DEPOSITION IN AN 8-INCH VITRIFIED CLAY SEWER PIPE

THESIS FOR THE DEGREE OF M. S.
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CHARLES H. RATHS

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DEPOSITION IN AN 8-INCH VITRIFIED CLAY SEWER PIPE

By

Charles H. Raths

AN ABSTRACT

Submitted to the College of Engineering
Michigan State University of Agriculture and
Applied Science in partial fulfillment of
the requirements for the degree of

MASTER OF SCIENCE

Department of Civil Engineering

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Approved: Robert F. McCauley

CHARLES H. RATHS ABSTRACT

This thesis is concerned with deposition of sand particles in an 8-inch vitrified clay sewer pipe line.

Experiments were conducted with sand particles between 0.1 and 9.5 millimeters in size and having an average specific gravity of 2.7. The sand particles were studied for deposition at varying pipe slopes and depths of flow in an experimental 8-inch sewer pipe line, carrying sewage.

The experimental investigation showed that the pipe joints were the main cause of deposition in the sewer pipe line. Furthermore, an analysis of the experimental data indicated that a definite relationship exists between the size sand particle which deposits at a given pipe slope and the depth of flow in an 8-inch vitrified sewer pipe line.

In addition to the experimental work, a Questionnaire was sent to 52 city engineers in the State of Michigan to provide information relative to blockage problems in sanitary sewers. The concensus of opinion of these city engineers was that <u>roots</u> were the most common cause of sewage blockages and that <u>high quality</u> workmanship is the most important single consideration in the design and construction of sanitary sewers.

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I. INTRODUCTION

The determination of the minimum allowable grades for sewers is both an engineering and an economic problem. A better understanding of the engineering factors which determine the minimum allowable grade for a sewer can be of significant economic importance.

To illustrate the economics involved, consider the typical design standard for an 8-inch sewer. This standard dictates a minimum allowable grade of 0.40%. Assume an area is to be sewered at a grade of 0.30% rather than the usual 0.40%. At the end of one mile, the sewer at 0.40% would require five more feet of depth than the sewer at 0.30% slope.

If we estimate the average increase of two-and-a-half feet of depth to cost $0.80 per foot, the increased expense resulting from the use of 0.40% grade for one mile is $4224.00. However, as the depth of the sewer increases, so does the unit price of placing it. Extending the sewer an additional mile will result in an average difference of depth of seven-and-a-half feet. Thus, employing the sewer at an 0.40% grade could increase the cost for the second mile by $8,000 to $10,000.00 over that at 0.30%; the exact figure being dependent upon the soil conditions, topography and construction costs.

The Problem

Generally, there are several factors which must be considered in the over-all problem of minimum grades. Briefly, any comprehensive evaluation should take into consideration organic depositions, inorganic depositions, construction methods, "n" values and kinds of debris in order to determine the effect of each on blockages.

Organic Deposition

Grades must be adequate to assure that fecal matter and garbage do not deposit on the sewer invert. Such deposits result in septic conditions and foul odors. Of the organic materials which cause problems, the most important is grease. Most reports on sewer blockages mention the presence of adhered grease on the walls of the pipe. During periods of low flow, grease deposits tend to entrap and bind other organic materials as well as particles of sand and silt.

This type of deposit retards velocity, particularly during periods of minimum flow. Settlement of additional material within the stoppage area then tends to further block the pipe and, if the sediment is accompanied by grease depositions, the deposit may become hard and stiff. In time, this repeated cycle can completely fill the pipe.

Inorganic Deposits

With excessively flat grades, the transporting velocity and the depth of flow may not be sufficient to prevent inorganic depositions of sand and silt on the invert. Because of their larger specific gravities, inorganic materials deposit at higher velocities than organic materials. If the deposits are cemented by grease, serious blockages can result in short periods of time. Large deposits of heavy sand may also block sewers without an attendant cementation by slimes and grease.

Construction

Many city engineers have estimated that 50 to 80 per cent of all stoppages are directly attributable to poor joints or other small cracks through which root hairs enter and expand. Generally, improper bed construction with differental pipe settlement and improper joints are by far the most important causes of pipe stoppages.

"n" Values

The value of "n" for the Manning formula has been fairly well established for clear water flowing in clean pipes at depths greater than 0.1 to 0.2 times the pipe diameter. However, for sewers which have been in use for long periods of time, there is little information available upon the effect of slimes, grease and other depositions. The question which still remains appears to be this: Can "n" and the other hydraulic parameters in the Manning equation adequately take into consideration the altered pipe condition? Whether the present "rule of thumb" 0.013 "n" value, for pipes in service flowing full, is high or low is not known at this time.

Debris

Debris such as toilet paper, rags, etc., are common to all sanitary sewers. From time to time unusual items such as bricks, boards, clothes, etc., are found in sewers, but these instances are not common. Toilet paper, rags and similar materials may at times accelerate blockages which are initiated by some of the previously mentioned causes; but it is doubtful that debris is, in itself, a principal cause of stoppages.

Investigation Objective

The major purpose of this project has been a reexamination of the present design practice relative to 8-inch sanitary sewers. 8-inch sewers have been selected for the study because of their common usage. This sewer size probably represents 70 to 80 per cent of all the new sanitary sewers now installed.

Two provocative factors in initiating the investigation were: 1) a questionnaire sent to cities in Michigan and 2) correspondence by the author with several cities in California. Some of these cities reported 8-inch pipes which operated at grades of less than the standard design value of 0.40%. Furthermore, a considerable number of the engineers contacted tended to doubt that velocity was the sole factor in preventing deposition or blockage. Judging from their reports, depth of flow, construction practices and tightness of joints were considered by these engineers to be equally important in affecting blockage.

The work reported here was restricted to an evaluation of the manner in which deposition was affected in sewer pipes carrying sewage by the following variables:

(1) Depth of flow
(2) Slope of the sewer pipe line
(3) Particle's size and specific gravity of deposited materials

Also of interest were observations on the influence of suspended solids in the sewage flow upon the self-cleansing capacity of a sewer.

Incidental to the above objectives was an examination of the data to observe how Manning's "n" varied at low depths of flow for this particular condition of joints and of material deposition on the inverts.

Lastly, it was hoped that the study would provide better experimental procedures for future investigations by constructive criticisms on the methods used in this investigation.

II. SURVEY OF ENGINEERING PRACTICE

In the latter part of 1959 a questionnaire was sent to 52 Michigan cities and towns. At the same time, 6 letters were written to cities in California and Texas which were known to have employed grades of less than 0.4% for 8-inch sewers. The purpose of both surveys was two-fold: (1) to find a practical basis for evaluating whether the minimum grades used in practice for sanitary sewers were reasonable and (2) to define the factors which influenced deposition and blockages in sanitary sewers.

The Questionnaire

The Questionnaire consisted of 11 questions which were devised to pin down some of the common causes of deposition previously mentioned. Space for comments was provided on the Questionnaire sheets to allow the engineers to express their opinions and experiences relative to the question.

The Questionnaire, in its entirety, is reproduced on the following tabulation pages.

Tabulation of Results

Replies to the Questionnaire were received from 29 city or town engineers. To effectively tabulate the replies, each engineer's reply was recorded separately. For the sake of brevity, some of the comments were shortened with the central theme retained.

To avoid confusion, the replies to each Questionnaire were tabulated in the following manner:

1) Replies that in the author's opinion were not pertinent to the question were omitted.

2) The left column on all the tabulation sheets indicates the city replying.

3) The questions which required a numerical, checking the appropriate square, or a yes and no type of answer are listed to the right of the replying city. If comments were asked for, the comments appear on the line immediately under the above answers.

4) The questions which required an opinion type of answer are listed immediately opposite the replying city or town.

5) Following the tabulation of questions is a section devoted to general comments by the replying engineers.

Conclusions of Questionnaire

A careful study of the Questionnaire replies served as a basis for the following conclusions:

1) Blockages are caused mainly by tree roots and poor or improper construction practices.

2) Sand particles are present in most sewers. The range in size commonly found is between 0.03 mm. and 3.0 mm.

3) The most frequently found organic materials are garbage and grease. These deposits occur in sewers where the flow is small or intermittent. However, not all sewers give evidence of organic deposition.

4) Generally, long bars of deposited material are not present in sewers. When these bars are present, they appear to have either a hard cemented or sticky nature and the bars seem to be composed of grease and detergent materials.

TABULATION QUESTION (1)

Which of the following most often causes blockage in your sanitary sewers? (House laterals, not mains or combined sewers):

City	Sand Deposits	Grease	Fecal Solids	Toilet Paper	Sanitary Napkins	Tree Roots	Garbage	Other
Allen Park	10%	40%			20%	20%	10%	
	Fecal solids, toilet paper & floating garbage do not cause blockage							
Ann Arbor					20%	80%		
Birmingham		10%				75%	15%	
	Solid settling results primarily from poor or reverse grade							
Dearborn					—	75%		
Detroit		40%				40%		20%
East Lansing		30%			5%	60%		5%
	Poor installation and improper bedding also a factor							
Escanaba	4%	4%	4%	4%	4%	80%		
Flint	5%	10%	1%	2%	2%	80%		
Grand Rapids	15%	3%				75%	2%	5%
Grosse Pointe Farms			Combined System			100%		
Grosse Pointe Park	5%		Combined System		2%	90%		3%
Hazel Park	50%				10%	40%		
	Grease is a factor where restaurants are located on dead end sewer							
Highland Park	10%	10%			10%	70%		
Holland	15%				3%	80%		2%
Jackson		5%				95%		
	Tree roots are almost always the primary cause of blockage							
Kalamazoo		30%			10%	40%		20%
Livonia		40%				60%		
Menominee	30%					65%		5%
	Sand deposits causing blockage normally due to broken pipes or bad joints							

TABULATION QUESTION (1) cont'd

City	Sand Deposits	Grease	Fecal Solids	Toilet Paper	Sanitary Napkins	Tree Roots	Garbage	Other
Niles						100%		
	Due to faulty construction, roots cause stoppage							
Oak Park						85%	15%	
	Tree roots are the one major problem							
Owosso		10%				10%	60%	20%
Pontiac						80%		
Royal Oak		34%				33%	33%	
St. Joseph		5%		10%		80%		
Sault Ste. Marie		x		x	x	majority	x	x
Traverse City				10%	10%	50%		20%
Trenton	60%(clay)							x
	Broken or mislaid tiles responsible for clay deposits							
Wyandotte		10%	(Combined System)			90%		
Ypsilante		10%				80%	10%	
Wayne	1%	15%		2%	2%	75%	5%	
	Changed from slip seal joints to lock type to prevent root entrance							
Warren	90%*				x	x	x	Acids
	*On new construction due to abuse of draining basements							

TABULATION QUESTION (2)

+

Do you find sand or other inorganic materials deposited in your sanitary laterals? If so, what approximate size (in mm.) is most frequently found?

Allen Park	Yes, no tests
Ann Arbor	Very seldom
Birmingham	In most cases very little sand
Detroit	Occasionally sand is found
East Lansing	Yes, particles of silt size
Escanaba	Variable sizes, enter through manholes
Flint	All sizes from coarse to fine
Grand Rapids	Yes, size unknown
Grosse Pt. Park	Average size found is 0.03 mm.
Hazel Park	Small amounts
Highland Park	Sand's presence is due to builders' moving earth and new basement construction
Holland	Sand which comes from manholes; size is 3 mm.
Jackson	Particles of sizes 0.6 mm. to 1.2 mm.
Kalamazoo	No
Livonia	Sand from basement construction and taps
Menominee	Sand, size estimated at 0.3 mm.
Niles	Very little except during construction
Oak Park	Some fine silty sand is found
Owosso	Very little
Pontiac	Sand seldom found, area has clay soil
Royal Oak	Very fine silt found
Sault Ste. Marie	No records maintained
Traverse City	Small amounts of fine to medium sand
Ypsilanti	Due to old joints, stones from 3/8" to 1-1/2" frequently cause blockage
Wayne	Sand deposited has an average size of 0.5 mm.
Warren	Sand is found in sewers near construction

TABULATION QUESTION (3)

Do you find deposits of organic materials (garbage, fecal or other) in your laterals? If so, what is the nature of the material most frequently found?

Allen Park	Yes, when otherwise plugged
Ann Arbor	No
Birmingham	Most materials found consist of garbage and grease
Detroit	Not often
East Lansing	Grease and roots
Escanaba	Where flows are small and intermittent at upper ends, fresh domestic sewage is found
Flint	Occasionally sludge and grease are found where grades are flat (0.3% to 0.4%)

TABULATION QUESTION (3) cont'd

Grand Rapids	In combined sewers, leaves and twigs are found which are hard to penetrate
Grosse Pte. Park	Hair and massed roots
Hazel Park	None
Highland Park	Fecal matter and ground garbage are found deposited in the first 600' from the upper end
Holland	Some garbage and very little grease
Jackson	No
Kalamazoo	Tree roots
Livonia	Garbage at head ends due to lack of flow
Menominee	No
Niles	No
Oak Park	Garbage disposal wastes most frequently found
Owosso	Yes, type hard to determine
Pontiac	Very seldom
Royal Oak	Garbage and grease
St. Joseph	No
Traverse City	Very seldom
Trenton	No, garbage disposal units required in all new dwellings
Ypsilanti	Garbage most frequently found; and, eggshells and corn most often
Wayne	Near restaurants considerable amounts of grease found
Warren	Ground vegetable wastes found due to supermarkets

TABULATION QUESTION (4)

Do you find long bars of deposited material in laterals? If so, does the material appear to be cemented together or of a sticky nature?

Allen Park	Sticky bars (grease)
Ann Arbor	No
Birmingham	Occasionally long bars found cemented together by grease and detergent where grades are flat
Dearborn	Long bars of detergent composition found
Detroit	Not often, appear cemented
Escanaba	Small deposits of cemented material found
Flint	No
Grand Rapids	The grease bars are hard but easy to penetrate; if mixture of materials, bars are quite sticky
Grosse Pte. Park	Cemented together
Hazel Park	No
Highland Park	No
Holland	No
Jackson	No
Kalamazoo	Yes, sticky nature

TABULATION QUESTION (4) cont'd

Menominee	No
Niles	No
Oak Park	No
Owosso	No
Pontiac	No
Royal Oak	No
St. Joseph	Grease bars due to a fishery
Traverse City	Bars are rarely sticky
Trenton	No
Ypsilanti	No
Wayne	Yes, near restaurants and from disposal units
Warren	No

TABULATION QUESTION (5)

For laterals that give trouble due to blockages, how full does the pipe usually flow (average) and what is the slope?

