sand sample used in the experiment ($d_{50} = 0.3$ mm)

6.3 Layout of the Experiment Rig

Layout of the experiment rig consist of the following parts:

- The inlet upstream tank
- The outlet downstream tanks
- Sediment feeder
- Sediment trap
- Pumps
- The main pipes
- Manometer
- Tubing arrangements

A schematic diagram for Layout of Experiment rig is shown in Fig(6.2)

6.4 Description of the experiment layout

6.4.1 Tanks:

6.4.1.1 Inlet tanks:

Inlet tank receives the pumped water by pipe of 75mm diameter. Its Dimension is 60-cm diameter by 100-cm height.

It has two openings for 100-mm diameter UPVC pipes and drain valve was attached to the bottom of the tank to be used for emptying the tank in case of cleaning the tank see fig (6.3).



Fig (6.3) Inlet tank (upstream tank)

6.4.1.2 Outlet tanks:

This tank receives the flow from the 100mm-diameter UPVC pipe and pumped the water to inlet tank by 75-mm pipe. It was designed to have enough capacity to receive the flow from the pipe. Its dimension is 1.2m diameter by 1m height.

In the middle of the tank there is sediment trap for collecting the sediments see fig (6.4).



Fig (6.4) Outlet tank (Downstream tank)

6.4.1.3 Additional tanks:

There are two additional tanks to complete the network flow from inlet tank and outlet tanks. There is drain valve at the bottom of each. A dimension of each tank is 50 cm diameter by 100 cm height.

6.4.2 Pipes:

6.4.2.1 Main Pipes

The main pipe used is UPVC Pipe 8-meter length and 1% slope, and 100mm diameter. The pipe has four small window sections, each section has the dimension of 30 cm. Fig (6.5) and Fig (6.6) show the windows section and the sediments transport in pipe bed. Also the pipe has four pressure tubings as shown in Fig (6.7a) & Fig (6.7b).



Fig (6.5) Window section A



Fig (6.6) Window section B

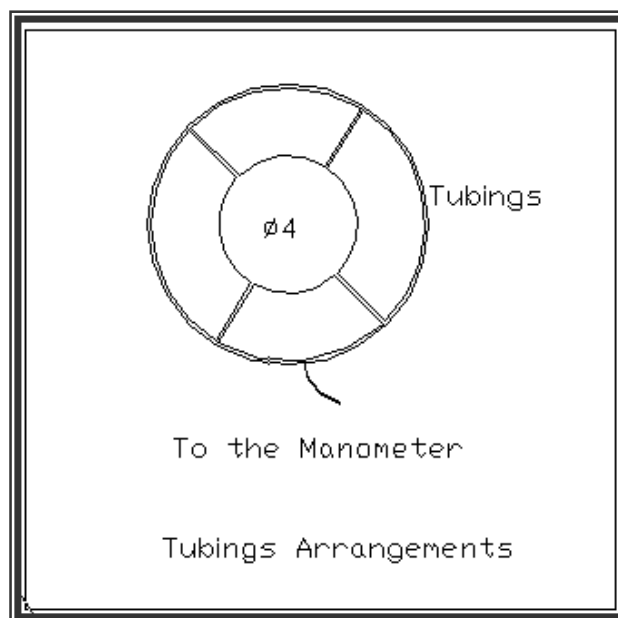


Fig (6.7a) Tubing ring



Fig (6.7b) Tubing ring

6.4.2.2 Feed pipe:

One pipe has diameter of 75 mm and length of 10m to transfer the pumped water from downstream tank to upstream tank

6.4.3 Gate valve:

There is one gate valve 75 mm through feed pipe to control the amount of flow of water.

6.4.4 Sediment feeder:

On top of the upstream tank there is the sediment feeder. The sediment feeder had a controlling gauge to control the rate of the feeded sediment see Fig (6.8a) & Fig (6.8b).

The sediment has been weighted using sensitive scale as shown in Fig (6.9).



Fig (6.8a) Sediment feeder

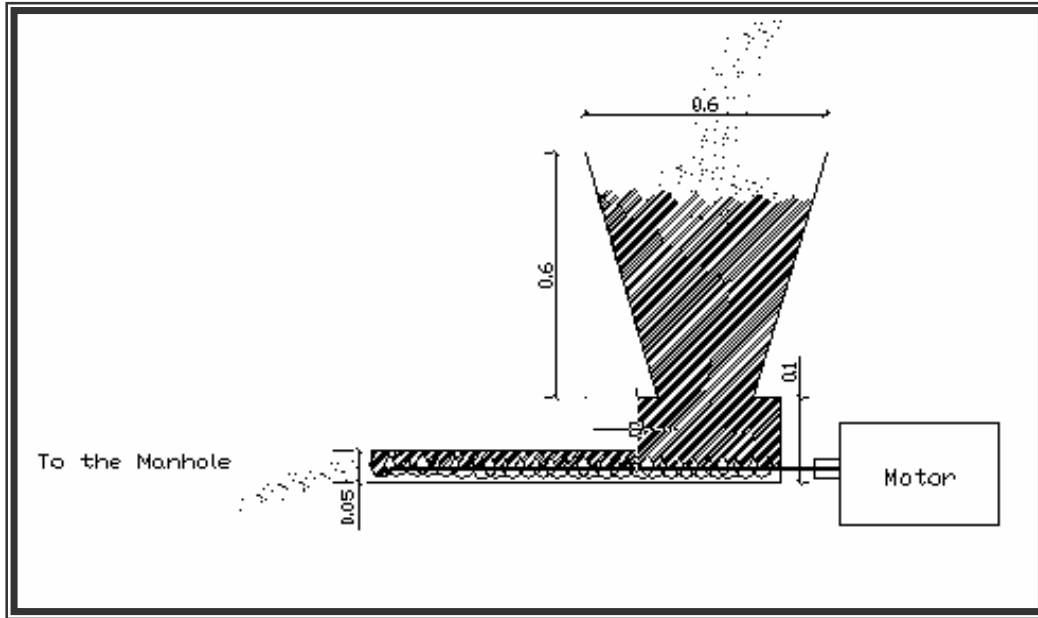


Fig (6.8b) Sediment feeder



Fig (6.9) Sensitive scale

6.4.5 Sediment trap:

Inside the downstream tank a sediment trap was placed. This is for collecting the sediments that passing through pipe. The sediment trap had fine mesh.

6.4.6 Manometer and pressure tappings:

Four pressure tappings were installed at the main pipe, each pressure tappings was established by two holes on the bottom and two holes on the upper halves of pipe. At the bottom of each ring a tube is attached to the manometer board as shown in Fig (6.10).

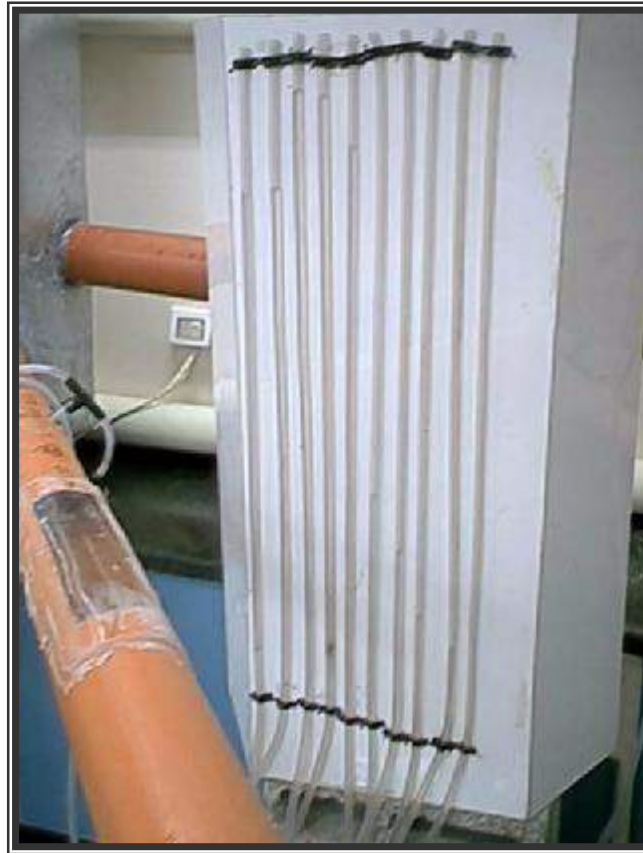


Fig (6.10) Manometer

6.4.7 Main Pump

This pump is used to transfer the water from downstream tank to upstream tank.

The main pump had flow and head as required:

Type: SAER BP-7/B

$V = 230$

HP = 3

$Q = 6 - 63 \text{ m}^3/\text{hr}$

$H = 19.4 - 8.5 \text{ m}$

The pump and system curves shown in Fig (6.11)