Birmingham	Generally, the flow and slope are small when blockage occurs
Dearborn	1/4 full at 0.4% slope
Detroit	1/10 full at 1/8" per foot
East Lansing	1/3 to 3/4 full where slope is less than 0.2%
Escanaba	Generally 3/4 full at slopes 0.2 to 0.3%
Flint	About 1/4 full where slope is 0.2 to 0.4%
Holland	1/2 to 3/4 full at a slope of 0.3%
Kalamazoo	1/4 full for old sewers at 0.32% slope
Livonia	3/8 full for slopes between 0.3% and 0.4%
Menominee	Bad joints rather than slope is the problem
Oak Park	Old flat sewers flowing less than 1/2 full are the biggest problem
Owosso	1/3 to 1/2 full, slope unknown
Royal Oak	Slope does not appear to be a factor, depth of flow unknown
Traverse City	Slopes are 0.3% to 0.4%, but blockage is due to to poor workmanship
Ypsilanti	About 1/4 full, slope unknown
Wayne	Older sewers at 0.25% grade flowing 1/4 to 1/3 full are troublesome
Warren	Generally 0.25% or less

TABULATION QUESTION (6)

Has the greatly increased use of automatic washing machines and detergents affected pipe blockage? (a) Keeps pipes clean (b) No effect (c) Tends to cement particles

	(a)	(b)	(c)
Allen Park		x	
Ann Arbor		x	
Birmingham			x
Dearborn			x
Detroit			x
	No great increase		
East Lansing			x
	Detergents adhere to pipe walls and act as a binder for other materials		
Escanaba		x	
Flint		x	
Grand Rapids		x	
Grosse Pte. Park	x		
Hazel Park		x	
Highland Park	x		
Holland		x	
Jackson		x	
Kalamazoo		x	
Livonia		x	
	Lint and hair are deposited in sewers due to automatic washers		
Menominee		x	
Niles		x	
Oak Park			x
Owosso		x	
Pontiac		x	
Royal Oak		x	
St. Joseph		x	
Sault Ste. Marie	x		
	Additional water tends to flush sewers		
Traverse City	x		
	Damage to asphaltic joints should be considered		
Trenton			x
	More noticeable on screens at pumping stations		
Wyandotte			x
Ypsilanti	No opinion, but failures in Orangeburg pipe have occurred in recent years which prior to the heavy use of automatic washers did not happen		
Wayne			x
	Automatic washers and detergents seem to compact solid materials		
Warren		x	

TABULATION QUESTION (7)

Do you think flat grades (of less than 0.40%) have any effect upon blockage of laterals?

Ann Arbor	No
Birmingham	Other than roots, flat grades for small pipes are the main cause of blockage
Dearborn	Yes, solids tend to settle out
East Lansing	Yes, flow is sluggish causing deposition
Escanaba	Yes, flat grades allow deposition to occur more readily at places where roots and bad joints hinder flo
Flint	Flat grades create low velocities which in turn allows grease to adhere to the pipe walls and act as a collecting place for sludge
Grand Rapids	Our troublesome sewers do not indicate that flat grades cause deposition
Grosse Pte. Park	No, if properly laid to a uniform grade
Hazel Park	Yes
Highland Park	At the upper end of laterals, the grade should be increased
Holland	Sometimes yes, flat grades call for better workmanship and amount of flow is a consideration
Jackson	Flat grades cause deposits. Unless grade is a very flat one, the sludge build-up will not cause blockage
Kalamazoo	Yes, due to lack of flow
Livonia	Yes, small velocity allows particles to deposit
Menominee	No, we have 8" pipe at 0.32% and have no trouble
Niles	Only when grade is so flat that sewage backs up from receiving sewers at high flow
Oak Park	Flat grades cause problems with garbage deposition
Owosso	Yes, material builds up due to lack of pressure behind it
Pontiac	No
Royal Oak	Yes in periods of dry weather
St. Joseph	Yes
Sault Ste. Marie	Perhaps, alt[illegible] the city has several sewers at 0.25% whi[illegible]erate without any trouble
Trenton	Not unless th[illegible]es have poor uneven joints
Ypsilanti	Yes, flat grade[illegible]nd poor workmanship account for many blocka[illegible]
Wayne	Yes, we have mo[illegible] blockage on flat grades due to the settling [illegible]t of solids
Warren	A 0.4% grade is [illegible]ost desireable

TABULATION QUESTION (8)

In your opinion, how important is good workmanship in relation to blockage of sewers (pipes laid true horizontally and vertically, good joints, care with bedding to prevent settlement). (a) Less important than 0.40% grade (b) Not too important unless workmanship is very poor (c) Not too important unless pipe is cracked (d) The most important single consideration

	(a)	(b)	(c)	(d)
Allen Park				x
Ann Arbor				x
Birmingham				x
Dearborn				x
Detroit	x			
	Good design cannot be neglected			
East Lansing				x
	Bedding is also an important consideration			
Escanaba				x
	Proper pipe slope and size are important			
Flint				x
Grand Rapids	Poor joints and horizontal alignment cause the most problems			
Grosse Pte. Park				x
Hazel Park	x			
Highland Park				x
Holland				x
	Good workmanship prevents deposition even on flat slopes			
Jackson				x
	Good joints, horizontal and vertical alignment prevent deposition of sludge. True grades prevent damage when cleaning			
Kalamazoo				x
Livonia				x
	Poor construction causes deposition			
Menominee				x
	Construction defects cause a large share of stoppages			
Niles				x
	Good workmanship prevents 75% to 90% of the troubles			
Oak Park				x
Owosso				x
	More trouble with sewers where inspection problems arose with contractors			
Pontiac				x
Royal Oak				x
St. Joseph				x
Sault Ste. Marie		x		

TABULATION QUESTION (8) cont'd

	(a)	(b)	(c)	(d)
Traverse City				x
	Good workmanship permits flat grades			
Trenton				x
Ypsilanti				x
	Workmanship and good joints are by far the most important			
Warren	x			x

TABULATION QUESTION (9)

What is the normal period in months between flushings for your system?

	Sewer Grade in per cent						
	0.6%+	0.5%	0.4%	0.3%	0.2%	0.1%	0.0%
Dearborn (40)*	12 mo.	12 mo.	6 mo.	6 mo.	6 mo.		
Flint (130)	none	none	18mo.	8 mo.	2 mo.		
Kalamazoo (100)	6 mo.			3 mo.			1 mo.
Livonia (10)	12 mo.						
Pontiac	12 mo.	12 mo.	12 mo.	12 mo.			
Ypsilanti						6 mo.	

GENERAL NOTE The cities not recorded here generally flush and clean only when required or flush on a continuous schedule.

+ Per cent grade, 8" pipe
* Man hours per mile of flushing

TABULATION QUESTION (10)

What grades do you recommend for 8" laterals which you know will flow less than half full?

Allen Park	0.4% minimum with 4 downspouts connected at upper end
Ann arbor	0.32% minimum, more if possible
Dearborn	0.4%
East Lansing	Not less than 0.2% if possible
Escanaba	A 0.5% to 1.0% slope is necessary to maintain sufficient velocity
Flint	0.4% minimum to maintain velocity
Grand Rapids	0.8% for flows of d/4; for d/2 and greater, 0.5% minimum

TABULATION QUESTION (10) cont'd

Holland	Not less than 0.4%
Jackson	0.4% minimum
Kalamazoo	0.4%
Livonia	0.4%
Menominee	Not less than 0.32%
Niles	0.25% minimum
Oak Park	We use minimum pipe velocities of 2.5 f/s
Owosso	Minimum of 0.4%
Pontiac	0.5% or more if possible, 0.2% minimum
Royal Oak	0.4% minimum, steeper grade is of little benefit for carrying solids when the pipe is carrying a very small flow
St. Joseph	0.4%
Traverse City	0.32%, could be less where good workmanship is present
Ypsilanti	0.4%, except at upper end use 0.5%
Wayne	0.4% due to increase of garbage and detergents
Warren	0.5%, we attempt to have a velocity of 2.23 f/s

TABULATION QUESTION (11)

Can you give us any information as to typical average and maximum flows in an 8" lateral?

INSUFFICIENT REPLIES

QUESTIONNAIRE GENERAL COMMENTS OF PARTICULAR INTEREST

Grand Rapids

"We have what may be an unusual condition in Grand Rapids, in that generally most of the sanitary sewer facilities in the flat areas are of the combined sewer type. Whenever there is a rainfall of any intensity they receive a flushing, which in a measure makes up for the flat grades . . . . .

"We find that our biggest difficulties are with overloading of the combined sewers during unusual rains and with roots getting into some of the joints. The overloading causes a temporary backup into the laterals and the root growth which may develop rapidly causes an impediment increasing the incident of deposits."

Trenton

"My personal experience has been that blocking has been primarily due to faulty joints, poor bedding of pipe, lack of proper supervision in backfill operations and poor inspection, rather than poor design grades. I make this observation in view of the fact that I have watched the City of Trenton grow from 6,000 population to 18,000 during the past five years and have had personal knowledge of the construction of approximately 30 miles of city sanitary sewer and house connections for about 4,000 new homes . . .

"Other than broken crick, the failure of joints has been observed to be the most common cause of sewer plugging. This joint failure can be directly attributed to poor alignment of pipe when laid, improper bedding, cracked pipe hubs, and poor and insufficient amount of joint compound. Any one of the above-mentioned malpractices lets the soil or tree roots get into the sewer and causes an obstruction to the general flow. These field practices can only be corrected by adequate inspection on the job so that the pipe itself is given a fair chance to do the job that it was originally designed for.

"Generally speaking, we have had little or no problem on plugged up sewers because of grades, but find our trouble to be in the joints and cracked pipe, as mentioned before. With contractors bidding projects cheaper than ever before, it is my recommendation that engineers draw up specifications tighter than ever before and specify stronger pipe with the best joints possible. As an additional check on the contractors, I intend to write into all future specifications that the sewer contractor shall furnish a two-year mantainenance bond for the repair of any collapsed or improperly working sewer.

"A possible cause for pipe failure can be attributed to the handling of sewer pipe from box car to truck or from the truck to the pile, and then the laborer dropping the pipe into the trench and having it hit on the bell end. This rough handling could result in collapse of the pipe when the backfill load is distributed over the entire length of the pipe."

QUESTIONNAIRE GENERAL COMMENTS OF PARTICULAR INTEREST

Warren

"It is often the case that a locality adopts a standard based on tradition or precedent because there are not means to investigate. Another consideration is that the State Board of Health has the right to approve all sewers to be built. They have a set of curves and I find the Bible of Design therein."

Ypsilanti

"We have sewers which were laid by good workmen which have very little grade due to lack of elevation between the house and sewer. Yet, we have very little trouble with stoppage in these sewers. The major portion of our house connections were made with poor workmanship and no matter what the grade, we have continuing problems with stoppage."

"My experience on both ends of the sewer business leads me to firmly believe that workmanship and joints are the two most important items in regard to preventing stoppage in sewers. Poor workmanship can ruin a well-designed sewer system and make a poorly designed system impossible . . . . Weak joints are responsible for many stoppages."

5) Blockage problems occur when the average depth of flow is about one-third full or less and at a slope of approximately 0.3 per cent.

6) Generally, the increased use of automatic washers and detergents has had little effect on causing blockages. But, their use tends to cement particles already deposited on the inverts.

7) The engineers' experiences and opinions indicate a fairly even division on whether or not flat grades are responsible for deposition.

8) The majority of the engineers are of the opinion that good workmanship is the single most important consideration concerning blockage of sewers. Good joints are also an important factor.

9) The normal period for flushing of sewers, regardless of grade, is between 6 to 16 months.

10) The majority of the engineers recommend 0.40% grade for pipes which will flow less than half full.

Letter Survey

The 1929 edition of American Sewerage Practice, by Metcalf and Eddy, listed the results of a survey on minimum gradients for sewers. Several of the reporting municipalities employed the following minimum grades for 8-inch sewers:

1) Corpus Christi, Texas	0.20%
2) Fresno, California	0.10%
3) Modesto, California	0.16%
4) Sacramento, California	0.20%
5) Stockton, California	0.143%
6) Visalia, California	0.24%

Because the above grades were well below the suggested minimum, the author corresponded with the above cities to determine if these grades had caused excessive problems.

The author's main interest was what effect the above-mentioned minimum grades had on blockage. Therefore, in an effort to better understand the problem of blockage, the following questions were asked of the six city engineers:

1) With these grades do you have much trouble with blockage?

2) Outside of tree roots and poor construction, what causes blockage in your system?

3) What sort of deposits do you find in your inverts, if any? What is the approximate size of these deposits, if measureable?

4) If you have ever measured Kutter's "n" in your system, what values have you obtained?

5) In your opinion what is the most important criterion in determining minimum grades, considering both field and theoretical considerations?

Results of Correspondence

5 of the 6 city engineers contacted replied to this initial correspondence.

The results were tabulated in the same manner as in the Questionnaire previously discussed.

Conclusions

The following conclusions were drawn from the replies to the Letter Survey:

1) Grades which produce sluggish velocities are a factor in producing blockages. However, the blockage problem is related to the type of load being carried by the sewer.

2) Other than tree roots and poor construction, blockages are created by grease and garbage disposal units.

3) Deposits generally found on inverts are grease, garbage disposal unit refuse and sand.

4) The range of "n" for sewers, as determined by tests, is between 0.011 and 0.013.

5) The most important factor in a properly operating sewer is adequate velocities. (Whether the basis for this last opinion is long standing convention or field experience cannot be determined from the questionnaire.)

TABULATION QUESTION (1)

With these grades do you have much trouble with blockage?

Corpus Christi	Troubles due to slow flow velocities have resulted in blockages at a grade of 0.2%. This grade has been employed in areas which have a very flat terrain.
Modesto	Grades of 0.16% were employed in the 1930's. The use of these grades has resulted in blockages and erosion of the concrete sewers due to sulphides. The problems can be traced to slow flow velocities.
Sacramento	In general, the 0.2% grade caused no problem relative to blockages. And this was particularly true where good workmanship was obtained.
Stockton	With the grade of 0.143%, trouble depends on the load being carried by the sewer. For sewers flowing at least half full we have had no problems.
Visalia	Since 1950 we have employed grades of 0.15 and 0.2%. With these grades we have no blockage problems.