The calculation sheets for the required pump is shown in Appendix B

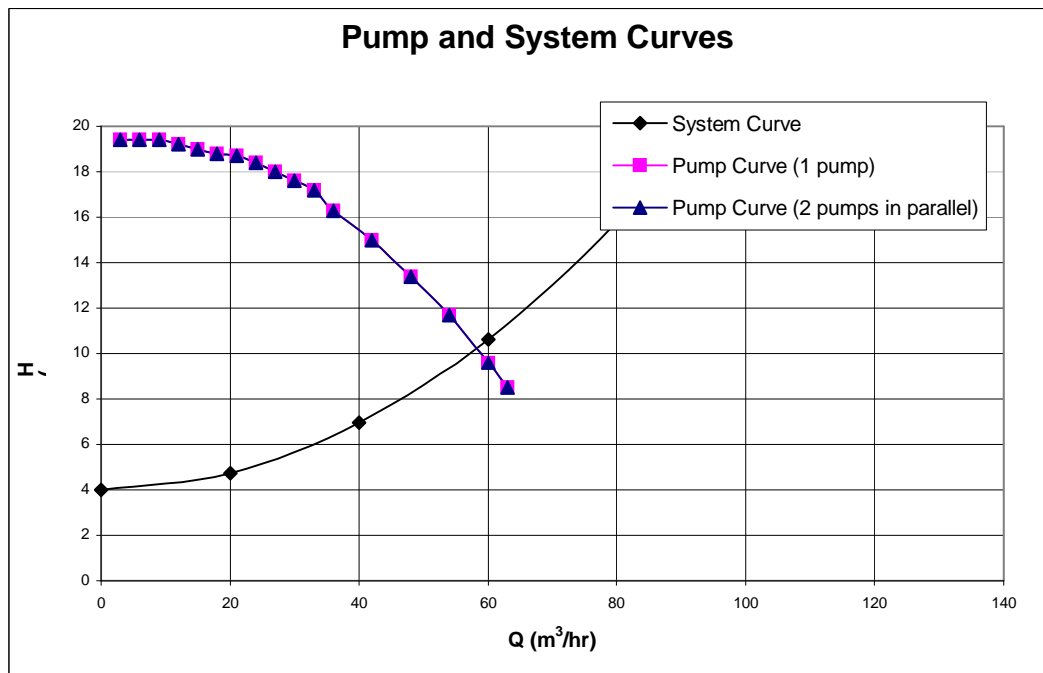


Fig (6.11) Pump and System Curves

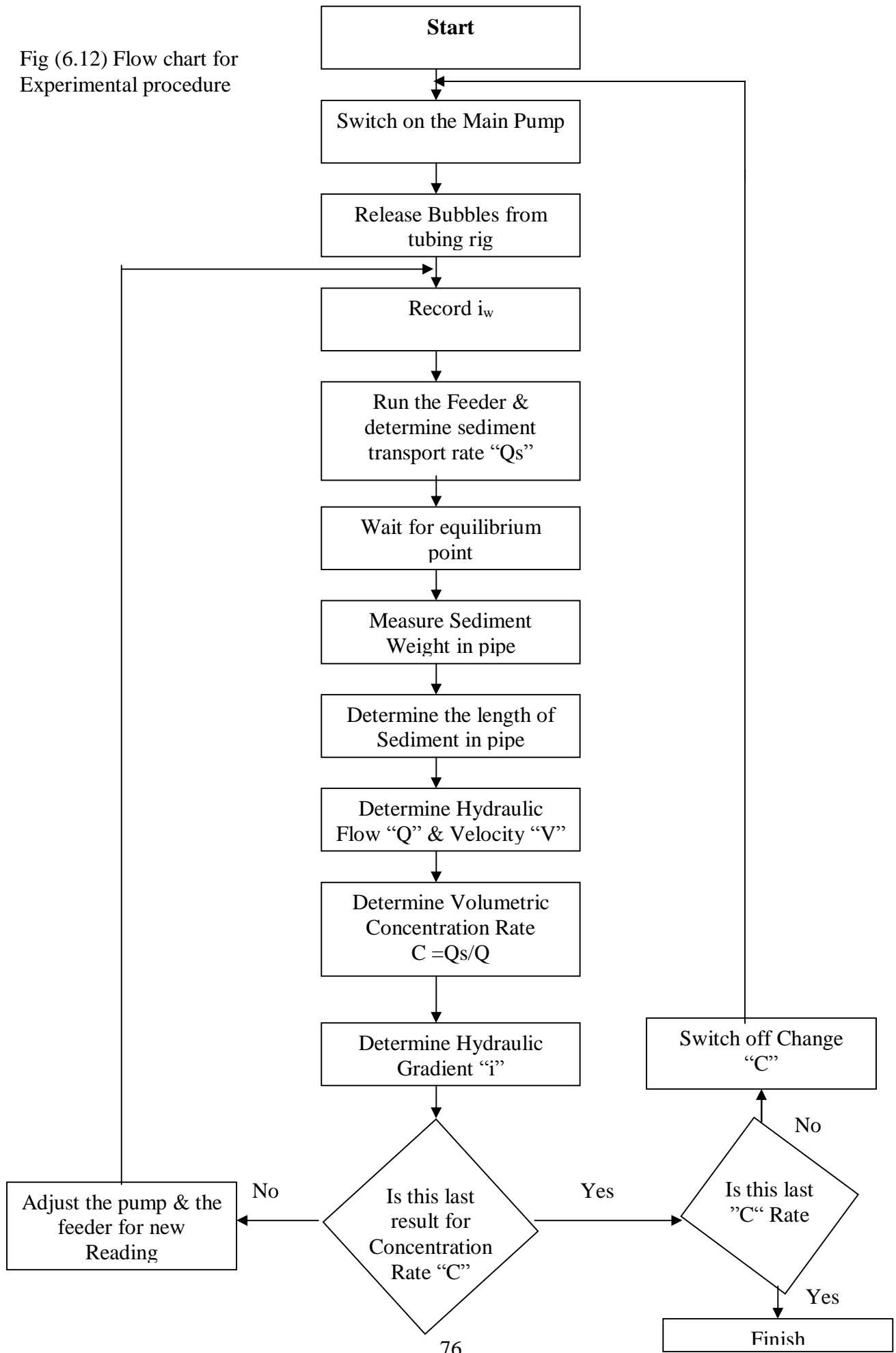
6.5 Experimental Program:

6.5.1 Measurement Procedure:

1. The main pump is switched on.
2. The gate valve is controlled to achieve the full flow.
3. The tubing ring was prepared to release the bubbles from the manometer.
4. The sediment feeder was filled by weighted sand and adjusted by the gauge to give the required rate of discharge of sediment and then it was turned on to supply the pipe with required rate of sediment. The sediment discharge rate from the feeder was taken by the use of stopwatch and a flask.
5. Then, until the equilibrium stage occurred readings from the manometer were taken to get the head loss for water –sediment suspension.
6. The flow discharge and flow velocity were calculated using stopwatch and small tank.
7. After that, the sediment was emptied and the unused sediment was weighted. And the sediment, which was settled in manhole, was weighted then the sediment in pipe was calculated.
8. The steps from 1 to 7 were repeated.

Flow chart for Experimental procedure is shown in Fig (6.12)

Fig (6.12) Flow chart for Experimental procedure



Chapter 7

Analysis of Experimental Results

7.1 Introduction:

The proposed method represented the roughness parameter “K_{ss}” in “TP” which represents to overall roughness of the pipe.

Also the proposed method deal with high velocity of water flow and most of hydraulic parameter that affect the hydrodynamic forces in close drain, are included in the shear parameters “SP” and Transport parameter “ TP”.

7.2 Geometrical Relationships:

Consider the pipe section shown below with the following terms:

Ab	Cross sectional area of the deposited bed
Af	Cross sectional area occupied by the water
Pb	Width of the bed interface
Ps	Perimeter of the pipe in contact with the bed
hb	Depth of the deposited bed
γ	The angle subtended by Pb, in radians
Pw	Perimeter of the pipe in contact with the flow

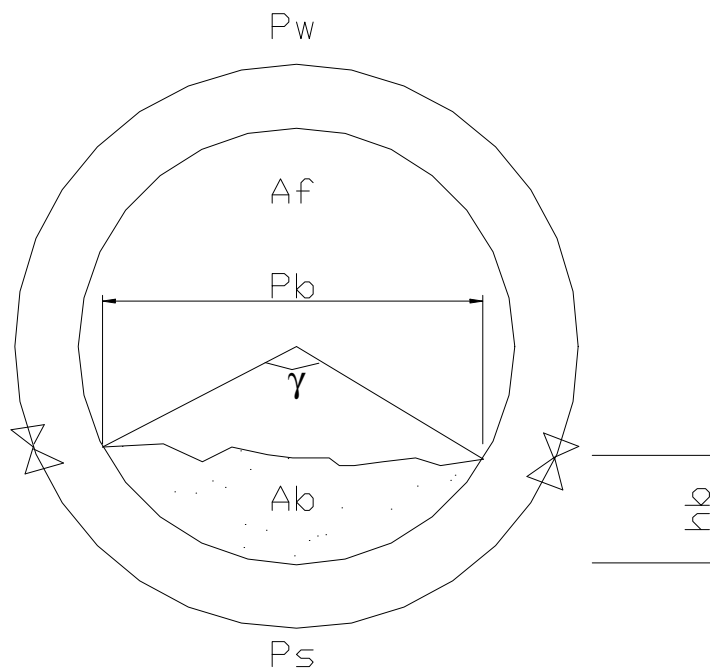


Figure (7.1) Schematic Diagram

The geometrical relationships of quantities shown in Figure (7.1) are as the following.

$$\frac{A_b}{A} = 1/2\pi(\gamma - \sin \gamma)$$

$$\gamma = 2 \cos^{-1}(1 - 2hb/D)$$

γ The angle subtended by P_b , in radians

$$\frac{A_f}{A} = 1 - \frac{A_b}{A}$$

$$\frac{P_s}{P} = \frac{\gamma}{2\pi}$$

$$\frac{D_{Ef}}{D} = \frac{[\gamma - \sin \gamma]}{\gamma + 2 \sin(\gamma/2)}$$

$$R = D/4$$

$$\frac{Pb}{D} = 2\sqrt{hb/D - (hb/D)^2}$$

$$h_b/D = 0.786(Ab/A)^3 - 1.184(Ab/A)^2 + 1.385(Ab/A) + 0.0035$$

$$Ab/A = -0.86(hb/D)^3 + 1.286(hb/D)^2 + 0.579(hb/D) - 0.0015$$

7.2.1 Hydraulic parameters calculations:

Hydraulic Radius

$$R_b = D_{Eb}/4 \quad \text{for area occupied by the sediment}$$

$$R = D_{Ef}/4 \quad \text{for area occupied by the fluid}$$

hb	D	hb/D	Ab/A	Af/A	Pb/D	γ	Pw/P	Ps/P	DEb/D	DEf/D
0	0.1	0	0	1	0	0	1	0	0	1
0.01	0.1	0.1	0.0684	0.9316	0.6	1.287002	0.795063	0.204937	0.13148449	0.961247
0.02	0.1	0.2	0.15886	0.84114	0.8	1.85459	0.704683	0.295317	0.25895702	0.893783
0.03	0.1	0.3	0.26472	0.73528	0.916515	2.318559	0.630803	0.369197	0.38186505	0.810195
0.04	0.1	0.4	0.38082	0.61918	0.979796	2.738877	0.563873	0.436127	0.49951562	0.715006
0.05	0.1	0.5	0.502	0.498	1	3.141593	0.499746	0.500254	0.61101547	0.610774
0.06	0.1	0.6	0.6231	0.3769	0.979796	3.544308	0.43562	0.56438	0.71517045	0.499176
0.08	0.1	0.8	0.84442	0.15558	0.8	4.428595	0.29481	0.70519	0.89383928	0.258273
0.09	0.1	0.9	0.93432	0.06568	0.6	4.996183	0.204429	0.795571	0.96126648	0.130371
0.1	0.1	1	1.0035	-0.0035	0	6.283185	0	1	1	0

Table (7.1) Geometrical Relationship

7.2.2 Volumetric concentration:

$$C = Q_s/Q$$

$$C = Q_s/Q \cdot 10^6 \quad \text{part per million (ppm)}$$

The geometrical relationships are for constant pipe diameter shown in table 7.1 and Figure (D.1) to Figure (D.8) in Appendix D .

7.2.3 Cross sectional area of the bed

$$A_b = (W_s / P_s) / L_s$$

A_b = Cross sectional area of the bed

W_s = weight of the sediment fed in to the pipe

L_s = the length of the sediment inside the pipe

A_b shown in Fig(7.1)

7.2.4 Hydraulic Gradient “i”

The hydraulic gradients were obtained after analysis of data.

7.2.5 Roughness parameter “Kss”

All the experimental data in case of deposited bed has been used in the corresponding values of “Kss” instead of “Ks”, then those values were applied to *Colebrook White Equation* for roughness of pipe.

$$K_{ss} = \frac{P_w K_s + P_b d_{50}}{P_w + P_b}.$$

K_{ss} = total roughness

K_s = Surface roughness of Pipe (see table 7.2)

P_w = wet perimeter of pipe

P_b = width of sediment

	Roughness “Ks” (mm)	
	New	Cleaned
Vertically casted iron	0.09	0.2
UPVC	0.04	0.1
Sspun concrete	0.09	0.2
Asbestos-cement	0.02	0.07
clayware	0.07	0.2

Table (7.2) Surface Roughness “Ks” (mm)

7.3 Calculation of sediments load (Sample).

The procedure of Calculating the weight of sediment in pipe is shown in table (7.3).

	Procedure of Calculation the weight of sediment in pipe	
A	Total weight of sediments in sediment feeder	3000 g
B	Unused sediment	1076 g
C	Weight of used sediment = (A-C)	1924 g
D	Dry weight	1354 g
E	Weight of sediment settled in manhole	550 g
F	Weight of sediment in pipe = (C-D-E)	70 g

Table (7.3) Procedure of Calculating the weight of sediment in pipe

Cross sectional area of the bed

The Procedure of Calculating the parameters are shown in the following example.

$$\frac{A_b}{A} = 1/2\pi(\gamma - \sin \gamma)$$

$$\gamma = 2 \cos^{-1}(1 - 2hb/D)$$

γ The angle subtended by P_b , in radians

$$hb = 0.001$$

$$A = \frac{0.1^2 \pi}{4} = 0.00785 \text{ m}^2$$

so,

$$\gamma = 2 \cos^{-1}(1 - 2 \times 0.001/0.1) = 0.4$$

$$\frac{A_b}{A} = 1/2\pi(0.4 - \sin 0.4) = 0.0067$$

$$A_b = 0.0067 \times A = 0.0067 \times 0.00785 = 0.0000526 \text{ m}^2$$

$$\frac{A_f}{A} = 1 - \frac{A_b}{A}$$

$$\frac{A_f}{A} = 1 - 0.0067 = 0.9933$$

$$A_f = 0.9933 \times 0.00785 = 0.00780 \text{ m}^2$$

$$\frac{P_s}{P} = \frac{\gamma}{2\pi} = \frac{0.4}{2\pi} = 0.0637$$

$$P_s = 0.0637 \times \pi \times 0.1 = 0.02 \text{ m}$$

$$P_w = \pi \times 0.1 - 0.02 = 0.294 \text{ m}$$

$$\frac{D_{Ef}}{D} = \frac{[\gamma - \sin \gamma]}{\gamma + 2 \sin(\gamma/2)}$$

$$R = D/4$$

$$R = 0.1/4 = 0.025$$

$$\frac{P_b}{D} = 2\sqrt{hb/D - (hb/D)^2}$$

$$\frac{P_b}{D} = 2\sqrt{0.001/0.1 - (0.001/0.1)^2}$$

$$\frac{P_b}{D} = 0.199$$

$$P_b = 0.199 \times 0.1 = 0.0199$$

$$K_{SS} = \frac{P_w K_s + P_b d_{50}}{P_w + P_b}$$

$$K_{SS} = \frac{0.294 \times 0.04 + 0.0199 \times 0.3}{0.294 + 0.0199} = 0.0564$$

$$h_b/D = 0.786(A_b/A)^3 - 1.184(A_b/A)^2 + 1.385(A_b/A) + 0.0035$$

$$A_b = (W_s/P_s)/L_s$$

$$A_b = (0.07/2570)/2.5 = 1.089\text{E-}05$$

$$h_b = (0.786(A_b/A)^3 - 1.184(A_b/A)^2 + 1.385(A_b/A) + 0.0035) \times D$$

$$h_b = (0.786\left(\frac{1.089\text{E-}05}{0.00785}\right)^3 - 1.184\left(\frac{1.089\text{E-}05}{0.00785}\right)^2 + 1.385\left(\frac{1.089\text{E-}05}{0.00785}\right) + 0.0035) \times 0.1$$

$$= 0.000542 \text{ m}$$

Then from manometer readings the head losses and the hydraulic gradient can be calculated.