TABULATION QUESTION (2)

Outside of tree roots and poor construction, what causes blockage in your system?

Corpus Christi	Low velocities and items entering through the manholes are the main causes of blockage.
Modesto	Blockage also comes from the increased use of garbage disposals. Deposits of egg shells, coffee grounds, citrus rinds, etc., add to the problems of sluggish velocities by combining with such items as sanitary napkins
Sacramento	When blockage does occur, it is often traceable to poor joints, rough manhole bottoms or breakage. Grease is also a problem.
Stockton	The other main cause of blockage in our system is deposition of solids during periods of low flow.
Visalia	The main causes of blockage in our system have been grease and oil from gas stations.

TABULATION QUESTION (3)

What sort of deposits do you find in your inverts, if any? What is the approximate size of these deposits, if measureable?

Corpus Christi	Deposits on the inverts are a build-up of the solids dropped out at low velocity. I have seen the entire line blocked.
Modesto	With increased use of garbage disposals, deposits of egg shells, coffee grounds, citrus rinds, etc., add to the problem of sluggish lines.
Sacramento	Deposits found on the inverts are off the streets in the form of gravel and dirt which gain access through gutter drains. For smaller sewers we have very little trouble with deposition due to slow velocities.
Stockton	Deposits usually found in our inverts are grease, sanitary napkins, small toys and undergarments.
Visalia	Deposits found in our system are oil, grease, sand and dirt.

TABULATION QUESTION (4)

If you have ever measured Kutter's "n" in your system, what values have you obtained?

Corpus Christi	Kutter's "n" has not been measured.
Modesto	Tests on our lines justify the continued use of "n" equal to 0.013.
Sacramento	No attempt has been made to determine the actual coefficient.
Stockton	To date, no measurements have been made.
Visalia	The "n" value found in our system is 0.011.

TABULATION QUESTION (5)

In your opinion what is the most important criterion in determining minimum grades, considering both field and theoretical considerations?

Corpus Christi	The most important criterion for grades is the velocity that can be obtained considering all factors.
Modesto	We consider velocity to be the most important criterion in determining minimum grades.
Sacramento	Minimum grades are not a problem where good workmanship is present.
Stockton	It is our practice to design sewer systems at grades such that a minimum velocity of two feet per second is obtained.
Visalia	From field observations, I would say that a 0.129% grade is adequate for an 8" clay pipe flowing full.

III. PREVIOUS WORK

Velocity and Deposition

Historically, studies on minimum grades have been made for developing measures to prevent depositions on sewer inverts. Early research was primarily concerned with relating transporting power to flow velocity. One of the first such studies was by Ogden[1] and associates of Cornell University in 1899 who reported data on flow velocities required to initiate movement of different materials. Particles with specific gravities of between 1.26 and 3.0 were found to move at velocities of 1.25 to 2.75 feet per second. The effects of particle dimension and depth of flow were not mentioned by Ogden.

At about this same time, a graduate student at Cornell, G.D. Holmes[2] was investigating the effect of imperfect joints upon the flow in sewer pipes. Holmes was mainly concerned with the roughness of the joints as produced by the interior projections of the mortar in the cemented joint. While some of Holmes' work may have been in error, the comparative results of his studies were interesting: that is, the velocity of flow in the smooth-jointed pipe was found to be 15 to 20 per cent greater than that in the pipe with rough joints.

Another factor which was found to exert a strong influence on flow velocities was poorly laid pipe.

Yarnell and Woodward[3] conducted experiments on 12-inch tiles, properly and improperly laid and having different grades with these comparative results:

1) For pipes flowing approximately full, the velocity of the properly laid pipe was 3%, greater or less, than that for poorly laid pipe.

2) For pipes flowing at about 0.4 depth, the velocity in the correctly laid pipe was 32-53% greater than that for the improperly laid pipe. Furthermore, the differences in velocities appeared to increase with increasing slope for this depth of flow.

Little reliable information is available with regard to velocities and depths of flow which prevent deposition. However, about 1900 Metcalf and Eddy[4] made an extensive survey of the then current practice and solicited opinions from practicing engineers with regard to minimum allowable velocities of flow in sanitary sewers. As a result of these questionnaires and their own experiences, Metcalf and Eddy recommended a minimum full-pipe velocity of two feet per second in the early edition of their classic volume Sewerage and Sewage Disposal. It was the belief of those authors that sewers which flowed full at a velocity of two feet per second or more experienced little difficulty from blockage due to depositions. Because of the stature of the Metcalf and Eddy consulting engineering firm, their recommended two foot per second minimum velocity became the most universally accepted convention throughout the nation. Since that time, text books and authorities have used, and still use, the two foot per second value as a standard for the design of sanitary sewers.

Transportation of Particles

Around 1910 E.C. Murphy[5] studied the behavior of a stream carrying sand by noting sand particle motion through glass sections of a channel. Murphy observed that the movement of sand depended mainly on the velocity of flow, specific gravity of the particles, particle size and shape, and the roughness of the bed. It was also noted that flat particles travelled by sliding; those nearly spherical by rolling or by a combined rolling and jumping. The jumping motion of the particles was more or less dependent on the particle's specific gravity and the velocity of flow. The less the specific gravity and the greater the flow velocity, the higher and longer was the jump.

Perhaps the most pertinent observation made by Murphy was on the carrying capacity of a stream. Murphy found that a stream carried more than 4 times the weight of 3.5 - 5.0 mm. sizes in the presence of 0.27 - 0.31 mm. particles than in their absence.

Depth of Flow and Deposition

In 1937 Watson[6] proposed quite a different view for the cause of deposition. Watson contended that velocity although important, was a secondary consideration when compared to the effect of depth of flow in the sewer: " . . . in circular sewers at all events, and more particularly in the average branch up to 30 inches diameter, the basic cause of silting is insufficient depth of flow, rather than velocity; and especially is this evident in lateral sewers which only receive intermittent discharges. . .".

Watson's views were upheld by W.M. Ogden[7] in a paper presented in 1937. Ogden believed that velocity was not the sole criterion for self-cleansing pipes, but that depth of flow and pipe diameter were also important parameters for preventing deposition. In this same regard, L.B. Escritt[8] pointed out that observation and experiment indicated that large diameter sewers required a greater velocity for self-cleansing than small diameter pipes.

Type of Material Transported by Sanitary Sewers

Ogden[7] also held that when dealing with separate sanitary sewers, it was necessary to keep in mind the character of the bulk materials being transported. Since most transported materials in sewers have a low specific gravity (close to that of water), Ogden believed that self-cleansing requirements were mainly functions of pipe smoothness and depth of flow. Ogden's paper also mentioned another important consideration: that many researchers had neglected the effect of suspended material in the water when investigating scouring velocities.

Pipe Roughness and Deposition

Shields[9] approached the problem of sediment transport by assuming that the force exerted on a particle by moving water could be determined from "drag" relationships. Shields proposed the following:

1) Critical tractive forces were related to the initiation of general bed movement.

2) Particle size and weight and bed roughness were important parameters.

3) At advanced stages of transport, particle diameter was no longer pertinent in representing the bed roughness.

Camp[10] stated the above relationships from Shields in a mathematical expression which included the above parameters and one dimensionless constant, as follows:

Slope for bed movement, $s = \frac{\beta}{R}\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)$ (1)

Velocity for bed movement, $V = \sqrt{\frac{8\beta}{f} g \left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right) d}$ (2)

Where γ_s and γ_w were the specific gravity of the particle and of water, d the particle diameter, β a constant which related to bed movement and f was the friction factor of the Darch-Weisbach equation. Camp indicated that for bed movement, a β value of 0.04 was usually required. Complete scouring with saltation (complete suspension of particles) resulted from a β of 0.80.

Examination of Equation (2) has shown that the velocity required for bed movement is a function of the friction factor f. Usually, equation (2) has been expressed in terms of Manning's "n" by substitution of equation (3) for f.

$$f = n^2 8g / (1.486)^2 R^{\frac{1}{3}} \quad (3)$$

$$\text{or} \quad V = 1.486/n \; R^{\frac{1}{6}} \sqrt{B\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right) d} \quad (4)$$

Camp held that the ability of a steam to transport sediment was dependent on the mean velocity and friction factor. Since the friction or tractive factor was found to increase with decreasing depth, Camp's hypothesis was to the effect that less velocity was required for self-cleansing at lower depths of flow than for pipes flowing full.

Observations by Green[11], a consulting engineer, led him to believe that the roughness coefficient was influenced by smoothness of the pipe material and the care with which the joints were constructed. It is interesting to note that Green placed great emphasis on joint construction. However, this view was not unanimous; Camp[12] expressed the opinion that sewer joints have little effect on roughness.

Manning's "n" for Sanitary Sewers

Experiments of Wilcox[13] and Yarnell and Woodward[3] indicated that the values of Manning's "n" for clean sewer pipe and drain tile were between 0.0095 and 0.011. An interesting feature of their work was the apparent increase of "n" with slope for a constant depth of flow.

Yarnell and Woodward have conducted the most comprehensive tests to date upon the determination of "n". Moreover, their work included studies on the most commonly used pipe sizes as well as the different pipe materials. In addition to determining "n", Yarnell and Woodward also used their data to derive equations for sewer pipe flow.

Another important qualification relative to "n" was made by Camp[12], who stated that "n" varied with pipe size because of the effective absolute roughness of the pipe's interior.

In 1952, Cosen[14] made an extensive study of "n" for clay and asbestos pipe at slopes of 0.25% and 0.4%. The results of Cosen's experiments showed close agreement with "n" values as obtained by Wilcox, and Yarnell and Woodward.

A study recently completed by Bloodgood and Bell[15] revealed that "n" varied with flow, pipe diameter and pipe slope as well as with depth. The conclusions reached by Bloodgood and Bell from their studies on 8-inch pipe were:

1) "n" values were smaller for a given pipe size at an 0.4% slope than they were for a pipe at an 0.25% slope.

2) At a slope of 0.4%, "n" values ranged between 0.00957 and 0.01105 for clay pipe.

3) The rate of flow affected "n" values.

4) "n" values differed considerably for 4-inch and 8-inch pipe line tests.

5) The ratio of depth of flow to pipe diameter, d/D,

appeared to be the principle factor in variation of "n" values.

Sliming of Sanitary Sewers

As a result of work previously mentioned, Cosen[14] made this interesting proposal: The sliming of pipe walls, which is common to all sewers, results in an "n" value which is the same for all pipe materials when other conditions are equal.

Cosen's opinion in this regard agrees with recent research on slime effects. A study conducted by Reid[16] to determine the effect of slime on pipe walls showed that slime growth reduces pipe velocity with time. Furthermore, testing indicated that after eleven weeks the velocities in glazed, unglazed, asbestos and concrete pipes were approximately the same for any given flow. The conclusions reached in this study, as quoted directly from Reid's report, were:

" . . . 1) The slime growth decreases as the sewer velocity increases even if discharge is increased.

2) The rougher and more porous pipe surfaces sustain a greater amount of slime growth than the smoother surfaces, in other words, roughness affects slime growth. For example, slime growth in asbestos-cement pipe is about twice that of glazed pipe.

3) The sewer velocity is reduced by resistance offered by slime growth with a consequent reduction in sewer flow capacity -- slime growth affects flow capacity.

4) There is a balance point between cohesive force produced between the pipe surface and slime layers and tractive force. This balance point lies between the 10th and 12th week, but depends upon velocity and discharge of flow and roughness of pipe surface.

5) Slime growth tends to be unimolecular and follows a geometric curve.

6) From the sulfides test, it is shown that odors in partial flow under proper ventilation come from slime and not sewage.

7) Sulfides produced by slime are increased with slime quantity and time. In other words, odor is produced due to slime but not by sewage and it varies with amount of slime growth in the pipe.

8) Sulfides produced by slime is increased with time but tends to be geometric.

9) It is interesting to know that the velocity in asbestos-cement pipe is higher than that of the other three pipes even if slime growth in asbestos-cement pipe is highest. This is because there are fewer joints in asbestos-cement pipe than in the others and small eddies occur below each joint in the pipe.*. . . "

" . . . *Note: Individual length in asbestos-cement pipe =7'
" " concrete pipe =3'
" " glazed clay pipe =2'
" " unglazed pipe =2'
. . ."

Johnson[17] was of the opinion that the surface condition of the pipe walls changes with the pipe's age. Furthermore, Johnson felt that "n" values should be investigated in conduits carrying sewage and not clear or river water as had been done in many of the investigations.

Engineering Practice, Observation and Thought

Presently, the general rule[18] used in designing sewage collection systems is to obtain a flow velocity of two feet per second for the pipe flowing full. Presumably, this is established so that sufficient eroding velocities exist at flows less than full. Less than full conditions can be illustrated by two typical situations:

1) L. B. Escritt[8] reported that studies conducted in Australia to determine the flow velocity in 21 sewers showed that pipes were generally self-cleansing with velocities lower than 2 feet per second. The observed velocities ranged from 0.37 to 1.75 feet per second and in none of the cases did the sewers require flushing when the velocity was 1.24 feet per second or more. Unfortunately, Escritt made no specific mention of flow depths.

2) Experiments by Hatton[4] on two 24-inch sewers discharging creek water and carrying considerable clay showed no appreciable deposition when flowing at a depth of 5 inches with a velocity of 1.31 feet per second.

Since sewer grades define pipe line velocities, velocities can be discussed in terms of slopes (viz., for 8-inch pipe, a 0.4% slope produces a full flow velocity of two feet per second). The text, <u>American Sewerage Practice</u>[4], by Metcalf and Eddy lists exceptions to the rule that the minimum gradient for an 8-inch pipe be 0.4%. Two such cases were reported by J.R. McClintock and James H. Hazelhurst. McClintock revealed that examinations of the sewers in Englewood, New Jersey, showed a number of pipe lines with low grades which operated satisfactorily. In particular, several 8-inch pipes at grades of 0.10%, 0.15%, and 0.20% performed adequately. Hazelhurst, an engineer who had practiced extensively in the southeastern coastal states, stated that problems seldom existed where grades for 8-inch pipes were at least 0.25%, providing that the sewers were properly constructed.

Green[11] reported that a collection system in Mendota, Illinois, with 8-inch tiles at a grade of 0.25%, gave no trouble (no deposits and clean pipe surfaces) over a period of 20 years.

As previously mentioned, correspondence by this author with Charles A. Marco, Assistant City Engineer of Visalia, California, revealed that the city had constructed numerous 8-inch lines on grades from 0.15% to 0.20%. With these grades, blockage problems have not resulted.

On the other hand, the author's survey of 52 Michigan cities showed that many of the engineers felt that flat grades were definitely one of the major causes of blockage. This trend of thought was reflected in the opinions of superintendents and engineers as reported to the Boston Society of Civil Engineers.[9] Here, the general concensus of opinion was the same as that mentioned previously for current practice; that is, employing a slope of 0.4% which produces a full flow velocity of two feet per second.

An important variable in the design of sewer systems is the selection of the roughness coefficient. Typical values employed in practice can be illustrated by the following paragraphs.

In 1928 Alexander Potter reported to Metcalf and Eddy[4] that his observations on a sewer system in New Jersey composed of vitrified pipe and small brick sewers indicated the average Kutter "n" value to be 0.014. Metcalf and Eddy also reported that studies by S.M. Cotten on vitrified clay pipes of diameters 24, 30 and 36 inches showed Kutter's "n" values of 0.0111, 0.0117, and 0.0125 respectively. It should be noted that the "n" values as computed by Kutter and Manning are in close agreement.

Tests conducted by the City of Visalia, California, on their sewer system showed the value of Manning's "n" to be 0.011. Modesto, California, indicated that tests on their system justified the continued use of an "n" equal to 0.013.

The most frequently selected Manning's "n" value in modern practice is 0.013. This value has been arrived at by experiment, observation and the experience of designing engineers. The majority of engineers, reporting to the Boston Society of Civil Engineers[9], also recommended the use of an "n" value of 0.013.

Observations on the cause of blockage in sewer systems vary considerably. Concerning stoppages, the following replies are typical of those from the 1933 "Round Table"[19] discussion in response to the question

"What are the chief causes for sewer stoppage or obstrution?": Columbus, Ohio, replied that tree roots, broken pipes and changes in alignment were the main stoppage factors and Shreveport, Louisiana, stated that grease on the pipe walls, tree roots and foliage and improper use of the sewer were responsible for blockage problems. The "Round Table" listed 32 replies which represented the nation geographically. Of the replies, only 2 made reference to sand stoppage and only 1 to flat grades as the primary causes of blockage.

IV THEORETICAL CONSIDERATIONS

The problem of deposition can be examined analytically by the general principles of statics and hydraulics. Any consideration of self-cleansing velocities must include the effects of traction, friction factors, invert gradient, and particle size and specific gravity.

Tractive Force

A particle resting on a pipe invert is held in its position by the friction force between the particle and the pipe invert. The friction force involved must be defined to include the effects of the pipe invert and pipe joints. For the particle to move, it is necessary that the traction force exerted by the flowing medium on the particle be equal to or greater than the friction force. Therefore, for impending motion, the tractive force is equal to the friction force.

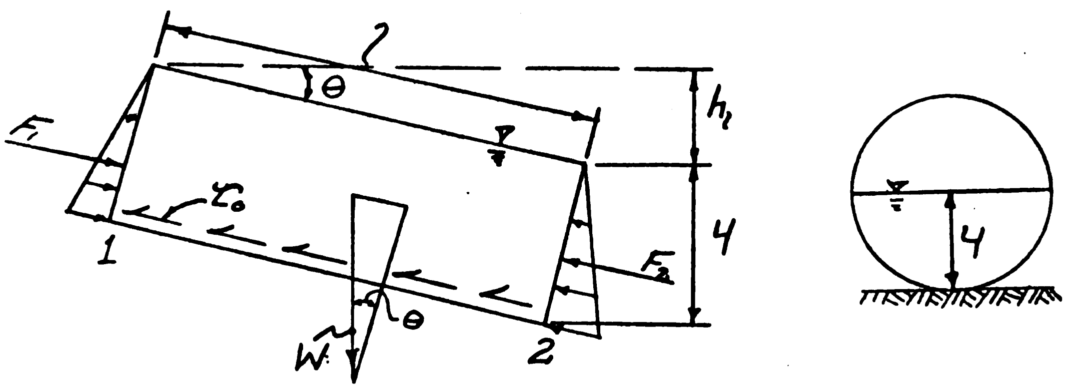


Figure 4-1. Free Body of Pipe

The average tractive shear force acting on the invert and joints can be developed from the free body diagram shown in Figure 4-1 where θ is the small angle representing the sewer slope, τ_o is the average unit shear force, ρ is the wetted perimeter and l is the length of the free body section.

Equilibrium in the θ plane requires that:

$$F_1 + W\sin\theta - F_2 - \rho\tau_o l = 0$$

For uniform flow or small l distance, $F_1 = F_2$, then

$$W\sin\theta = \rho\tau_o l \qquad (1)$$

Letting sine $\theta = \theta$ for small angles we have $\theta = \frac{h_l}{l}$. The weight W of the flowing medium in the free body is $W = A\gamma l$ where γ equals the liquid unit weight; then equation (1) can be written as:

$$\rho\tau_o l = \frac{Wh_l}{l} = \frac{A\gamma l h_l}{l}$$

or

$$\tau_o = \frac{A\gamma h_l}{\rho l}$$

Noting that $\frac{A}{\rho}$ equals the hydraulic radius R and $\frac{h_l}{l}$ the pipe slope s for uniform flow, we have:

$$\tau_o = R\gamma s \qquad (2)$$

Equation (2) represents the average unit tractive force of the flowing medium on the pipe invert.

Self-cleansing Velocities

To derive an expression for scouring velocities, assume the particle shown in Figure 4-2 is acted on by the tractive or drag force T and a friction force, F.

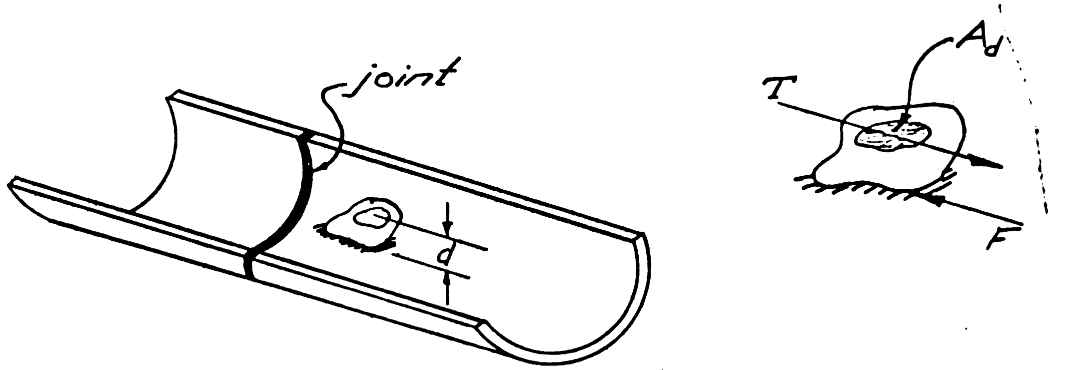


Figure 4-2. Particle on Pipe Invert

For impending motion, the sum of the forces must be zero. F, the friction force, is equal to the normal weight N of the particle times the friction factor f_s when f_s is the average friction coefficient between the particle and the pipe invert and joints. Therefore, when A_d is the mean particle surface area:

$$F = T = \tau_o A_d$$

or,

$$T = \gamma R s A_d = f_s N$$

The weight N of the particle submerged in water can be expressed as:

$$N = (\gamma_s - \gamma)(1 - p_s) A_d d$$

where γ_s is the particle unit weight, p_s the porosity, d the mean particle diameter and $A_d(d)$ the particle volume.

The tractive force can be expressed as:

$$\gamma R s A_d = f_s (\gamma_s - \gamma)(1 - \rho_s) \cdot A_d d$$

Letting S equal the critical self-cleansing slope and the constant ϕ equal $f_s(1-\rho_s)$* we have:

$$S = \phi(\gamma_s - \gamma)\, d \Big/ R\gamma \quad (3)$$

The velocity required for scour can be obtained from the Manning Equation by substitution of the critical self-cleansing slope S. Thus:

$$V = \frac{1.486}{n} R^{\frac{2}{3}} \sqrt{\frac{\phi(\gamma_s - \gamma)d}{R\gamma}}$$

or removing 1/R from the radical, the velocity is:

$$V = \frac{1.486}{n} R^{\frac{1}{6}} \sqrt{\frac{\phi(\gamma_s - \gamma)d}{\gamma}} \quad (4)$$

Equation (4) therefore indicates that the velocity required for scour of a particle is a function of: (1) the flow depth, (2) pipe friction, (3) a variable parameter ϕ and (4) the particle size and unit weight. It should be noted that equation (4) is analogous to equation (4) of Section III except for the constant notation. Also note that an appropriate average "n" for sewers includes the effect of the joints. The value $\frac{(\gamma_s - \gamma)}{\gamma}$ is an expression of the relationship of the specific gravity of the particle material and the hydraulic parameter R in equation (4) is a function of the pipe size and the flow depth.

*see Shields' consideration(2), p. 30

Self-Cleansing Velocities at all Depths of Flow

For a sewer of given diameter, self-cleansing velocities at different depths of flow require that the unit tractive force remain constant. For the derivation of a relationship which provides constant tractive force, it is convenient to designate lower case letters for flow less than full and upper case letters for full flow. $t = T$ where t is the unit tractive force for the sewer partially full and T is the unit tractive force for the sewer flowing full. Thus:

$$\gamma r s = \gamma R S$$

and,

$$s = \frac{R}{r} S \qquad (5)$$

The self-cleansing velocity at all depths of flow can again be obtained from the Manning Equation.

$$v = \frac{1.486}{n} r^{2/3} s^{1/2} \qquad (6)$$

$$V = \frac{1.486}{N} R^{2/3} S^{1/2} \qquad (7)$$

Dividing equation (6) by equation (7) we obtain:

$$\frac{v}{V} = \frac{N}{n} \left(\frac{r}{R}\right)^{2/3} \left(\frac{s}{S}\right)^{1/2} \quad (8)$$

Equation (8) gives the relationship which defines the ratio of $\frac{v}{V}$ for all depths of flow. Substituting R/r for s/S from equation (5), we have a function for determining the ratio of $\frac{v}{V}$ for self-cleansing velocities at all depths:

$$\frac{v}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{\frac{1}{6}} \qquad (9)$$

<u>Tables for Self-Cleansing Velocities and Slopes</u>

The three basic equations for self-cleansing slopes and velocities are:

$$S = \frac{\phi(\gamma_s - \gamma)d}{R\gamma} \quad \text{or} \quad s = \frac{\phi(\gamma_s - \gamma)d}{r\gamma} \qquad (3)$$

$$V = \frac{1.486}{N}(R)^{\frac{1}{6}}\sqrt{\frac{\phi(\gamma_s - \gamma)d}{\gamma}} \qquad (4)$$

$$\frac{v}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{\frac{1}{6}} \qquad (9)$$

These equations can conveniently be expressed in the tabular form of Table 4-2.

Columns (1), (2), (3) and (5) in Table 4-1 can be found in any standard hydraulic text. Column (4) is the r/R value of column (3) raised to the 1/6 power. Column (6) is obtained from equation (9) for the different depths of flow.

Table 4-2 has been constructed from the data of Table 4-1. In this second table, the required slope and velocity for self-cleansing of an 0.5 mm. particle in an 8-inch pipe with a full flow "n" of 0.012 are shown for different depths of flow. It has been assumed that an "n" of 0.012 sufficiently represents the overall average friction coefficient. Construction of Table 4-2 has been as follows: Column (2) the hydraulic radius, found in any standard hydraulic text; Column (3) the self-cleansing slope, from equation (3), Column (4) the required self-cleansing velocity resulting from equations (4) and (9), and, Column (5) the flow, determined by the product of area of flow and velocity of flow. The ϕ value used in equations (3) and (4) is 0.08.

The determination of ϕ for the construction of Table 4-2 was based on the work of Shields[9]. Shields' work showed that ϕ may lie between 0.04 and 1.0. A value of 0.04 was necessary to start movement; and, adequate cleansing resulted with a ϕ value of 0.08. Shields indicated that a ϕ value of 0.8 caused the particle to be transported by suspension in the flowing medium. The use of Table 4-1 in calculating Table 4-2 can perhaps best be illustrated in the following example.