The diagram for head losses calculation is shown in Fig (7.10).

h_f = head losses for intervals through discharge pipe.

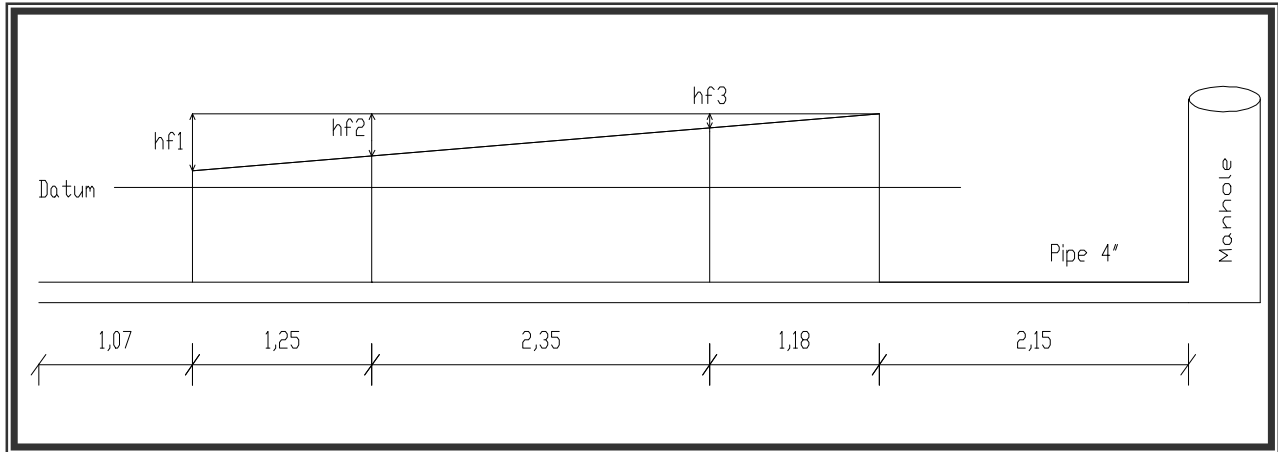


Fig (7.2) The diagram for head losses calculation

7.4 Sediment flow rate cases:

There are three cases for sediment flow rate from sediment feeder. The hydraulic effects on flow have been studied according to these cases. The sediment flow rate cases are shown in Table (7.4).

	height (m)	diameter (m)	Volume (m ³)	Time (sec)	Weight (gram)	Sediment flow Q _s (m ³ /sec)
Case 1	0.202	0.038	0.000229	12.8	350	1.78887E-05
	0.205	0.038	0.0002324	12.8	358	1.81544E-05
	0.204	0.038	0.0002312	12.7	346	1.8208E-05
					Av.	1.80837E-05
Case 2	0.17	0.038	0.0001927	14.2	385	1.35705E-05
	0.185	0.038	0.0002097	12.4	338	1.69117E-05
	0.178	0.038	0.0002018	12.1	324	1.66752E-05
					Av.	1.57192E-05
Case 3	0.12	0.038	0.000136	13.6	366	1.00018E-05
	0.13	0.038	0.0001474	12.4	336	1.18839E-05
	0.14	0.038	0.0001587	12.5	334	1.26956E-05
					Av.	1.15271E-05

Table (7.4) Sediment flow rate cases

7.5 Factors for derived formula.

The derived formula has been applied to all experimental data because the transport parameter T contains the roughness parameter “ K_s ” which is a direct measure of the overall roughness of the pipe with and without sediment and because the hydraulic gradient “ i ” in the shear parameter is a function of the roughness of the pipe.

The friction factor “ f ” in the Darcy Weisbach formula is a function of the pipe roughness and Reynold number, i.e.

$$i = \frac{f V^2}{2 g D}$$

and

$$f = \phi(V, 4R/v, K_s/R)$$

So according to the above function the hydraulic gradient can represent the type of the pipe and the sediment transport conditions.

To calculate the values of the Hydraulic parameter (K) and Transport parameter (T) a computer program has been prepared for each set of data and the following equation was obtained by applying regression analysis:

$$T = 0.0561 \times K^{3.54}$$

The relationship for the above equation is shown in Fig (7.11).

The list of experimental results is shown in table (7.5)

Table (7.5) the list of experimental results

Exp	Ws (g)	Ls (m)	Qs (m ³ /sec)	Q (m ³ /sec)	C	V (m/ sec)	Viscosity	T°	Ks	s	D	d50
Case # 1												
1	70	2.5	1.808E-05	0.0129	1.402E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
2	53	3	1.808E-05	0.0129	1.402E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
3	231	3	1.808E-05	0.0129	1.402E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
4	253	3	1.808E-05	0.0129	1.402E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
5	243	2	1.808E-05	0.0129	1.402E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
6	189	2	1.808E-05	0.0129	1.402E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
7	96	2	1.808E-05	0.0129	1.402E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
8	157	2.5	1.808E-05	0.0129	1.402E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
9	105	3	1.808E-05	0.0129	1.402E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
Case # 2												
1	106	3	1.572E-05	0.0129	1.219E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
2	153	2	1.572E-05	0.0129	1.219E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
3	98	4	1.572E-05	0.0129	1.219E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
4	201	4	1.572E-05	0.0129	1.219E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
5	96	3	1.572E-05	0.0129	1.219E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
6	85	2.5	1.572E-05	0.0129	1.219E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
7	77	3	1.572E-05	0.0129	1.219E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
8	178	2	1.572E-05	0.0129	1.219E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
9	133	2	1.572E-05	0.0129	1.219E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
10	104	3	1.572E-05	0.0129	1.219E-03	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
Case # 3												
1	78	2	1.15E-05	0.0129	8.930E-04	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
2	0	0	1.15E-05	0.0129	8.930E-04	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
3	49	1	1.15E-05	0.0129	8.930E-04	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
4	0	0	1.15E-05	0.0129	8.930E-04	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
5	0	0	1.15E-05	0.0129	8.930E-04	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
5	55	2	1.15E-05	0.0129	8.930E-04	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
6	0	0	1.15E-05	0.0129	8.930E-04	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
7	71	2	1.15E-05	0.0129	8.930E-04	1.65	1.00E-06	25	0.04	2.65	0.1	0.3
8	69	2	1.15E-05	0.0129	8.930E-04	1.65	1.00E-06	25	0.04	2.65	0.1	0.3

Ws = Weight of sediment in pipe (g) V = flow velocity (m/sec) d50 = grain diameter at 50% Passing

Ls = Length of sediment in pipe T = Temperature c°

Qs = sediment transport rate (m³/sec) Ks = pipe wall roughness

Q = flow rate (m³/sec) s = specific gravity

C = Concentration rate (m/m) D = Pipe diameter (m)

Exp	A	Ab	hb	P	γ	Ps	Pw	Pb	Kss	R	Z
Case # 1											
1	0.00785	1.08949E-05	5.42E-04	0.314	0.4006	0.02003	0.294	0.015	0.052	0.025	3.849
2	0.00785	6.87419E-06	4.71E-04	0.314	0.696	0.0348	0.279	0.014	0.052	0.025	3.851
3	0.00785	2.99611E-05	8.77E-04	0.314	0.696	0.0348	0.279	0.019	0.056	0.025	3.818
4	0.00785	3.28145E-05	9.27E-04	0.314	0.567	0.02835	0.286	0.019	0.056	0.025	3.817
5	0.00785	4.72763E-05	1.18E-03	0.314	0.4006	0.02003	0.294	0.022	0.058	0.025	3.806
6	0.00785	3.67704E-05	9.96E-04	0.314	0.567	0.02835	0.286	0.020	0.057	0.025	3.813
7	0.00785	1.8677E-05	6.79E-04	0.314	0.567	0.02835	0.286	0.016	0.054	0.025	3.835
8	0.00785	2.44358E-05	7.80E-04	0.314	0.4006	0.02003	0.294	0.018	0.055	0.025	3.830
9	0.00785	1.36187E-05	5.90E-04	0.314	0.567	0.02835	0.286	0.015	0.053	0.025	3.842
Case # 2											
1	0.00785	1.37484E-05	5.92E-04	0.314	0.4006	0.02003	0.294	0.015	0.053	0.025	3.845
2	0.00785	2.97665E-05	8.73E-04	0.314	0	0	0.314	0.019	0.055	0.025	3.831
3	0.00785	9.53307E-06	5.18E-04	0.314	0.4006	0.02003	0.294	0.014	0.052	0.025	3.851
4	0.00785	1.95525E-05	6.94E-04	0.314	0.567	0.02835	0.286	0.017	0.054	0.025	3.834
5	0.00785	1.24514E-05	5.69E-04	0.314	0.401	0.02005	0.294	0.015	0.053	0.025	3.847
6	0.00785	1.32296E-05	5.83E-04	0.314	0.4006	0.02003	0.294	0.015	0.053	0.025	3.846
7	0.00785	9.98703E-06	5.26E-04	0.314	0.567	0.02835	0.286	0.014	0.053	0.025	3.848
8	0.00785	3.46304E-05	9.59E-04	0.314	0.4006	0.02003	0.294	0.019	0.056	0.025	3.819
9	0.00785	2.58755E-05	8.05E-04	0.314	0.4006	0.02003	0.294	0.018	0.055	0.025	3.829
10	0.00785	1.3489E-05	5.88E-04	0.314	0.401	0.02005	0.294	0.015	0.053	0.025	3.845
Case # 3											
1	0.00785	1.51751E-05	6.17E-04	0.314	0.401	0.02005	0.294	0.016	0.053	0.025	3.843
2	0.00785	0	0.00E+00	0.314	0	0	0.314	0.000	0.040	0.025	3.966
3	0.00785	1.90661E-05	6.86E-04	0.314	0.401	0.02005	0.294	0.017	0.054	0.025	3.837
4	0.00785	0	0.00E+00	0.314	0	0	0.314	0.000	0.040	0.025	3.966
5	0.00785	0	0.00E+00	0.314	0	0	0.314	0.000	0.040	0.025	3.966
5	0.00785	1.07004E-05	5.39E-04	0.314	0.401	0.02005	0.294	0.015	0.052	0.025	3.849
6	0.00785	0	0.00E+00	0.314	0	0	0.314	0.000	0.040	0.025	3.966
7	0.00785	1.38132E-05	5.93E-04	0.314	0.401	0.02005	0.294	0.015	0.053	0.025	3.845
8	0.00785	1.34241E-05	5.87E-04	0.314	0.401	0.02005	0.294	0.015	0.053	0.025	3.845