By equation (4) the velocity at full flow required for self-cleansing is:

$$V = \frac{1.486}{0.012} (0.167)^{\frac{1}{6}} \sqrt{0.08 \left(\frac{2.65-1.0}{1.0}\right) \frac{0.5}{304.8}} = 1.35 \text{ ft/sec}$$

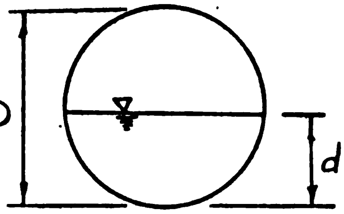
and from equation (3) the required slope is:

$$S = \frac{0.08}{0.167} \left(\frac{2.65-1.0}{1.0}\right) \frac{0.5}{304.8} = 0.130\%$$

Now consider the case where the pipe is flowing 0.4 full ($d/D = 0.4$). From Table 4-1 or equation (9) $\frac{v}{V} = 0.77$;

Table 4-1. Hydraulic Elements of Sewer Pipe

(1) d/D	(2) a/A	(3) r/R	(4) $r/R^{1/6}$	(5) N/n	(6) v/V
1.0	1.00	1.00	1.00	1.00	1.00
0.9	0.95	1.19	1.03	0.93	0.96
0.8	0.86	1.22	1.03	0.89	0.92
0.7	0.75	1.19	1.03	0.85	0.88
0.6	0.63	1.11	1.02	0.82	0.84
0.5	0.50	1.00	1.00	0.80	0.80
0.4	0.37	0.86	0.98	0.79	0.77
0.3	0.25	0.68	0.94	0.78	0.73
0.2	0.14	0.48	0.89	0.79	0.70
0.1	0.05	0.25	0.80	0.81	0.65



d = flow depth
a = cross-section area of flowing sewage
r = hydraulic radius
n = Manning's "n"
v = velocity
q = sewage flow in gpm
s = slope required (in per cent) for self-cleansing

Table 4-2. Computed Minimum Velocities and Slopes for Self-Cleansing of an 8-inch Pipe

(1) d/D	(2) r (ft)	(3) s	(4) v (ft/sec)	(5) q (gpm)	
1.0	0.167	0.130	1.35	212	
0.9	0.198	0.109	1.30	194	
0.8	0.203	0.107	1.24	167	
0.7	0.198	0.109	1.19	140	
0.6	0.185	0.117	1.13	112	
0.5	0.167	0.130	1.08	85	
0.4	0.143	0.152	1.04	61	Illust.
0.3	0.113	0.192	0.99	39	Examp.
0.2	0.080	0.271	0.95	21	
0.1	0.042	0.517	0.88	7	

Data for Table 4-2: 8-inch pipe diameter
(N)full = 0.012
0.5 mm. sand grain on invert
ϕ value of 0.08

therefore, the velocity (v) required for self-cleansing at this depth of flow is:

$$V = 0.77\,(1.35) = 1.04 \text{ ft/sec}$$

Obtaining r from Table 4-1 for a flow of 0.4 full, the necessary slope is:

$$s = \frac{0.08}{0.143}\left(\frac{2.65-1.0}{1.0}\right)\frac{0.5}{304.8} = 0.152\,\%$$

V MATERIALS AND APPARATUS

Since this investigation was concerned with minimum allowable grades for sanitary sewers, it was felt that the testing should simulate actual operating conditions. All the studies were therefore conducted with an experimental 8-inch vitrified clay sewer pipe line at the Mason, Michigan, Sewage Treatment Plant. Raw plant sewage was used in all experiments.

Particles Studied For Deposition

To study the influence of the pipe line slope, depth of sewage flow and particle size on the transporting capacity of sewage, sand particles were sieved into the sizes shown in Figures 5-1 and 5-2. The particles were used in a manner which will subsequently be described.

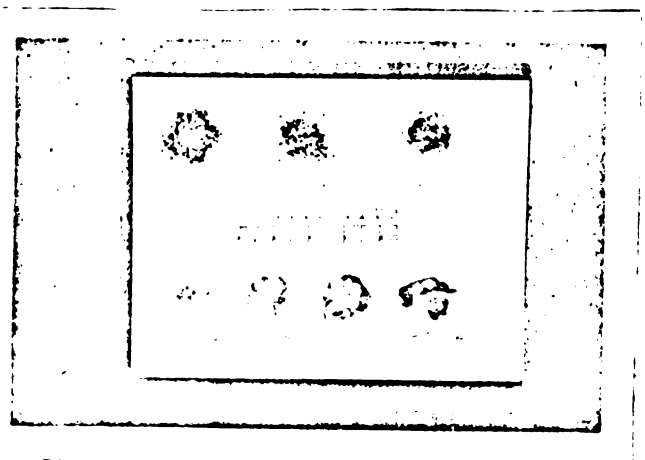


Figure 5-1. Particles Used in Studying Deposition (Specific Gravity of 2.7)

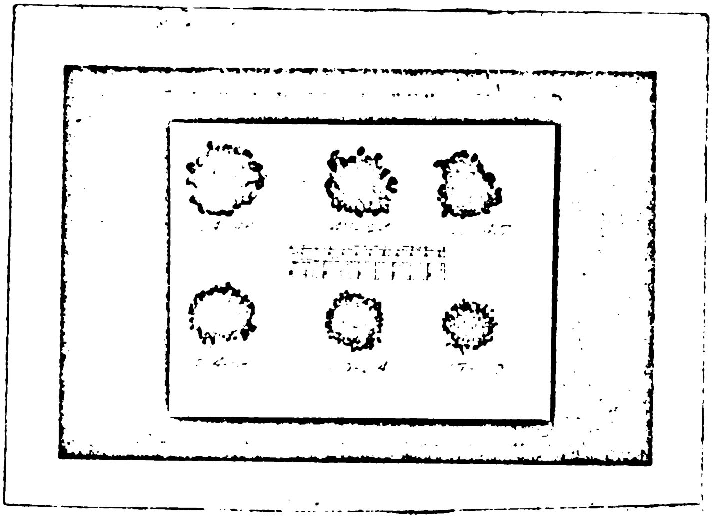


Figure 5-2. Particles Used in Studying Deposition (Specific Gravity of 2.7)

Sand was used for the deposition studies because of the easy determination of the particle size and specific gravity. Moreover, sand's higher specific gravity causes it to deposit more readily than organic material; a situation frequently found in sanitary sewers.

Apparatus for Influent Regulation

As shown in Figure 5-3a, the incoming plant sewage used in the experiments was lifted 11 feet from the plant junction box A to an overhead weir B by use of a 350 gpm Fairbanks Morris Sewage Pump C. 4-inch steel pipes were used on both the suction and discharge lines of the pump. Included on the discharge line was a globe valve D to regulate the quantity of sewage flowing into the experimental sewer pipe line.

The quantity of flow delivered to the experimental sewer line was measured with a 60° brass triangular weir E (Figure 5-3b) located at one end of a 1.5' x 2' x 4' leak-proof weir box F)Figure 5-3b). The weir box was constructed of marine plywood. Since the pump discharged downward into the weir box, a baffle system was installed to still the sewage and thus permit uniform effluent velocity.

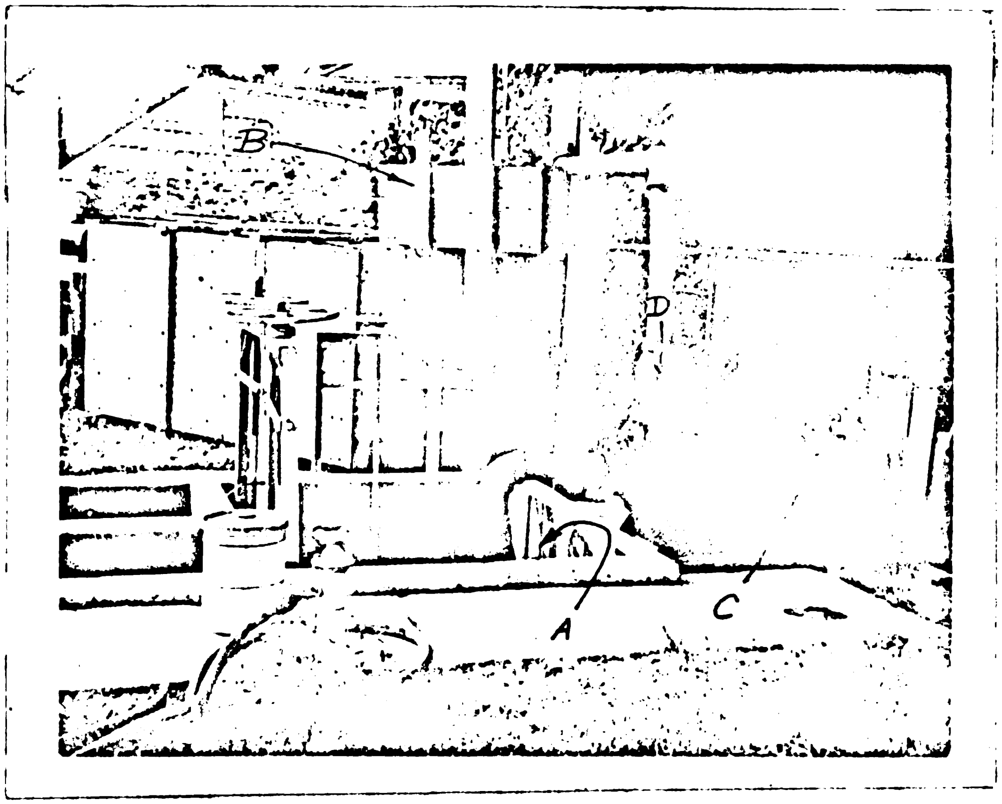


Figure 5-3a. Pumping Raw Sewage to the Surge Tank

A 4' x 4' x 8' leak-proof surge tank G(Figure 5-3b), constructed of 3/4-inch marine plywood, received the discharge from the weir box. The tank was reinforced with 2 x 4's (Figure 5-5) in such a manner that the bending stresses created by the hydrostatic pressures did not exceed 1600 psi. A baffle (Figure 5-5) was placed in the

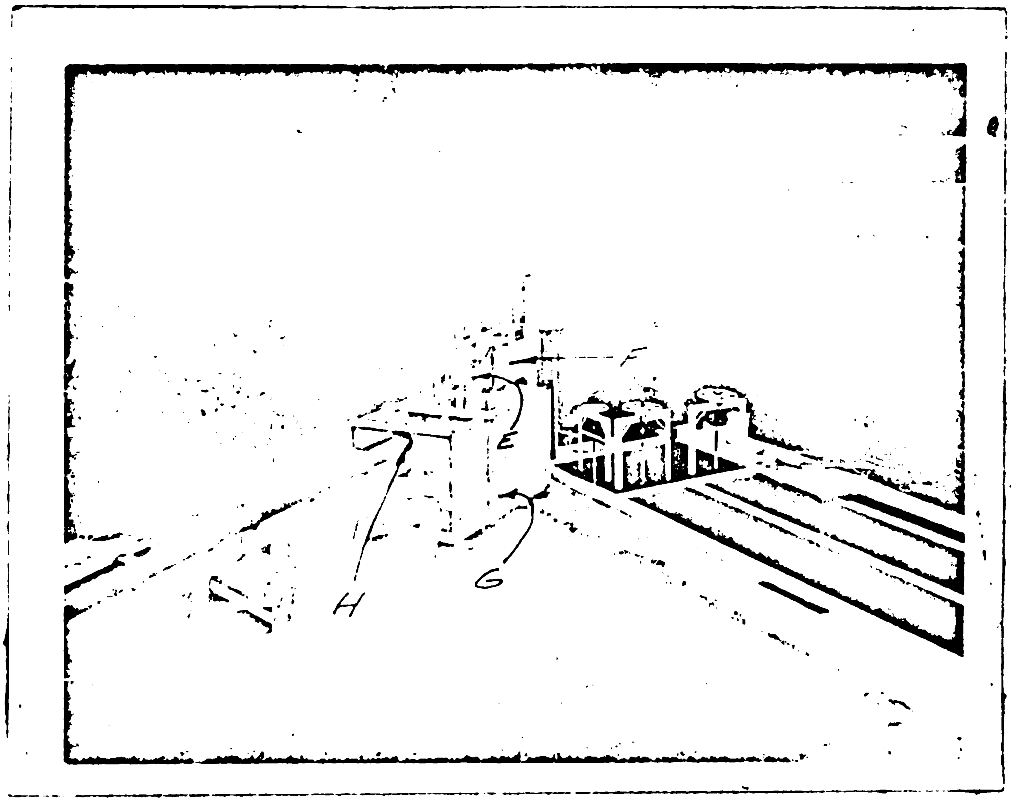


Figure 5-3b. Surge Tank, Weir Box and Pipe Line

tank for stilling and velocity distribution. The pipe line entrance H(Figure 5-3b) was located at one end of the surge tank; allowing the sewage to enter the pipe line with a minimum of disturbance.

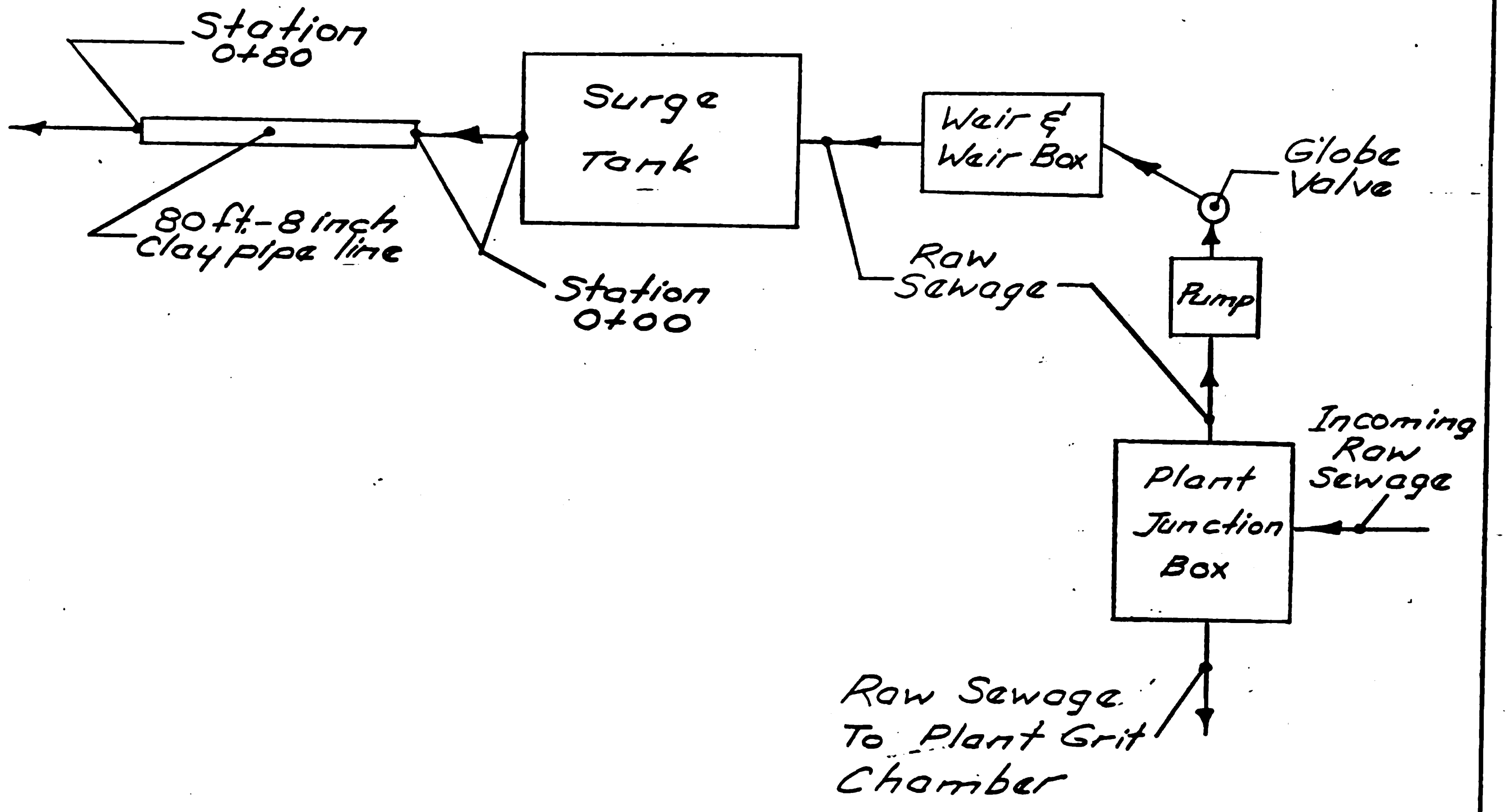
The Pipe Line

The experimental 80-foot long, 8-inch vitrified clay sewer line (Figure 5-6), was constructed of 3-foot pipe sections and 2-foot Tee sections. The Tees (I of Figure 5-7) were used for observation and for placing sand in the line to study deposition. The pipes were jointed together with slip-seal joints.

To study deposition at varying pipe slopes, a trestle system (Figure 5-7) was used to support the pipe in a

Schematic Plan Of Sewage Flow

Figure 5-4



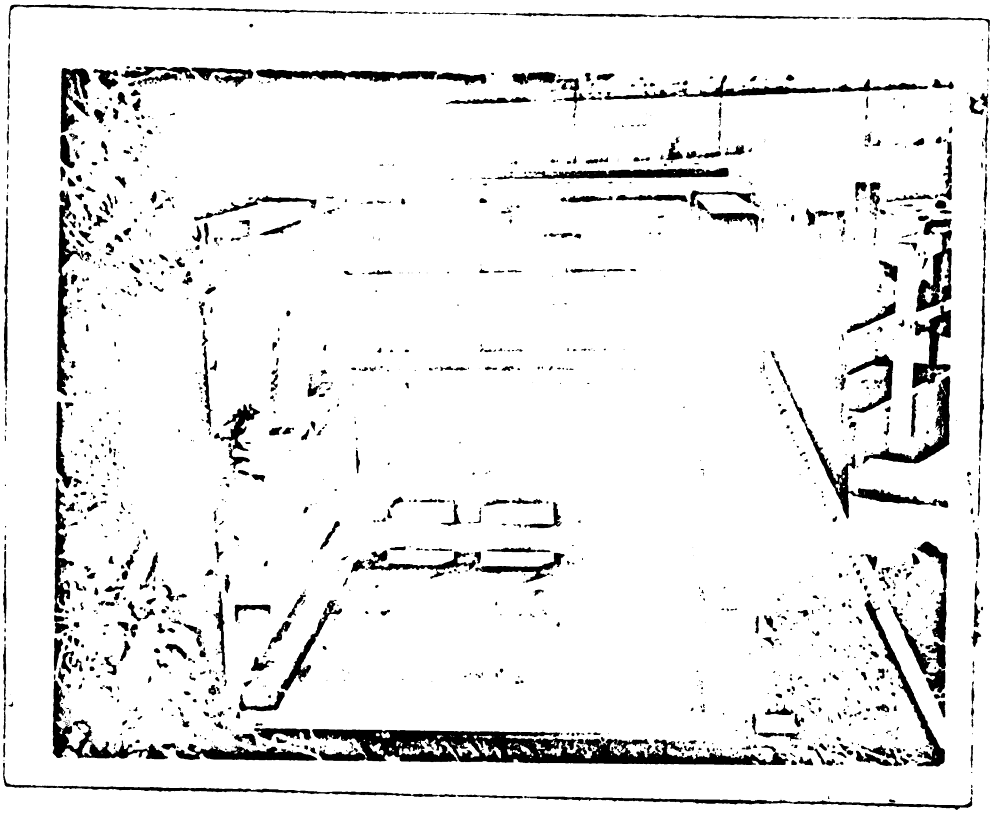


Figure 5-5. End View of Surge Tank During Construction

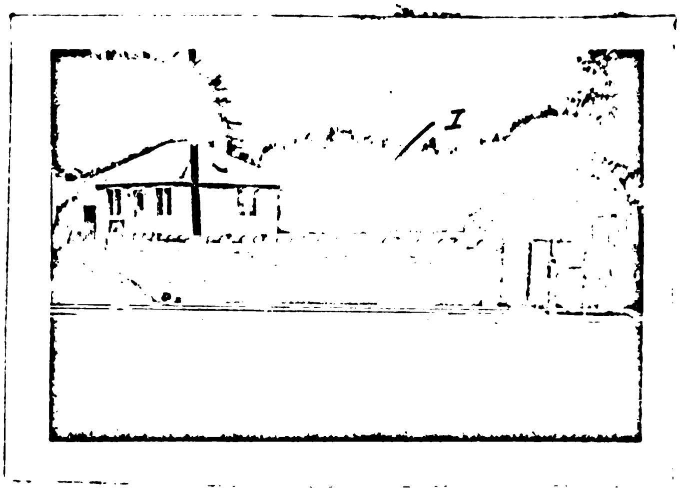
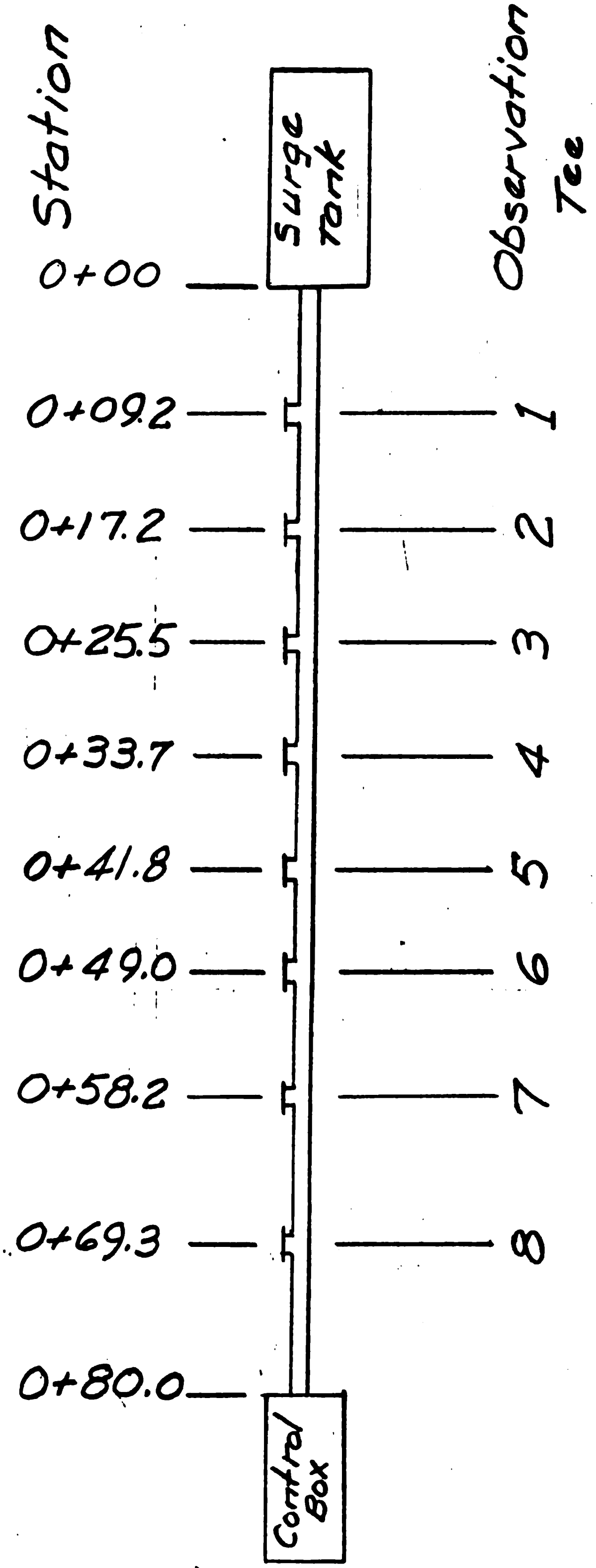


Figure 5-6a. Experimental Pipe Line

manner which allowed the line to be set at any desired grade. The members of the trestle were designed by the commonly accepted stress values for Douglas Fir Lumber.



Location of Pipe Line Observation Points

Figure 5-66

The main design criterion was a limiting pipe line deflection of 0.001 feet. This requirement determined the span between adjustable vertical supports to be approximately 9.6 feet for an estimated maximum loading of 40 lb/ft.

The pipe line between vertical supports was supported with two 2" x 8" planks J set on edge. The planks in turn rested on adjustable 4-foot long 2" x 6" horizontal members K. The horizontal members were secured to 3-foot high 2" x 6" vertical supports L, which were cross-braced with 1" x 4" boards M. To prevent settlement, the vertical supporting columns rested on 2" x 6" planks N, which acted as bearing plates.

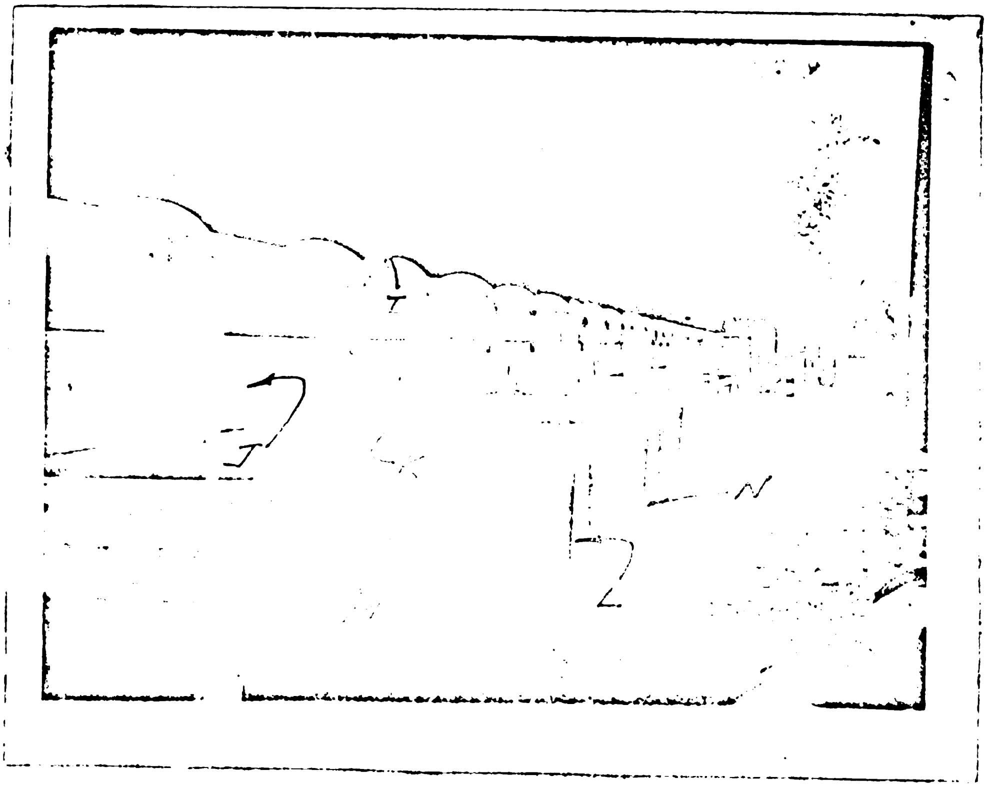


Figure 5-7. Supporting Mechanism of Pipe Line

Table 5-1. Experimental Pipe Line Stationing

Station*	Part Description
0 + 00	Surge Tank & Start of Line
0 + 01.1	Vertical Support
0 + 09.2	Vertical Support & Observation Tee (1)
0 + 17.2	Observation Tee (2)
0 + 19.2	Vertical Support
0 + 25.5	Observation Tee (3)
0 + 28.8	Vertical Support
0 + 33.7	Observation Tee (4)
0 + 38.4	Vertical Support
0 + 41.8	Observation Tee (5)
0 + 48.0	Vertical Support
0 + 49.0	Observation Tee (6)
0 + 57.5	Vertical Support
0 + 58.2	Observation Tee (7)
0 + 67.1	Vertical Support
0 + 69.3	Observation Tee (8)
0 + 76.7	Vertical Support
0 + 80.0	End of Line & Control Box

*Stationing is in the direction of flow

The vertical column supports L were slotted a distance of 2 feet down from the top. The horizontal members were slotted for distance of 1 foot from the ends. The horizontal bars were secured to the vertical column supports by ½-inch steel bolts placed through the slots as shown in Figure 5-7. With this arrangement, the entire line could be set at any grade with full assurance of proper alignment.

Producing Uniform Flow

Engineering calculations relative to sewer pipe lines usually assume uniform flow of sewage. Since uniform flow conditions greatly simplified this initial study, that type of flow was provided in all experimental work described here. Apparatus to produce this effect was constructed as shown in Figure 5-8.

Uniform flow was achieved by placing a 1.5' x 2' x 4' leak-proof control box, made of 3/4-inch marine plywood, at the end of the pipe line. The control box was fitted with a sluice gate O which permitted regulation of the sewage elevation at the outlet end of the line. The gate was positioned for each experiment to produce a uniform depth of flow for the entire line.

Miscellaneous Apparatus and Equipment

From the outlet control box, the sewage discharged into a 1' x 2' x 2' junction box P (Figure 5-8) constructed

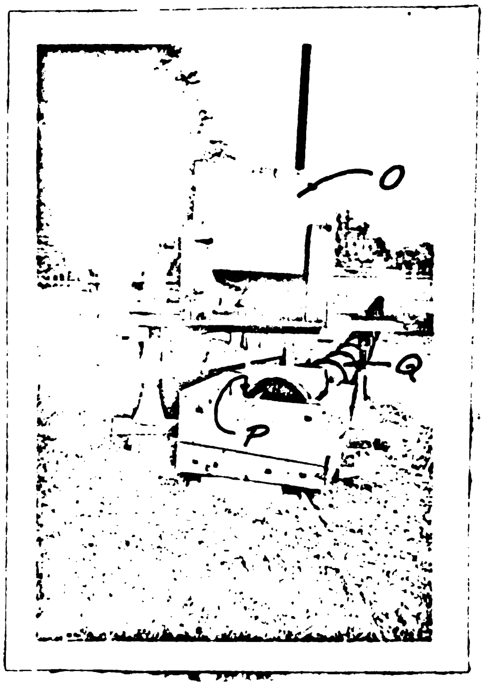


Figure 5-8. Outlet Control Box and Return Pipe Line (Prior to Alteration)

of 3/4-inch marine plywood with leak-proof joints. The junction box directed the sewage to an 8-inch clay pipe line Q (Figure 5-8), which carried the sewage to the plant grit chamber for treatment.