A = Cross sectional area

Ab = cross sectional area of the deposited bed

hb = Depth of the deposited bed

P = the perimeter of the pipe

γ = the angle subtended by Pb

Ps = Perimeter of the pipe in contact with the bed

Pw = Perimeter of the pipe in contact with the flow

Pb = width of the bed interface

Kss = Overall roughness parameter

R = Hydraulic radius

Exp	Manometer Readings				hf 3	hf 2	hf 1	i3	i2	i1	i
	1	2	3	4							
Case # 1											
1	0.362	0.275	0.23	0.085	0.087	0.132	0.277	0.074	0.019	0.116	6.96E-02
2	0.315	0.273	0.24	0.13	0.042	0.075	0.185	0.036	0.014	0.088	4.588E-02
3	0.315	0.275	0.245	0.13	0.040	0.070	0.185	0.034	0.013	0.092	4.622E-02
4	0.325	0.275	0.235	0.165	0.050	0.090	0.160	0.042	0.017	0.056	3.846E-02
5	0.325	0.275	0.245	0.13	0.050	0.080	0.195	0.042	0.013	0.092	4.905E-02
6	0.320	0.275	0.24	0.13	0.045	0.080	0.190	0.038	0.015	0.088	4.701E-02
7	0.320	0.27	0.24	0.13	0.050	0.080	0.190	0.042	0.013	0.088	4.771E-02
8	0.335	0.275	0.242	0.115	0.060	0.093	0.220	0.051	0.014	0.102	5.550E-02
9	0.325	0.28	0.23	0.08	0.045	0.095	0.245	0.038	0.021	0.120	5.980E-02
Average											
1	0.320	0.27	0.19	0.115	0.050	0.130	0.205	0.042	0.034	0.060	4.547E-02
2	0.320	0.27	0.185	0.11	0.050	0.135	0.210	0.042	0.036	0.060	4.618E-02
3	0.315	0.275	0.22	0.109	0.040	0.095	0.206	0.034	0.023	0.089	4.870E-02
4	0.325	0.285	0.16	0.115	0.04	0.165	0.210	0.034	0.053	0.036	4.103E-02
5	0.315	0.27	0.18	0.13	0.045	0.135	0.185	0.038	0.038	0.040	3.881E-02
6	0.325	0.278	0.175	0.125	0.047	0.150	0.200	0.040	0.044	0.040	4.122E-02
7	0.325	0.27	0.18	0.13	0.055	0.145	0.195	0.047	0.038	0.040	4.164E-02
8	0.310	0.27	0.19	0.125	0.04	0.120	0.185	0.034	0.034	0.052	3.998E-02
9	0.315	0.275	0.225	0.13	0.04	0.090	0.185	0.034	0.021	0.076	4.372E-02
10	0.310	0.27	0.2	0.12	0.04	0.110	0.190	0.034	0.030	0.064	4.256E-02
Average											
1	0.315	0.275	0.195	0.13	0.04	0.120	0.185	0.034	0.034	0.052	3.998E-02
2	0.310	0.275	0.21	0.14	0.035	0.100	0.170	0.030	0.028	0.056	3.777E-02
3	0.320	0.275	0.2	0.13	0.045	0.120	0.190	0.038	0.032	0.056	4.202E-02
4	0.310	0.265	0.175	0.125	0.045	0.135	0.185	0.038	0.038	0.040	3.881E-02
5	0.310	0.27	0.18	0.13	0.04	0.130	0.180	0.034	0.038	0.040	3.740E-02
5	0.320	0.27	0.175	0.135	0.05	0.145	0.185	0.042	0.040	0.032	3.827E-02
6	0.310	0.275	0.18	0.125	0.035	0.130	0.185	0.030	0.040	0.044	3.803E-02
7	0.305	0.27	0.185	0.125	0.035	0.120	0.180	0.030	0.036	0.048	3.794E-02
8	0.305	0.27	0.185	0.115	0.035	0.120	0.190	0.030	0.036	0.056	4.061E-02

hf = head losses

i = hydraulic gradient for each manometer reading

I = Total hydraulic gradients

	Exp.	K	T
Case # 1			
	1	83.51	192575.78
	2	67.82	192575.78
	3	67.49	192575.78
	4	61.56	192575.78
	5	69.31	192575.78
	6	67.98	192575.78
	7	68.87	192575.78
	8	74.19	192575.78
	9	77.25	192575.78
	Average	70.89	192575.78
Case # 2	1	67.41	167393.77
	2	67.70	167393.77
	3	69.88	167393.77
	4	63.85	167393.77
	5	62.31	167393.77
	6	64.20	167393.77
	7	64.56	167393.77
	8	62.78	167393.77
	9	65.83	167393.77
	10	65.22	167393.77
	Average	65.37	167393.77
Case # 3	1	63.18	122678.05
	2	63.38	122678.05
	3	64.67	122678.05
	4	64.24	122678.05
	5	63.06	122678.05
	5	61.91	122678.05
	6	63.59	122678.05
	7	61.58	122678.05
	8	63.71	122678.05
	Average	63.26	122678.05

K = Hydraulic parameter

T = Transport parameter

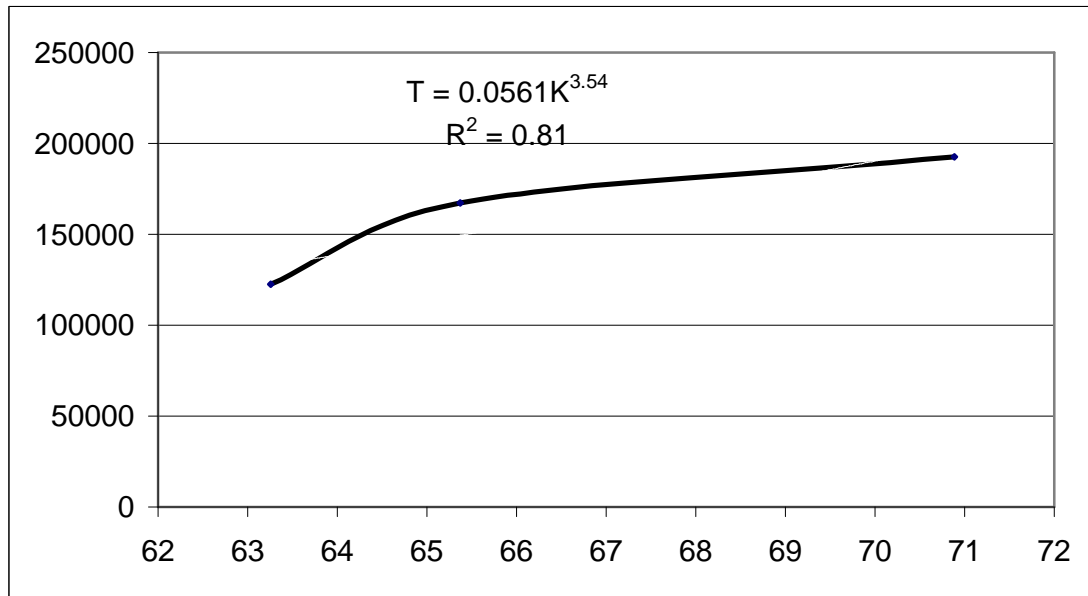


Fig (7.3) Relationship between the “K” Parameter and “T” parameter

7.6 Summary of coefficient for the proposed method

The proposed equation for high-speed velocity flow through storm sewer is as following equation.

$$T = 0.0561 \times K^{3.54}$$

$$\text{Transport Parameter } T = \frac{C \cdot d_{50} \cdot V^4}{v^2 \cdot (s-1)g}$$

$$\text{Shear Parameter } K = \left[\frac{i^{0.5} R_E^{1.5}}{v} \left(\frac{d_{50}}{D} \right)^{2/3} \text{Log} \left(\frac{14.8 R_E}{K_{ss}} \right) \right]$$

$$\frac{C \cdot d_{50} \cdot V^4}{v^2 \cdot (s-1)g} = 0.0561 \left[\frac{i^{0.5} R_E^{1.5}}{v} \left(\frac{d_{50}}{D} \right)^{2/3} \text{Log} \left(\frac{14.8 R_E}{K_{ss}} \right) \right]^{3.54}$$

Q_s = Sediment transport rate

Q = water transport rate

V = Velocity

n = Viscosity

s = Specific gravity of sediment

g = gravity

i = gradient

D = Pipe diameter

d_{50} = grain diameter at 50% Passing

7.6.1 Limitation of the proposed method:

There are some limitations of the proposed method as the following:

- 1- This proposed method is for high flow velocities more than 1.65m/sec for full flow.
- 2- The minimum grain diameter at 50% Passing is 0.3mm
- 3- The proposed equation is preferable and recommended for design of pipes with deposited bed because it is more accurate and principally derived for pipes, unlike the others which were derived originally for open channels.
- 4- The proposed method is applicable where $d_{50} > K_s$.

7.7 Relationships of proposed method:

There are many relationships have been derived according to proposed equation.

- **The relationship between V & d50.**

Fig (7.12) shows a Relationship between d50 and velocity, the velocity calculated according to several sediment diameters.

Also it has been shown that the velocity required increasing the flow uniform when the sediment diameter increases.

$$V \propto d50$$

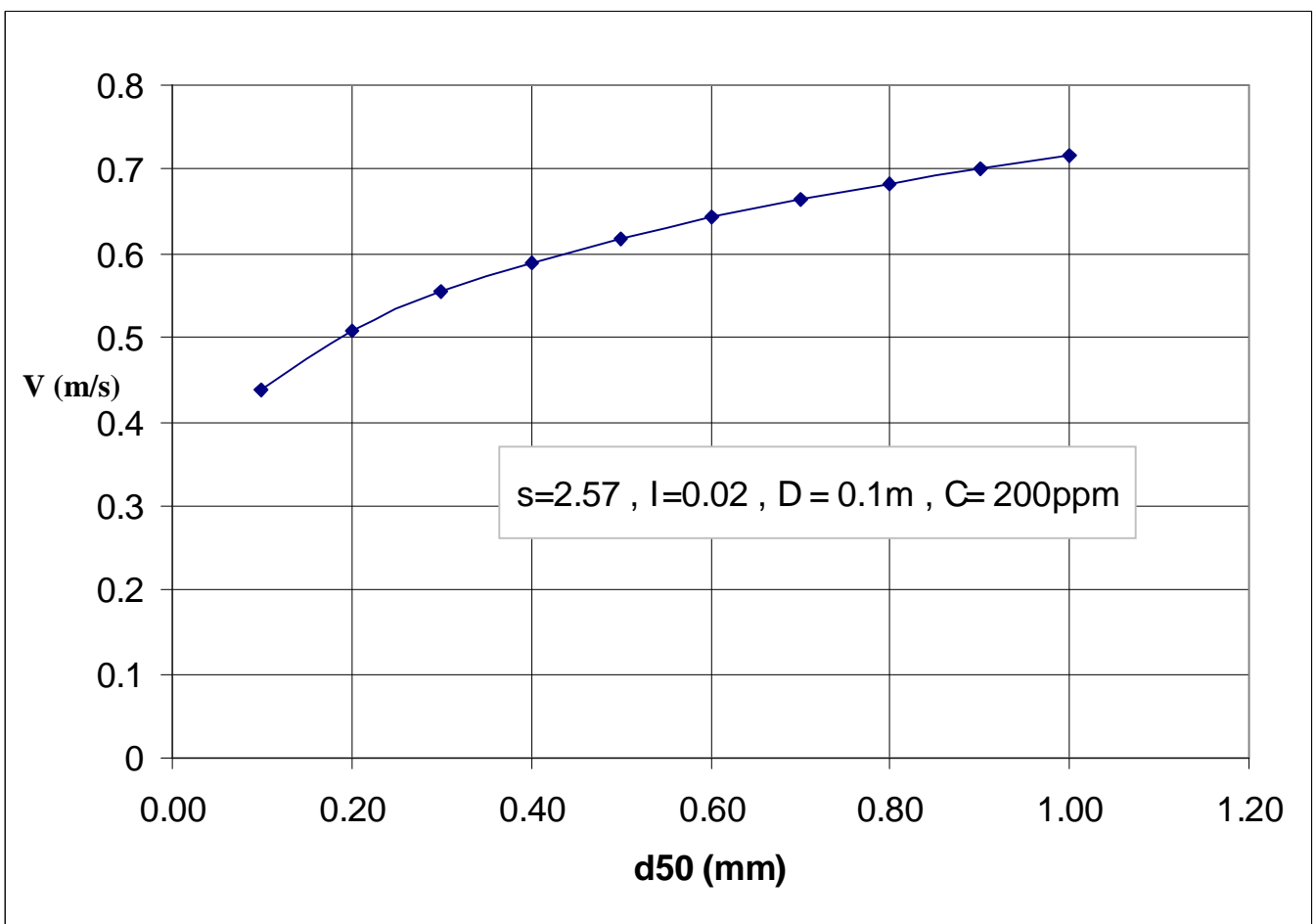


Fig (7.4) Relationship between d50 and velocity

- **The relationship between V & D.**

Fig (7.5) shows a Relationship between pipe diameter and velocity, when the diameter of pipe and the sediment diameter increase, the velocity required will increase.

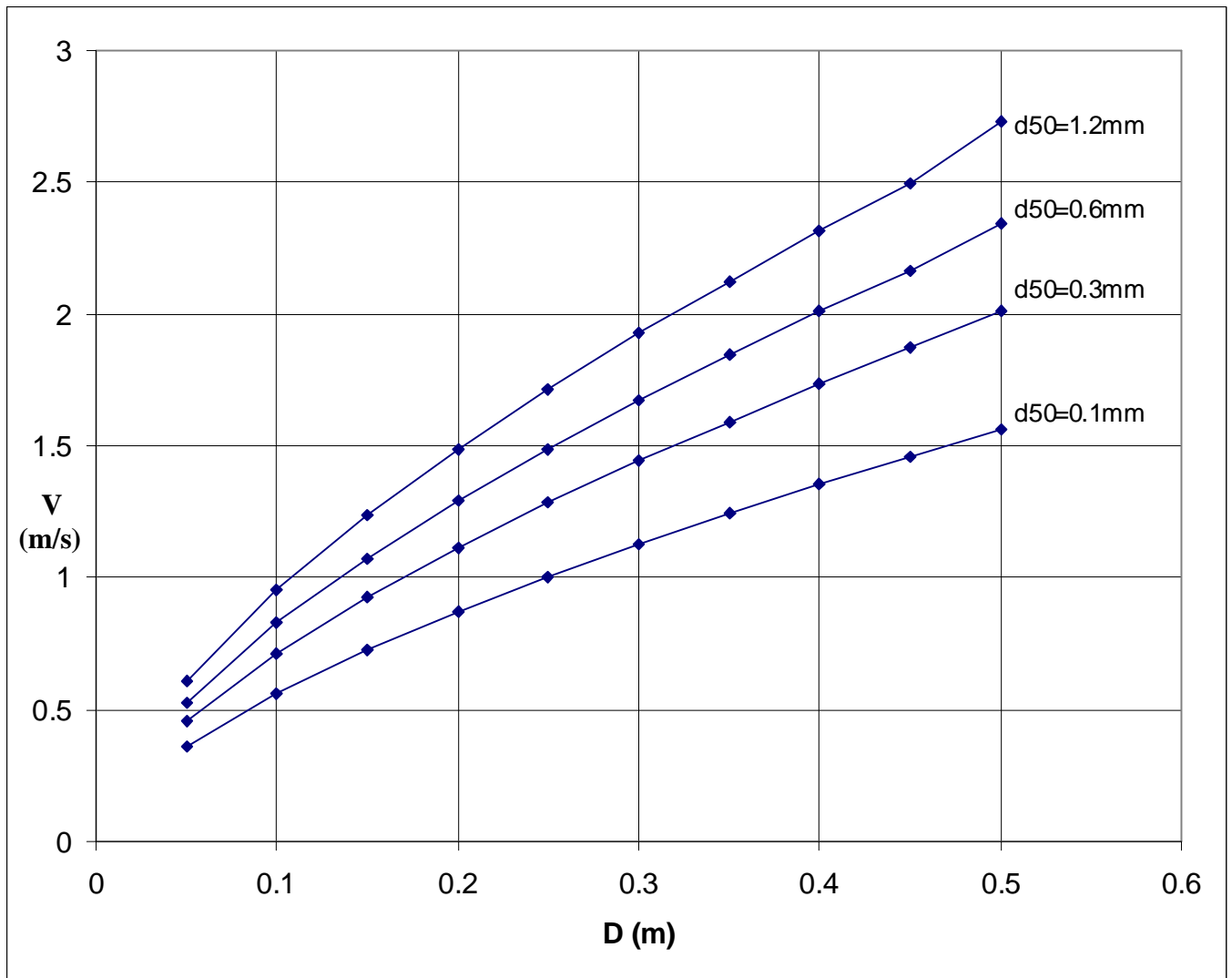


Fig (7.5) Relationship between pipe diameter and velocity

The plot for fig (7.5) are calculated for $d_{50} = 1.2\text{mm}$, 0.6mm , 0.3mm , 0.1mm
 $C=200\text{ppm}$ and $s = 2.57$

No codes of practice or methods are available (except Acker's for $D_{gr} < 60$) to describe the relationship between V and D so the proposed method gives a useful tool for predicting this relationship.

Sediment diameter is inversely related to the equilibrium velocity.

Sediment size d_{50} has two opposite effects on velocity:

- ▼ It behaves as a roughness of the bed which when increased raises the whole roughness k_{ss} which in turn increases the turbulence of the flow this leads to decrease in velocity required to transport the sediment i.e.

$$V \propto \frac{1}{d_{50}}$$

- ▼ When d_{50} increases the settling velocity increases which means a decrease in the rate of transport of sediment which lead to an increase velocity to maintain the equilibrium i.e. $V \propto d_{50}$

Comparison of Relationship between pipe diameter and velocity for proposed method and manning equation.

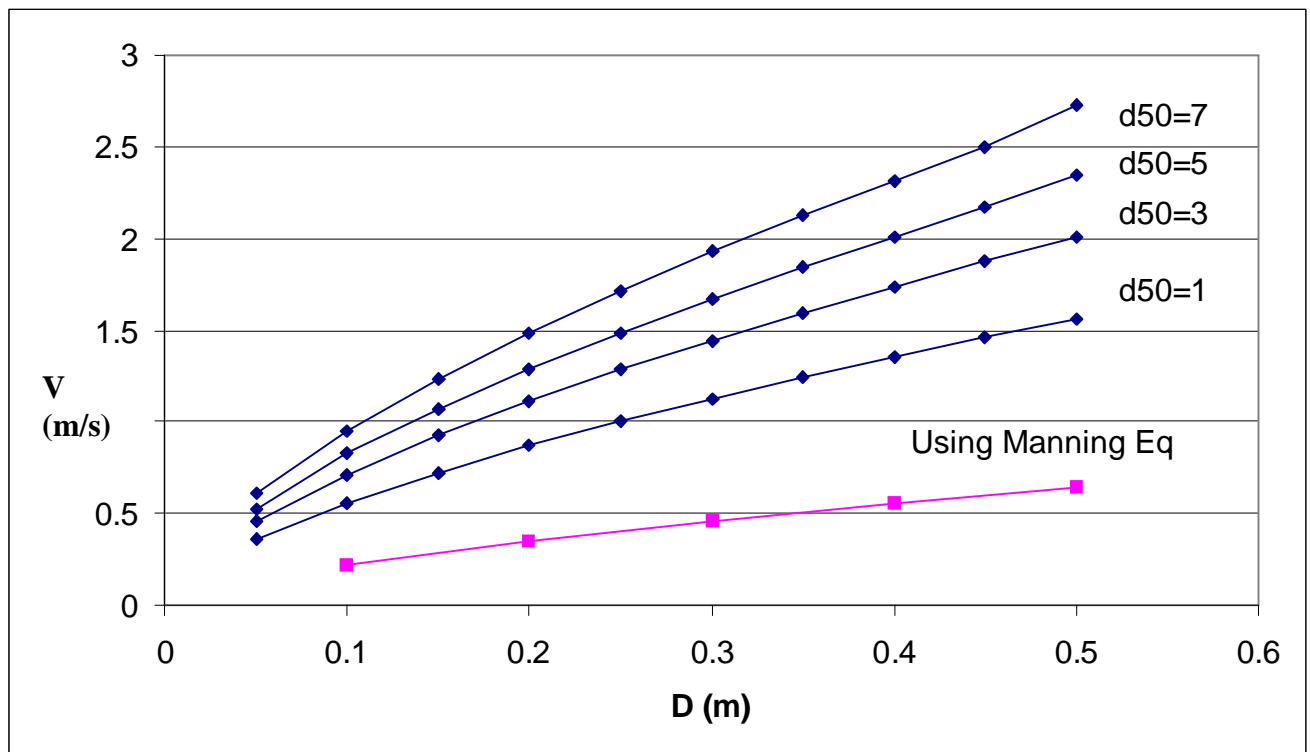


Fig (7.6) Relationship between pipe diameter and velocity for proposed method and manning equation.

- **The relationship between V & sediment concentration.**

Fig (7.14) shows inverse relationship between concentration and velocity, this is due to that when the sediment entrance into the pipe increase, the sediment depth will increase then the diameter of pipe will decrease since the velocity will increase.

$$V \propto 1/C \propto 1/hb$$

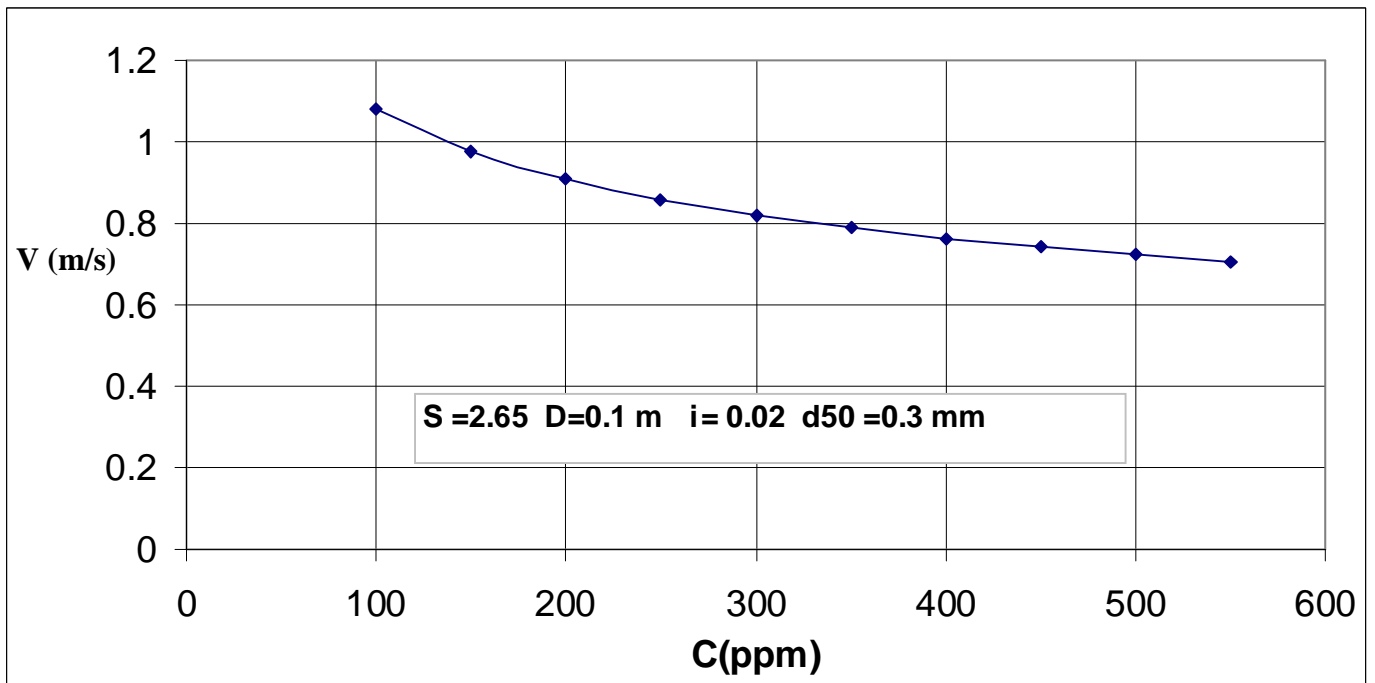


Fig (7.7) Relationship between concentration and velocity

The relationship between d50 and hydraulic gradient

Fig (7.15) shows inverse relationship between d50 and hydraulic gradient, where the increase in sediment diameter the hydraulic gradient will decrease.