Other miscellaneous equipment employed in the study were:

1) A surveyor's level for setting grades and elevations
2) Screw jacks for adjusting the pipe line slope
3) A metric scale for determining weights of sand used in studying deposition
4) A set of Tyler sieves
5) A pocket mirror for observation of pipe interior
6) An engineer's rule for measurements

7) A tank, scale and stop watch to calibrate the
60° triangular weir.

VI EXPERIMENTAL PROCEDURES AND RESULTS

Testing the Apparatus

Prior to conducting the studies which are described in this section, it was necessary to test the equipment and to evaluate its effectiveness for the desired experimental work. Preliminary testing was therefore necessary to answer these questions:

1) Would the pump produce a steady flow? If not, what corrective steps were necessary?

2) Were the baffles adequate for stilling and velocity distribution in both the weir box and surge tank?

3) Were leaks present in any of the constructed apparatus?

4) What procedure was most suitable for changing the pipe line slope?

5) Could the control box produce the desired uniform flow?

6) What was the best method for measuring the depth of flow in the pipe?

7) What method was most suitable for placing sand in the pipes for studying deposition?

8) What problems would arise that had not previously been considered?

The trial testing produced satisfactory answers to all of the above questions and provided insights regarding the operation of the equipment. The testing indicated that the following changes or alterations in equipment were necessary:

1) At high rates of discharge, the pump developed whirlpools about the end of the suction line. This was corrected by installing vanes to deflect

the whirlpools.

2) The flow area through the weir box baffle was insufficient and had to be enlarged.

3) Several leaks were discovered which would have caused inaccurate measurements.

4) Uniform flow was extremely difficult to obtain. This was discovered to be due to the large cross-sectional area of the sluice gate. Decreasing the box gate size corrected this difficulty.

5) Tests indicated that the most desirable time for placing the sand in the line was after uniform flow conditions had been created. Otherwise, the sand on the invert caused a backwater effect which made it almost impossible to determine when uniform flow conditions had been achieved.

Establishment of Pipe Grades

After trial testing had been completed, a base grade for the pipe line was established. Since the elevation of the pipe invert at the surge tank was fixed, all grades were set in relation to that point. Due to the ground topography, it was desirable to select 0.5 per cent as the base grade and to set all other grades relative to this base. By means of a dumpy level, reference points (marks) corresponding to the elevation required for an 0.5% slope were set on each vertical support to the nearest hundredth of a foot. From these reference points, the grades for all experiments could easily be set and checked to maintain an accuracy of 0.01% slope.

Experimental Program

The experimental program consisted of 52 data recording tests. The number of tests conducted at the following slopes were:

Grade (per cent)	Number of Tests
0.15	4
0.20	6
0.25	7
0.30	9
0.35	11
0.40	7
0.50	8

For each pipe slope investigated, the quantity of flow was varied over a sufficient range to provide data on the flow conditions necessary for self-cleansing relative to the various particle sizes considered.

As an example, the four tests on the 0.15% pipe slope were conducted at 33, 41, 50 and 57 per cent of the full flow pipe depth.

The flow in the pipe line was regulated by the previously mentioned globe valve. By varying the pipe flow, it was possible to note the effect of depth of flow and flow velocity on self-cleansing. Thus, a low d/D meant small depth of flow and flow velocity in the pipe line.

The third variable, maximum particle size eroded, was dependent upon the pipe slope and the quantity of flow. The particle sizes studied had a size range of 0.1 to 9.5 mm. and an average specific gravity of 2.7.

It was then possible in each test to observe the pipe slope and depth of flow which caused a given sand particle size to erode from the pipe invert and to successfully pass through the entire pipe line.

Experimental Procedure For a Test Run

As an example of the test procedures employed, a test at 0.3% grade will be described:

- Adjusting the Pipe Line Grade -

Since this test run was the first to be made at the 0.3% grade, elevations were calculated for each vertical support. The support's horizontal bar was then raised a distance in feet above the base grade equal to (0.005 - 0.003) times the distance of the support from the surge tank. (At 0.5% grade, the top edge of the horizontal bar was at the same elevation as the base mark on the vertical support.) The elevation of the horizontal bar at a given vertical support was adjusted by lifting the pipe line span with a screw jack as shown in Figure 6-1. After lifting the span, the clamping bolts were loosened and the horizontal support was raised to the desired point. The clamping bolts were then tightened, permitting the span to be lowered back onto the support at the proper elevation.

- Collection of Sand Particles -

After the required grade was set, 150 gram portions of various sand groups were selected for placing at the observation points (See Figure 5-6b). The size groups were:

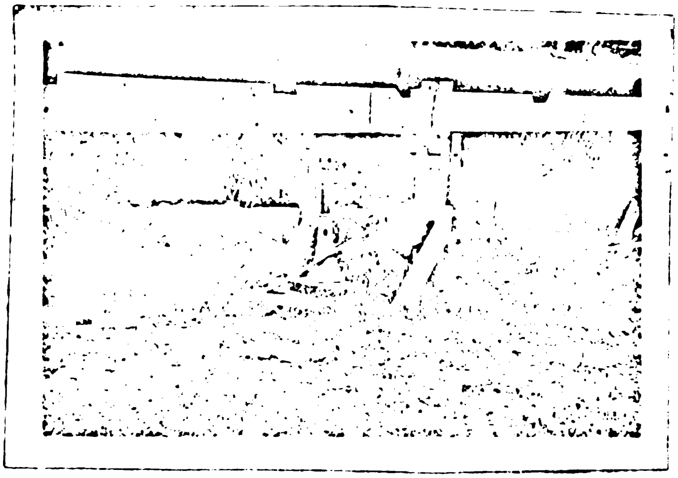


Figure 6-1. Screw Jack Used to Raise Pipe Line

Observation Point	Sand Group Size
2	6.4 - 9.5 mm.
3	4.7 - 6.4 mm.
4	3.3 - 4.7 mm.
5	2.4 - 3.3 mm.

- Pipe Flow Adjustment -

For the test run, the globe valve was opened to provide approximately the maximum attainable flow at the selected 0.3% pipe slope. In this particular test, the depth of flow d measured 0.28 feet for a $\frac{\text{depth of pipe flow}}{\text{pipe diameter}}$ or d/D of 0.42 at observation points 2 and 3, as shown in Figure 5-6a and 5-6b. Uniform flow was then achieved by adjusting the sluice gate in the control box until the depth of flow at observation points 4, 5, 6 and 7 measured within 0.01 feet of 0.28 feet. Flow depth was measured with an engineer's rule from a predetermined point on the upper lip of each pipe Tee to the sewage surface. The depth of flow was computed by subtracting this measured value from the distance between the reference point and the pipe invert.

It was noted that the pump surged due to the variable sewage flow entering the plant junction box. This surging sometimes caused the measured weir head reading to vary as much as -0.02 feet for the smaller flows with resulting changes in the pipe flow depth of as much as 0.04 feet. Therefore, particular attention was paid to the weir head on all runs and the pump globe valve was adjusted when required to provide a constant condition of uniform flow.

- Deposition Observations -

Once a uniform flow depth was established, the 150 gram portions of the various particle groups were placed in the flow through observation point openings 2 to 5. The largest group size (6.4 - 9.5 mm.) was placed on the pipe invert at point 2 and the other groups placed in decreasing sise at points 3, 4 and 5 respectively. No material was introduced at points 6, 7 and 8.

Immediately after placing the sand particles in the line, the weir head was measured by inserting an engineering rule into the weir box and noting the distance from the box bottom to the sewage surface. The previously determined distance of 0.33 feet between the box bottom and the V notch was subtracted from the weir reading to obtain the weir head. For this example, a value of 0.83 feet was recorded which indicated a weir head of 0.50 feet.

After a five to twenty minute time interval, the observation point invert was probed with a small wire to determine if any change had occurred in the amount of material previously placed on the pipe invert. If a considerable change was noted at the end of the time interval, the test run was considered complete. If no significant change was observed after an adequate period of time, the test run was also assumed completed. When a test was finished, the globe valve was closed and the line allowed to drain.

The final step for each experimental run was to flush the line of deposited particles. Flushing was accomplished by either opening the globe valve all the way and producing maximum flow or by employing a high pressure hose to wash out the deposited sand particles.

- Interpretation of Results -

When the pipe line had emptied, a visual check of the observation points was made. For this particular example, approximately all of the original material was still on the pipe invert at the initial points with the exception of points 2 and 5. At point 2, all the original 6.4 - 9.5 mm. material had been removed and at point 5 about 112 grams of the initial 150 grams of 2.0 - 2.4 mm. material had been removed. Observing the interior of the pipe line from point 3 by means of a hand mirror showed that all of the 6.4 - 9.5 mm. particles were on the invert just downstream from their original location at point 2. A check of points 6, 7 and 8 with the mirror failed to locate the particles removed from point 5. This indicated that the material had been carried completely through the pipe line.

From this example, the initial presumption was that particle sizes of about 2.4 mm. would erode. To check this presumption, the next experimental run was conducted with the same d/D and weir head values, but, with particles of 2.0 - 2.4 mm. as the largest size.

Criterion For Determining Deposition

The trial testing period clearly indicated that the pipe joints were the main cause of initial deposition in a clean pipe line. This observation was substantiated in all of the experimental runs. It was observed that those particles which eroded from their initial position on the pipe invert either washed completely through the pipe or were deposited just beyond a pipe joint further down the line. Another indication that the joints caused deposition was the disturbance in the liquid surface just past the joints as shown in Figure 6-2. In this photograph, which was taken looking down into an Observation Tee, a distinct effect of the pipe joint on the sewage flow can be noted.

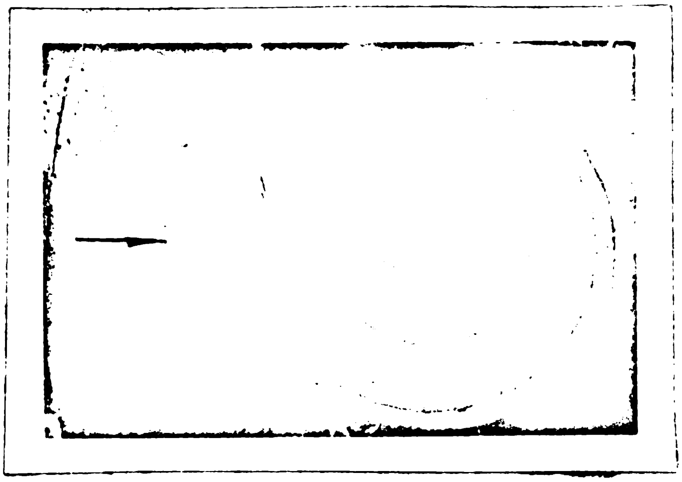


Figure 6-2. Surface Disturbance Created By the Joints. Arrow Shows Direction of the Flow

Two alternate criteria were consdered in defining whether or not a certain size particle would deposit after it was placed in the line. Deposition could be defined as a limiting condition for (1) erosion of the material from its original position or (2) successful passage of the material through the pipe line. The latter definition, successful passage through the line, was selected since the goal of this thesis was to define blockages relative to minimum grades. One primary consideration for selection of criterion (2) was the fact that, in time, enough material could accumulate at or near the joints to create a blockage condition.

The above definition, criterion (2), thus served as the basis for all experiments. However, application of this criterion (2) was not always black and white, especially when dealing with the smaller sized particles or when deposition for a given sized particle was impending. Often two or three size groups of particles eroded and caused the final deposits at the joints to be of mixed sizes. It was then necessary to sieve or separate the particles found at the joints and determine by comparison which sizes had deposited. The determination of sizes present in the mixed deposit at the joint was made by comparing particles of the mixed deposit to sand particles having the same size as those originally placed in the Observation Tees. If a sufficient quantity of any size group originally placed in the line was noted, it was assumed that criterion (2) was not satisfied. If less than 15% of any one size of the original material was present only in a mixed deposit and no where else on the pipe invert, it was considered that self-cleansing satisfied criterion (2).

For impending deposition, the sand particles were observed to creep or slide along the invert. This posed the question of whether, in time, the particles would pass through the entire pipe line. A typical case is shown in Figure 6-3. Impending deposition, and therefore complete erosion, was considered to exist if the wave motion, Figure 6-3, of a given particle size was present for most of the pipe line length.

Another important aspect relative to the observed

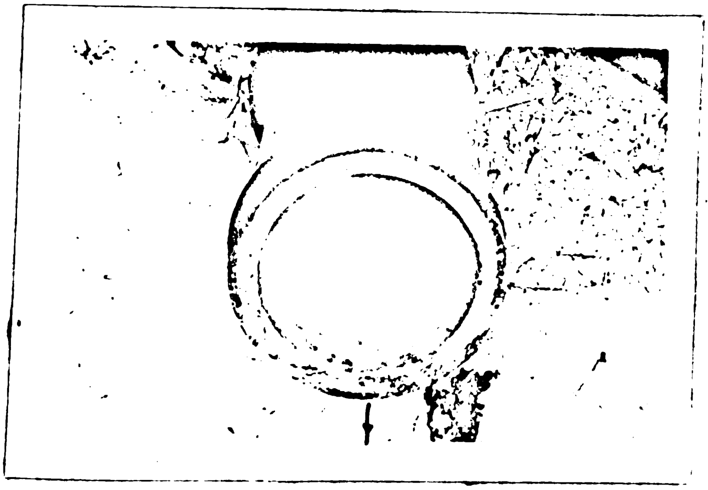


Figure 6-3. Impending Deposition - Arrow Shows Direction of Flow

depositions concerned the different particle sizes which would erode at a given depth of flow and pipe slope. It was noticed that when the largest size particle to erode had been identified, all smaller sized particles woule also erode. And, when the maximum particle size which would erode had been determined, larger sized particles remained on the pipe invert.