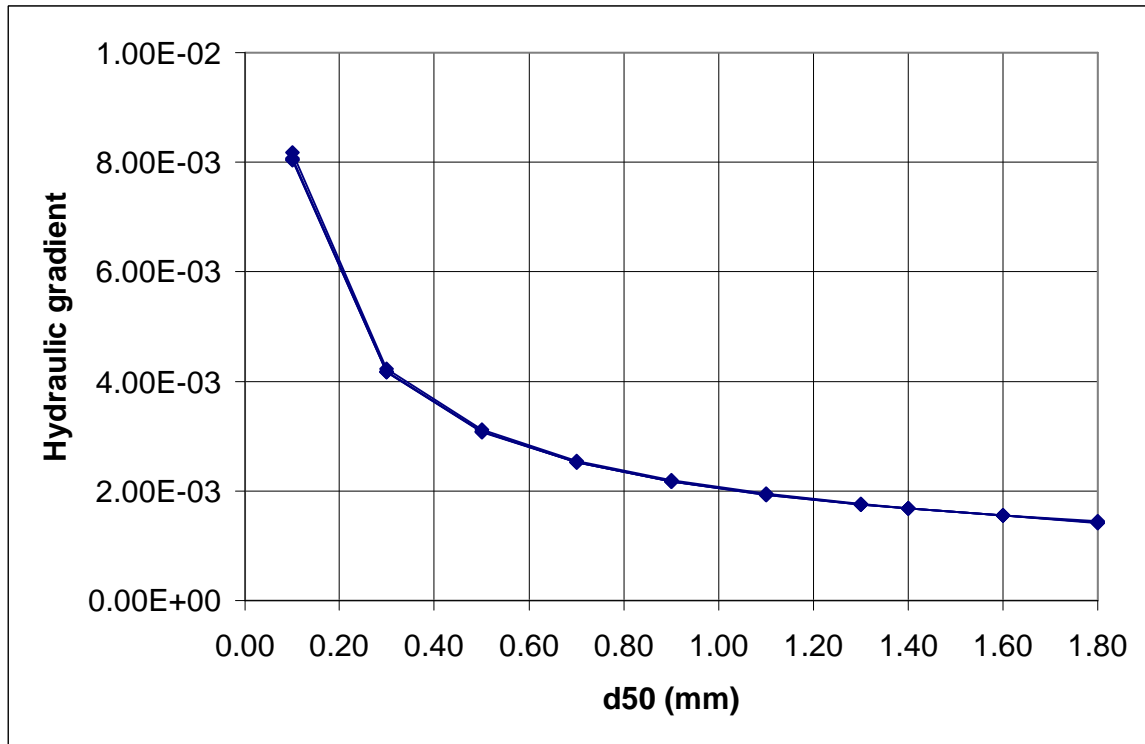


Fig (7.8) Relationship between d50 and hydraulic gradient

From above figure the following remarks can be brought out:

- ✓ The required hydraulic gradient is directly related to the sediment size this is due to the fact that any increase in d50 will increase the resistance.
- ✓ The bed depth is indirectly related to hydraulic gradient.

The bed depth has two opposite effects on “ i “

§ Roughness effect:

$$hb \propto Pb \propto K_{ss} \propto 1/V \propto i_1$$

§ Continuity effect

$$hb \propto 1/Af \propto V \propto 1/i_2$$

So the actual hydraulic gradient will be $i = i_2 - i_1$

For $d_{50} < K_s$ the proposed equation should not be applied because it might give misleading answers the proposed was derived for $d_{50} > K_s$.

Chapter 8

Conclusions & Recommendations

8.1 Conclusions

As a result of the experiments carried out during this dissertation, a number of conclusions can be made.

- 1- Hydrodynamic forces that exist through sewer have been derived using theoretical method.

- 2- The analysis of the results for high-speed velocity flow through storm sewer gave the following design formula.

$$T = 0.0561 \times K^{3.54}$$

and

$$\frac{C.d_{50}V^4}{v^2.(s-1)g} = 0.0561 \left[\frac{i^{0.5}R_E^{1.5}}{v} \left(\frac{d_{50}}{D} \right)^{2/3} \text{Log} \left(\frac{14.8R_E}{K_{ss}} \right) \right]^{3.54}$$

- 3- In this research there are many relationships have been applied according to proposed equation as following:
- It has been shown that the velocity required letting the flow uniform increase when the sediment diameter increases. ($V \propto d_{50}$)
 - When the diameter of pipe and the sediment diameter increase, the velocity required will increase.
 - The sediment depth will increase then the diameter of pipe will decrease since the velocity will increase. ($V \propto 1/C \propto 1/h_b$)
 - The increase in sediment diameter the hydraulic gradient will decrease.
- 4- Two additional parameters have been used through experimental work for high-speed velocity flow. These parameters are the bed depth “ h_b ” and the overall roughness of the pipe K_{ss} .

8.2 Recommendations

Due to the investigations in this research the following recommendations are given.

- To study the effect of non- – uniform transport of sediment through sewer system
- Economical analysis approach is required to give the optimum pipe diameter, flow velocity, slope of the pipe and shape of the pipe.
- To study the effect of sewer shape on the hydraulic flow.
- To study the uniform flow in case of partial flow.

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Appendices

Appendix A

A-1 Design Example:

It is proposed to design storm sewers under the following conditions:

$$s = 2.65$$

$$C = 100\text{ppm}$$

$$K_s = 0.3\text{mm}$$

$$D = 200\text{mm}$$

$$d_{50} = 0.3\text{mm}$$

$$V = 1.65$$

Find the hydraulic gradient:

Answer:

$$T = 0.0561 \times K^{3.54}$$

$$\frac{C \cdot d_{50} V^4}{v^2 \cdot (s-1)g} = 0.0561 \left[\frac{i^{0.5} R_E^{1.5}}{v} \left(\frac{d_{50}}{D} \right)^{2/3} \text{Log} \left(\frac{14.8 R_E}{K_{ss}} \right) \right]^{3.54}$$

Q_s = Sediment transport rate

Q = water transport rate

V = Velocity

n = Viscosity

s = Specific gravity of sediment

g = gravity

i = gradient

D = Pipe diameter

d₅₀ = grain diameter at 50% Passing

$$\frac{100 \times 10^{-6} \times 0.3^{-3} \times 1.65^4}{(1 \times 10^{-6})^2 \cdot (2.65 - 1)9.81} = 0.0561 \left[\frac{i^{0.5} (0.2/4)^{1.5}}{1 \times 10^{-6}} \left(\frac{0.3 \times 10^{-3}}{0.2} \right)^{2/3} \text{Log} \left(\frac{14.8(0.2/4)}{0.3 \times 10^{-3}} \right) \right]^{3.54}$$

$$i = 3.376 \times 10^{-4}$$

A-2 Physical Modeling of Storm Sewer Junction Hydraulics

The hydraulic analysis through a manhole focuses on the calculation of the energy loss from the inflow pipes to the outflow pipe. Several determining factors affect the computation of the energy loss coefficient in the HGL methodology. These include :

- the manhole size relative to the outlet pipe diameter,
- the depth of flow in the manhole,
- the amount of discharge,
- the inflow pipe angle,
- the plunge height, the relative pipe diameter,
- the floor configuration.

$$\Delta E = (C_1 C_2 C_3 + C_4) w \frac{V_o^2}{2g}$$

Where:

ΔE = Energy loss for an inflow pipe, m.

C_1 = Coefficient related to relative manhole size.

C_2 = Coefficient related to water depth in the manhole.

C_3 = Coefficient related to lateral flow, lateral angle, and plunging flow.

C_4 = Coefficient related to relative pipe diameters.

w = Correction factor for benching.

$\frac{V_o^2}{2g}$ = Velocity head, m/s².

Relative manhole size

$$C_1 = \frac{0.9 \left[\frac{b}{D_o} \right]}{\left[6 + \frac{b}{D_o} \right]}$$

Where:

b = Manhole diameter, m.

D_o = Outflow pipe diameter, m.

Once the manhole diameter is four times the outlet pipe diameter, or larger, the manhole is “large” and the coefficient C1 is assumed to be a constant equal to 0.36.

$$C_1 = \frac{0.9 \left[\frac{0.50}{0.10} \right]}{\left[6 + \frac{0.50}{0.10} \right]} = 0.409$$

Water Depth in the Manhole

$$C_2 = 0.24 \left(\frac{d_{mH}}{D_o} \right)^2 - 0.05 \left(\frac{d_{mH}}{D_o} \right)^3$$

Where:

d_{mH} = Depth in the manhole relative to the outlet pipe invert, m.

$$C_2 = 0.24 \left(\frac{0.02}{0.10} \right)^2 - 0.05 \left(\frac{0.02}{0.10} \right)^3 = 0.0537$$

Multiple inflows

$$C_3 = \text{Term1} + \text{Term2} + \text{Term3} + \text{Term4} + \text{Term5}$$

$$\text{Term 1} = 1.0$$

$$\text{Term 2} = \sum_{i=1}^4 \left(\frac{Q_i}{Q_o} \right)^{0.75} \left[1 + 2 \left(\frac{Z_i}{D_o} - \frac{d_{mH}}{D_o} \right)^{0.3} \left(\frac{Z_i}{D_o} \right)^{0.3} \right]$$

$$\text{Term 3} = 4 \sum_{i=1}^3 \frac{(\text{Cos } q_i)(HMC_i)}{\left(\frac{d_{mH}}{D_o} \right)^{0.3}}$$

$$\text{Term 4} = 0.8 \left| \frac{Z_a}{D_o} - \frac{Z_b}{D_o} \right|$$

$$\text{Term 5} = \left| \left(\frac{Q_A}{Q_o} \right)^{0.75} \sin q_A + \left(\frac{Q_B}{Q_o} \right)^{0.75} \sin q_B \right|$$

$$HMC_i = \left[0.85 - \left(\frac{Z_i}{D_o} \right) \left(\frac{Q_i}{Q_o} \right)^{0.75} \right]$$

- Q_0 = Total discharge in the outlet pipe, m³/s.
 Q_1, Q_2, Q_3 = Pipe discharge in inflow pipes 1, 2, and 3, m³/s.
 Q_4 = Discharge into manhole from the inlet, m³/s.
 Z_1, Z_2, Z_3 = Invert elevation of inflow pipes 1, 2, and 3 relative to the outlet
 Z_4 = Elevation of the inlet relative to the outlet pipe invert, m.
 D_0 = Outlet pipe diameter, m.
 b = Manhole diameter, m.
 dmH = Depth in the manhole relative to the outlet pipe invert, m.
 q_1, q_2, q_3 = Angle between the outlet main and inflow pipes 1, 2, and 3, degrees.
 HMC_i = Horizontal momentum check for pipe i.
 Q_A, Q_B = Pipe discharges for the pair of inflow pipes that produce the largest value
for term 4, m³/s.
 Z_A, Z_B = Invert elevation, relative to outlet pipe invert, for the inflow pipes that produce the largest value for term 4, m.

For the hydraulic model we assume that:

$$Z_1 = 0.3 \text{ m}$$

$$Z_2 = 0.15 \text{ m}$$

$$d_{mH} = 0.05$$

To calculate term 2

For i = 1

$$\begin{aligned} & \sum_{i=1}^4 \left(\frac{Q_i}{Q_o} \right)^{0.75} \left[1 + 2 \left(\frac{Z_i}{D_o} - \frac{d_{mH}}{D_o} \right)^{0.3} \left(\frac{Z_i}{D_o} \right)^{0.3} \right] \\ &= \left(\frac{0.00645}{0.0129} \right)^{0.75} \left[1 + 2 \left(\frac{0.3}{0.10} - \frac{0.02}{0.10} \right)^{0.3} \left(\frac{0.3}{0.10} \right)^{0.3} \right] \\ &= 2.77 \end{aligned}$$

for i = 2

$$\begin{aligned} & \sum_{i=1}^4 \left(\frac{Q_i}{Q_o} \right)^{0.75} \left[1 + 2 \left(\frac{Z_i}{D_o} - \frac{d_{mH}}{D_o} \right)^{0.3} \left(\frac{Z_i}{D_o} \right)^{0.3} \right] \\ &= \left(\frac{0.00128}{0.0087} \right)^{0.75} \left[1 + 2 \left(\frac{0.12}{0.10} - \frac{0.02}{0.10} \right)^{0.3} \left(\frac{0.25}{0.15} \right)^{0.3} \right] \\ &= 1.56 \end{aligned}$$

$$\text{Term 2} = 2.77 + 1.56 = 4.33$$

Term 3

$$4 \sum_{i=1}^3 \frac{(\cos q_i)(HMC_i)}{\left(\frac{d_{mH}}{D_o}\right)^{0.3}}$$

$$HMC_i = \left[0.85 - \left(\frac{Z_i}{D_o}\right) \left(\frac{Q_i}{Q_o}\right)^{0.75} \right]$$

$$HMC_1 = \left[0.85 - \left(\frac{0.3}{0.10}\right) \left(\frac{0.0129}{0.00645}\right)^{0.75} \right] = -0.9338$$

$$HMC_2 = \left[0.85 - \left(\frac{0.10}{0.10}\right) \left(\frac{0.0129}{0.00645}\right)^{0.75} \right] = 0.255$$

$$\mathbf{Term\ 3} = 4 * \left[\frac{(\cos 90^\circ)(-0.9338)}{\left(\frac{0.02}{0.10}\right)^{0.3}} + \frac{(\cos 45^\circ)(0.255)}{\left(\frac{0.02}{0.10}\right)^{0.3}} \right] = 2.72$$

$$\mathbf{Term\ 4} = 0.8 \left| \frac{Z_a}{D_o} - \frac{Z_b}{D_o} \right|$$

$$0.8 \left| \frac{0.3}{0.10} - \frac{0.10}{0.10} \right| = 1.60$$

$$\begin{aligned} \text{Term 5} &= \left| \left(\frac{Q_A}{Q_o} \right)^{0.75} \sin q_A + \left(\frac{Q_B}{Q_o} \right)^{0.75} \sin q_B \right| \\ &= \left| \left(\frac{0.00645}{0.0129} \right)^{0.75} \sin 90 + \left(\frac{0.00645}{0.0129} \right)^{0.75} \sin 45 \right| = 1.037 \end{aligned}$$

$$C3 = 1 + 4.33 + 2.72 + 1.60 + 1.037 = 10.69$$

Relative pipe diameters

$$C_4 = 1 + \left[\left(\frac{Q_i}{Q_o} + 2 \frac{A_i}{A_o} \cos q_i \right) \frac{V_i^2}{V_o^2} \right]$$

Where:

A_i, A_o = Cross-sectional area of inflow and outflow pipes, m².

θ_i = Angle between outflow pipe and inflow pipe i, degrees.

If θ_i for any pipe is less than 90 degrees or greater than 270 degrees, ($\cos \theta_i$) is replaced with 0. This sets the maximum exit loss to be the incoming velocity head.

$$C_4 = 1 + \left[\left(\frac{0.00645}{0.0129} + 2 \frac{0.00785}{0.00785} \cos 90 \right) \frac{0.821^2}{1.65^2} \right]$$

$$C_4 = 0.9017$$

$$\Delta E = (C_1 C_2 C_3 + C_4) w \frac{V_o^2}{2g}$$

$$\Delta E = (0.409 * 0.0537 * 10.69 + 0.9017) * 1 * \frac{1.65^2}{2 * 9.81}$$

$$\Delta E = 0.15775 \text{m}$$

Appendix B

Appendix C

Exp #

Sample

Sample weight		g
Unused sediment		g
Weight of used sediment		g
Dry weight		g
Weight of sediment settled in manhole		g
Weight of sediment in pipe		g

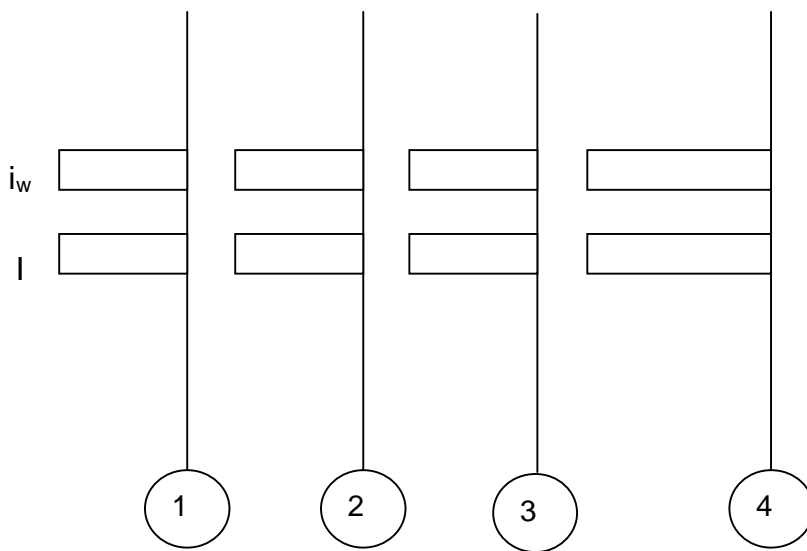
Sediment

Type of sediment	Sand	
d50	0.3	mm
Sediment opening type	2/3 open	
Sediment discharge rate	1.572E-05	m ³ /sec
Length of sediment in pipe		m

Flow

Pipe Diameter	0.1	m
Flow rate	0.0129	m ³ /sec
Velocity	1.65	m/sec

Hydraulic gradient



Manometer

Fig (C.1) Experimental Form

The geometrical relationships are for constant pipe diameter shown in table 7.1 and Figure (D.1) to Figure (D.8)

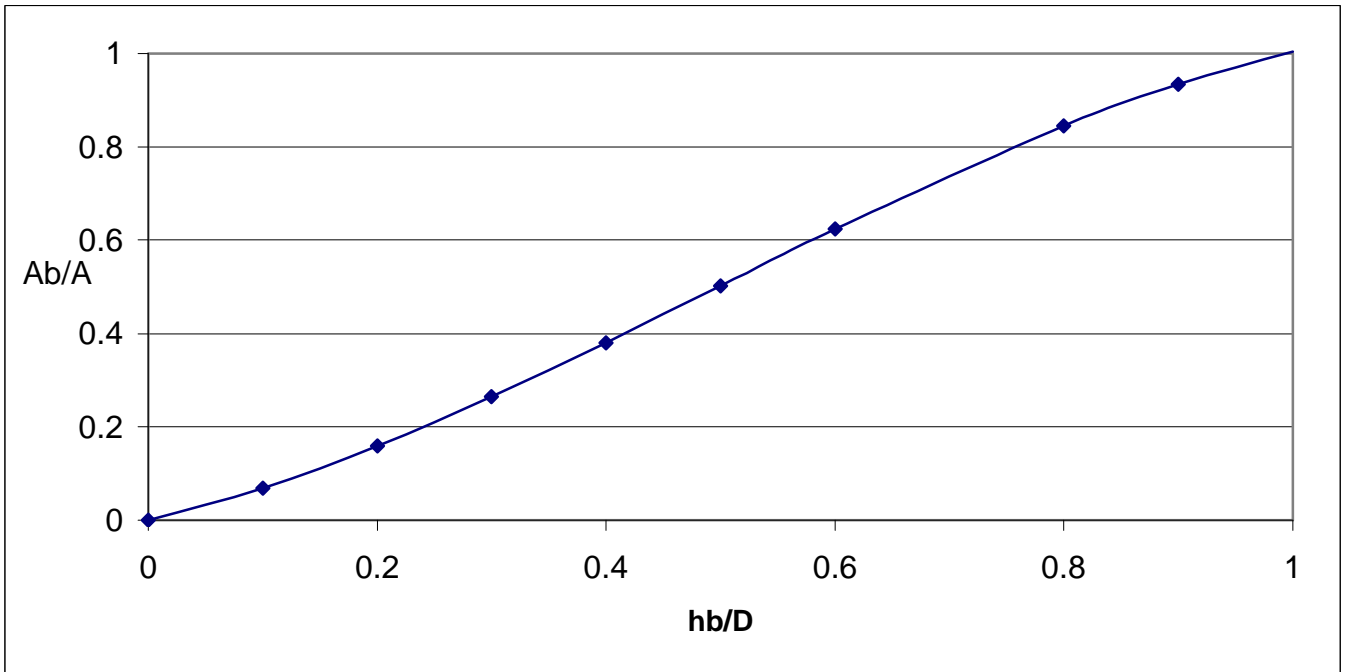


Fig (D.1) Geometrical Relationship between the Ab/A and hb/D

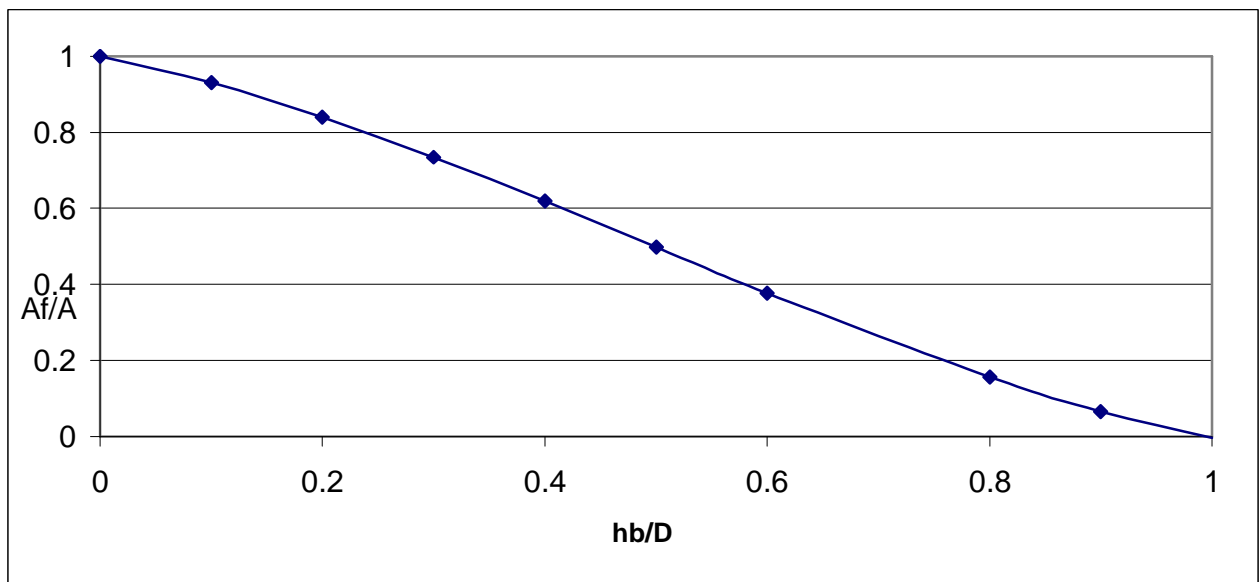


Fig (D.2) Geometrical Relationship between the Af/A and hb/D

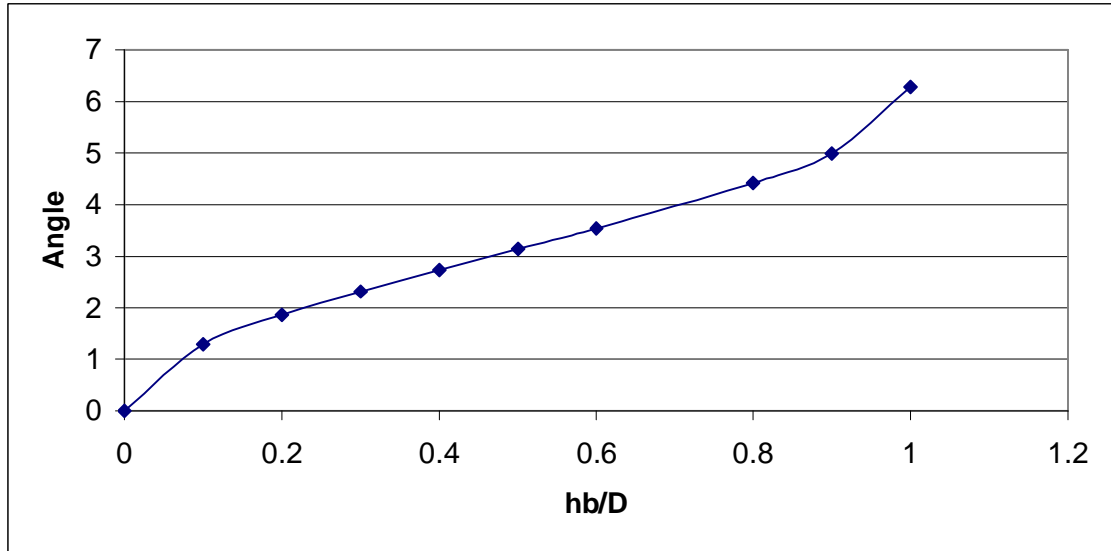


Fig (D.3) Geometrical Relationship between the hb/D and the angle

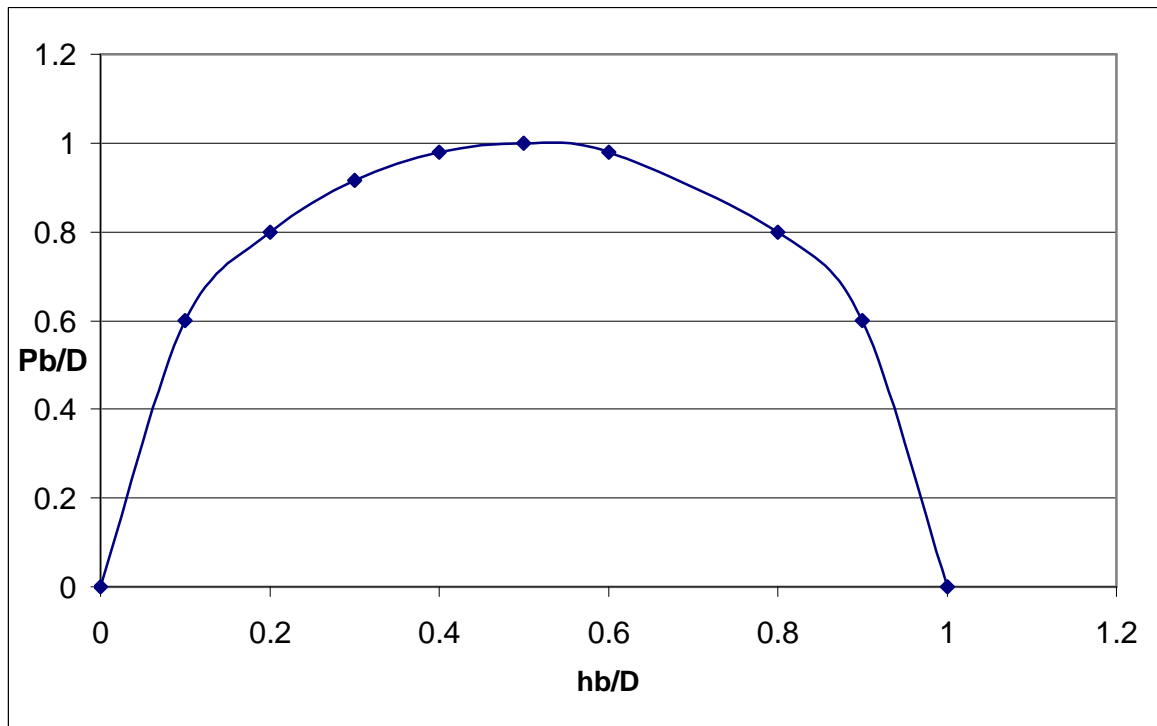


Fig (D.4) Geometrical Relationship between the hb/D and Pb/D

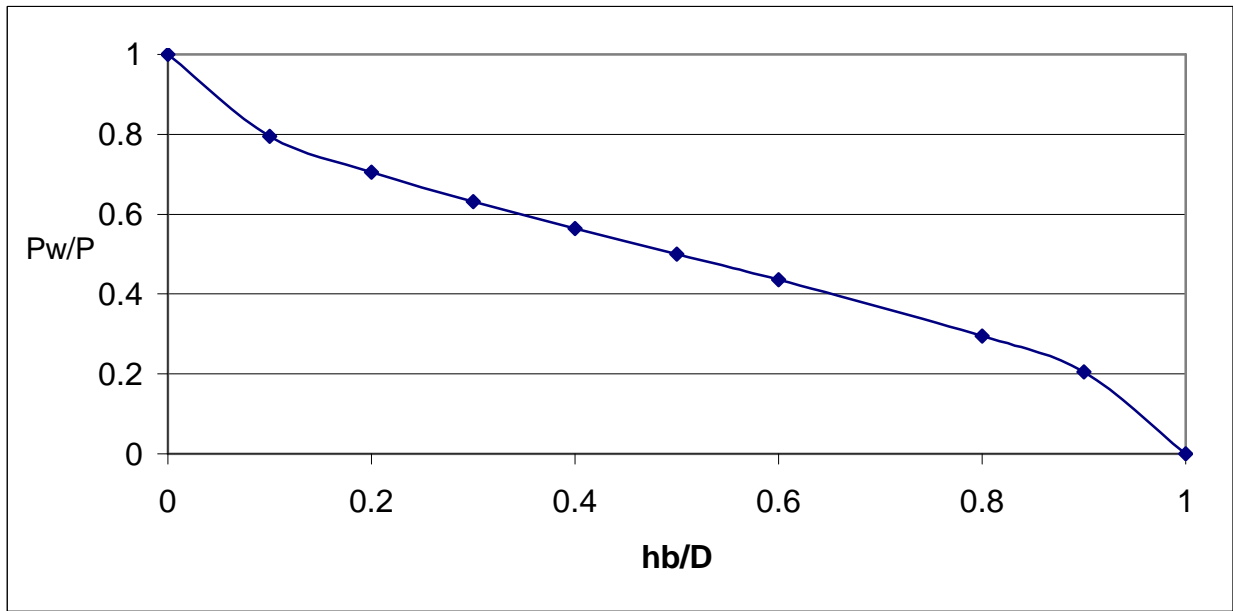


Fig (D.5) Geometrical Relationship between the hb/D and Pw/P

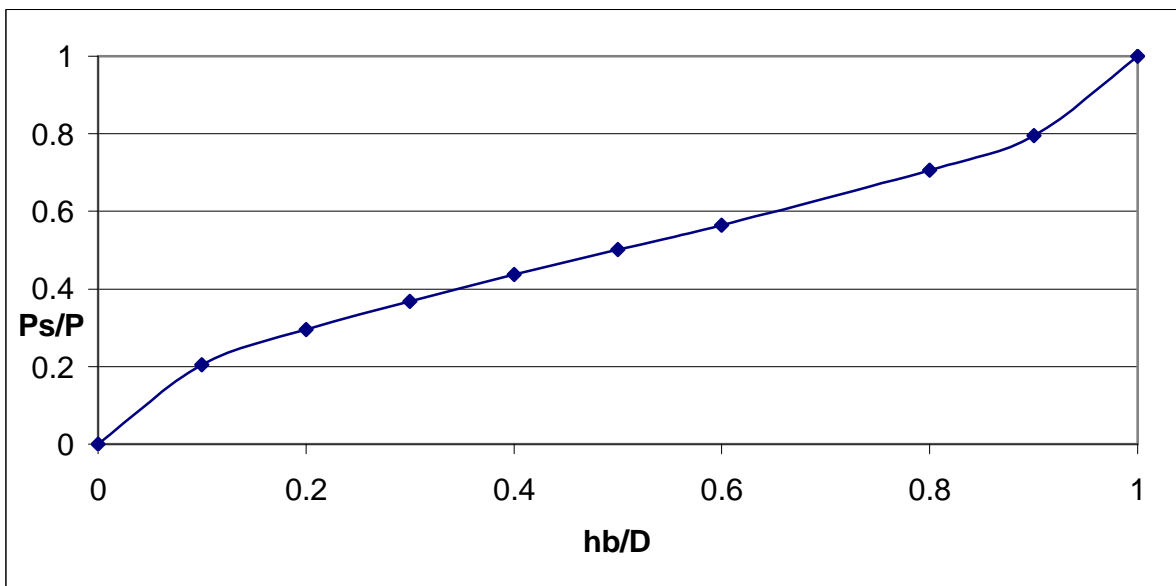


Fig (D.6) Geometrical Relationship between the hb/D and Ps/P

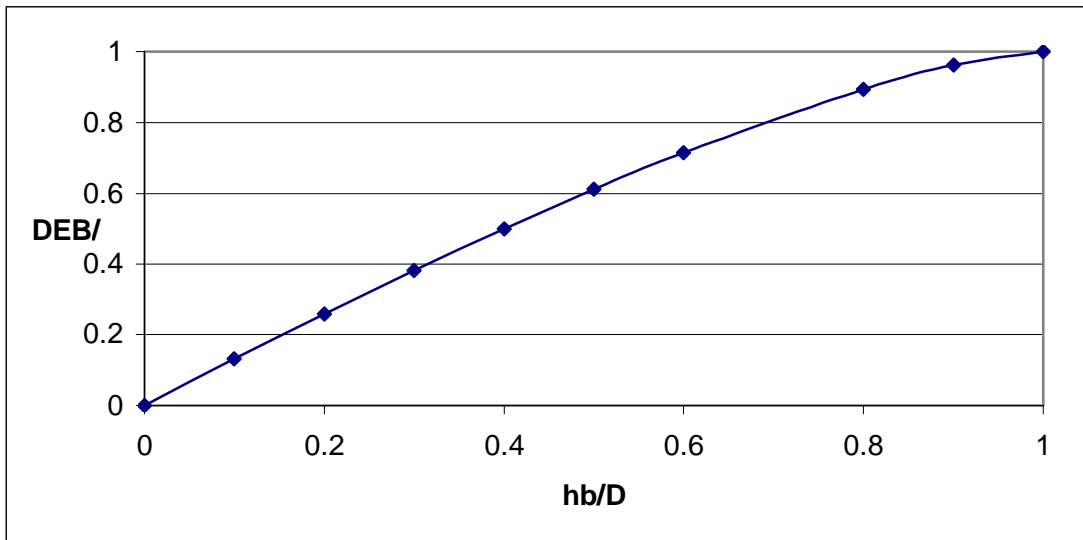


Fig (D.7) Geometrical Relationship between the hb/D and DEB/D

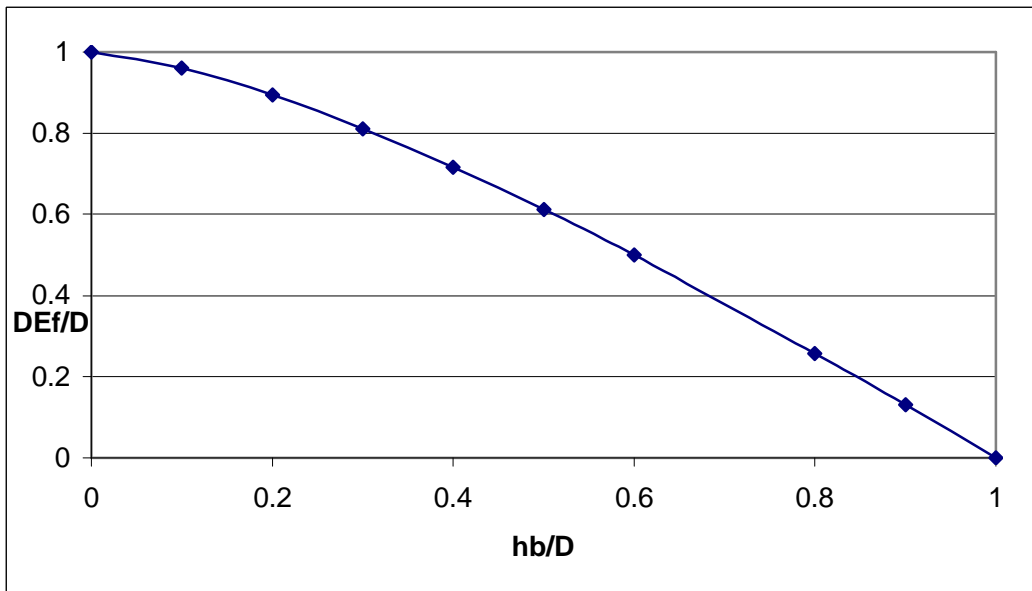


Fig (D.8) Geometrical Relationship between the hb/D and DEf/D