Weir Calibrations

When all tests for grades 0.50%, 0.40%, 0.35%, 0.30%, 0.25%, 0.20% and 0.15% were completed, the 60° weir was calibrated in place.

The calibration was conducted in the normal manner by weighing a predetermined amount of sewage for the weir

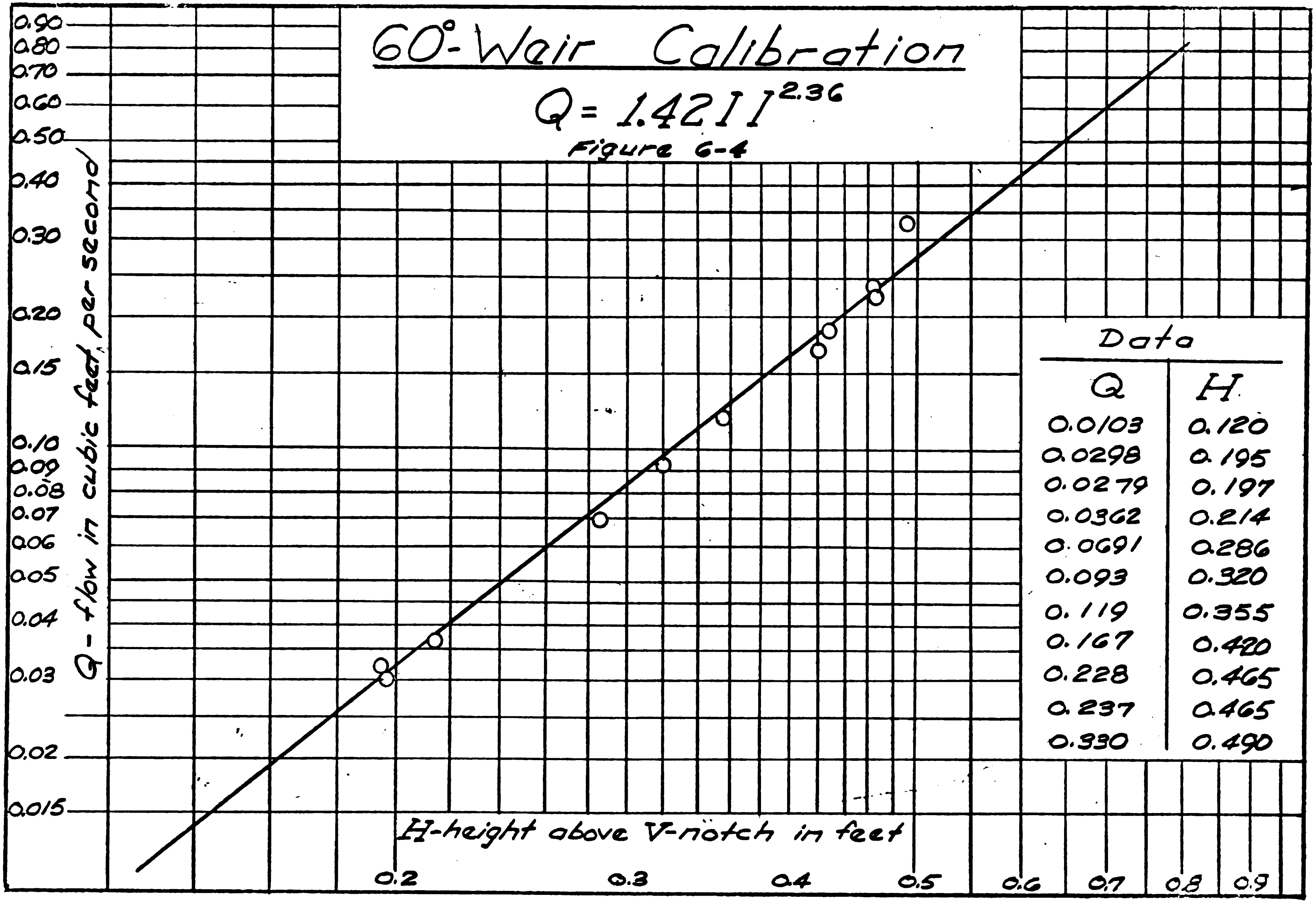


Figure 6-4

Data

Q	H
0.0103	0.120
0.0298	0.195
0.0279	0.197
0.0362	0.214
0.0691	0.286
0.093	0.320
0.119	0.355
0.167	0.420
0.228	0.465
0.237	0.465
0.330	0.490

when discharging at various selected heads. The flow Q was determined from the time required for a given weight of sewage to flow through the weir. The sewage used in the experiments had a unit weight of 62.6 lbs/ft^3.

The weir was completely calibrated on two successive days which provided a check against errors. Both calibrations yielded the same relationship, $Q = 1.42H^{2.36}$*, derived from the calibration data by the method of least squares. The difference between the calculated H power of 2.36 and the usual values of 2.45 to 2.49 probably was due to the facilities for measurement of flow depth over the weir. Depth measurements were made to the nearest hundredth. A distance of 0.33 feet was used between the V and the box bottom rather than the measured 0.325 feet. Therefore, flow data used in the following computations is considered reliable.

Specific Gravities of Particles

The specific gravities for the sand particles were determined by placing the sand size groups in distilled water.

Briefly, the measurement was accomplished by obtaining the dry weight of the sand group and then placing the sand into a beaker of a known volume of water. The specific gravity was calculated by dividing the dry weight

* See Figure 6-4 for calibration curve

of the sand by the weight of the displaced volume of water in the beaker. The specific gravities for the various particle sizes are given in Table 6-3.

Data

The data obtained in the experiments are presented in Tables 6-1 and 6-2. Table 6-1 pertains to the data collected from experimentation on the clay pipe line. The data presented in Table 6-2 represents the physical characteristics of the raw sewage used in the experiments.

Explanation Table 6-1

The data relative to the pipe line experiments are listed in ten columns:

1) % Grade, the per cent grade of the pipe line for the experiment

2) d/D, the ratio of the depth of flow to the inside diameter of the pipe line

3) p, the largest particle size in mm. that passed through the entire length of the pipe line without depositing at the joints

4) Y, the depth in feet of uniform flow of sewage in the pipe line

5) H, the distance in feet between the sewage surface and the V notch in the weir box

6) Q, the flow in cubic feet per second obtained from column 5 and Figure 6-4

7) A, the calculated cross-sectional area* of flow, in square feet, of the sewage in the pipe line

* Calculated by means of Tables 84 & 85, Handbook of Hydraulics by Horace W. King

8) R, the calculated hydraulic radius* in feet, of the sewage flow in the pipe line

9) V, the valocity in feet per second, obtained by dividing column 6 by column 7

10) "n", Manning's constant obtained from columns 1, 8, and 9 and the relationship $n = \frac{1.486R^{2/3}S^{1/2}}{V}$

Explanation Table 6-2

The plant sewage data of Table 6-2 covers the period of August 1, 1960 to September 2, 1960. During this period, the initial trials and the tests for data of Table 6-1 were conducted. All raw sewage data were obtained from the records of the Mason Sewage Treatment Plant.

The data are presented in four columns:

1) The date on which the physical characteristics were determined

2) The per cent grade at which the experiments were conducted on a given day

3) Suspended solids, ppm, in the raw sewage entering the Mason Plant

4) The per cent of the suspended solids in the raw sewage which were volatile

* Calculated by means of Tables 84 & 85, Handbook of Hydraulics by Horace W. King

Table 6-1. Experimental Deposition Data

(1) % Grade	(2) d/D	(3) p (mm.)	(4) Y (ft.)	(5) H (ft.)	(6) Q (cfs)	(7) A (ft2)	(8) R (ft.)	(9) V (ft/sec)	(10) "n"
0.30 cont'd	0.26	1.0	0.17	0.35	0.120	0.072	0.101	1.66	0.0107
	0.18	0.3	0.12	0.27	0.064	0.043	0.073	1.50	0.0094
	0.15	0.15	0.10	0.20	0.031	0.033	0.062	0.95	0.0135
0.35	0.42		0.28	0.58	0.390	0.139	0.148	2.80	0.0088
	0.39	3.3	0.26	0.50	0.275	0.126	0.140	2.18	0.0109
	0.375	--	0.25	0.55	0.350	0.120	0.136	2.92	0.0080
	0.33	--	0.22	0.41	0.170	0.101	0.123	1.69	0.0129
	0.32	2.0	0.21	0.44	0.205	0.096	0.120	2.12	0.0100
	0.30	--	0.20	0.44	0.205	0.088	0.114	2.31	0.0089
	0.27	1.4	0.18	0.38	0.145	0.076	0.105	1.91	0.0103
	0.24	--	0.16	0.36	0.125	0.064	0.095	1.95	0.0094
	0.195	0.4	0.13	0.29	0.076	0.048	0.079	1.58	0.0102
	0.18	--	0.12	0.29	0.076	0.043	0.073	1.77	0.0087
	0.12	--	0.08	0.21	0.035	0.024	0.050	1.47	0.0081
0.40	0.36	--	0.24	0.54	0.330	0.113	0.132	2.92	0.0084
	0.34	7.9	0.23	0.53	0.310	0.105	0.126	2.96	0.0080
	0.27	3.3	0.17	0.42	0.180	0.076	0.105	2.37	0.0089
	0.17	1.1	0.11	0.28	0.070	0.039	0.069	1.78	0.0089
	0.105	0.2	0.07	0.18	0.024	0.020	0.044	1.23	0.0096
[illegible]	0.39	--	0.26	0.61	0.45	0.126	0.140	3.57	0.0080
	0.29	6.4	0.19	0.50	0.275	0.084	0.111	3.27	0.0074
	0.21	--	0.14	0.38	0.145	0.084	2.72	2.72	0.0074
	0.20	3.3	0.13	0.37	0.135	0.050	0.081	2.72	0.0072
	0.09	0.2	0.06	0.17	0.021	0.016	0.038	1.35	0.0083

Table 6-1. Experimental Deposition Data

(1) % Grade	(2) d/D	(3) p (mm.)	(4) Y (ft.)	(5) H (ft.)	(6) Q (cfs)	(7) A (ft2)	(8) R (ft.)	(9) V (ft/sec)	(10) "n"
0.15	0.57	2.4	0.38	0.61	0.46	0.205	0.180	2.24	0.0082
	0.50	1.2	0.33	0.52	0.30	0.175	0.167	1.71	0.0102
	0.41	0.6	0.27	0.42	0.18	0.135	0.145	1.33	0.0119
	0.33	0.3	0.22	0.38	0.145	0.101	0.123	1.44	0.0099
0.20	0.47	--*	0.31	0.50	0.275	0.161	0.160	1.71	0.0114
	0.45	1.7	0.30	0.52	0.30	0.153	0.155	1.96	0.0098
	0.40	1.1	0.27	0.47	0.24	0.130	0.143	1.83	0.0099
	0.34	0.6	0.22	0.42	0.18	0.105	0.126	1.71	0.0098
	0.27	0.2	0.18	0.32	0.094	0.076	0.105	1.24	0.0120
0.25	0.47	4.0	0.31	0.55	0.35	0.161	0.160	2.17	0.0101
	0.39	2.4	0.26	0.46	0.225	0.126	0.140	1.78	0.0113
	0.34	1.2	0.23	0.42	0.18	0.105	0.126	1.71	0.0109
	0.29	1.0	0.19	0.35	0.12	0.084	0.111	1.43	0.0120
	0.285	--	0.19	0.34	0.11	0.082	0.109	1.34	0.0126
	0.25	0.4	0.17	0.31	0.09	0.068	0.098	1.32	0.0119
	0.195	0.3	0.13	0.29	0.074	0.048	0.079	1.54	0.0089
0.30	0.50	6.4	0.33	0.55	0.350	0.175	0.167	2.00	0.0123
	0.42	--	0.28	0.50	0.275	0.139	0.148	1.98	0.0115
	0.41	4.0	0.27	0.49	0.265	0.135	0.145	1.96	0.0115
	0.39	--	0.26	0.48	0.250	0.126	0.140	1.97	0.0112
	0.30	1.7	0.20	0.42	0.180	0.088	0.114	2.04	0.0094

* no definite particle deposition

Table 6-2. Plant Sewage Data

(1) Date	(2) %S	(3) Suspended Solids (ppm)	(4) Volatile Solids (% of suspended)
Aug. 1		340	54
2		140	
3		250	49
4		193	
5		160	59
8		201	27
9		150	
10		200	44
11		180	
12		200	45
15		170	52
16		280	
17	0.5 & 0.4	200	44
18	0.4	180	
19	0.4	179	55
22	0.3	260	59
23	0.3 & 0.25	150	
24	0.25	121	44
25	0.2	183	
26	0.2	133	42
29	0.15	200	47
30	0.15	190	
31	0.35	159	47
Sept. 1	0.35	129	
2		250	43

Temperature range of sewage: 68°F - 72°F

Table 6-3. Specific Gravities of Sand Particles

Size Range (mm)	Specific Gravity
6.4 - 9.5	2.61
4.7 - 6.4	2.67
3.3 - 4.7	2.63
2.4 - 3.3	2.60
2.0 - 2.4	2.72
1.7 - 2.0	2.69
1.2 - 1.7	2.73
1.0 - 1.2	2.71
0.6 - 1.0	2.76
0.4 - 0.6	2.76
0.3 - 0.4	2.76
0.2 - 0.3	2.75

VII DISCUSSION

An analysis of the data of Tables 6-1 and 6-2 indicated that:

1) The flow depth at a given pipe slope had a marked influence on the resulting deposition.

2) The solids suspended in the sewage flow had little or no influence on deposition.

The conclusion that suspended sewage solids did not influence deposition was reached by comparing columns (2) and (3) of Table 6-2 with Figure 7-1. These Table 6-2 values show a variance of suspended solids for tests conducted at given grades on different days. When d/D was plotted against p for the various slopes, as shown by Figure 7-1, the suspended solids' variance of Table 6-2 had little effect on the pattern of the plotted points for each pipe line slope. Moreover, the extreme range of the suspended solids did not have any apparent relationship to the Figure 7-1 plots of d/D against p for the various slopes.

It may be concluded, therefore, that the data presented in Table 6-1 contains only those variables which had the primary effect on the observed deposition.

Relationships Between Slope, Depth of Flow and Particle Size

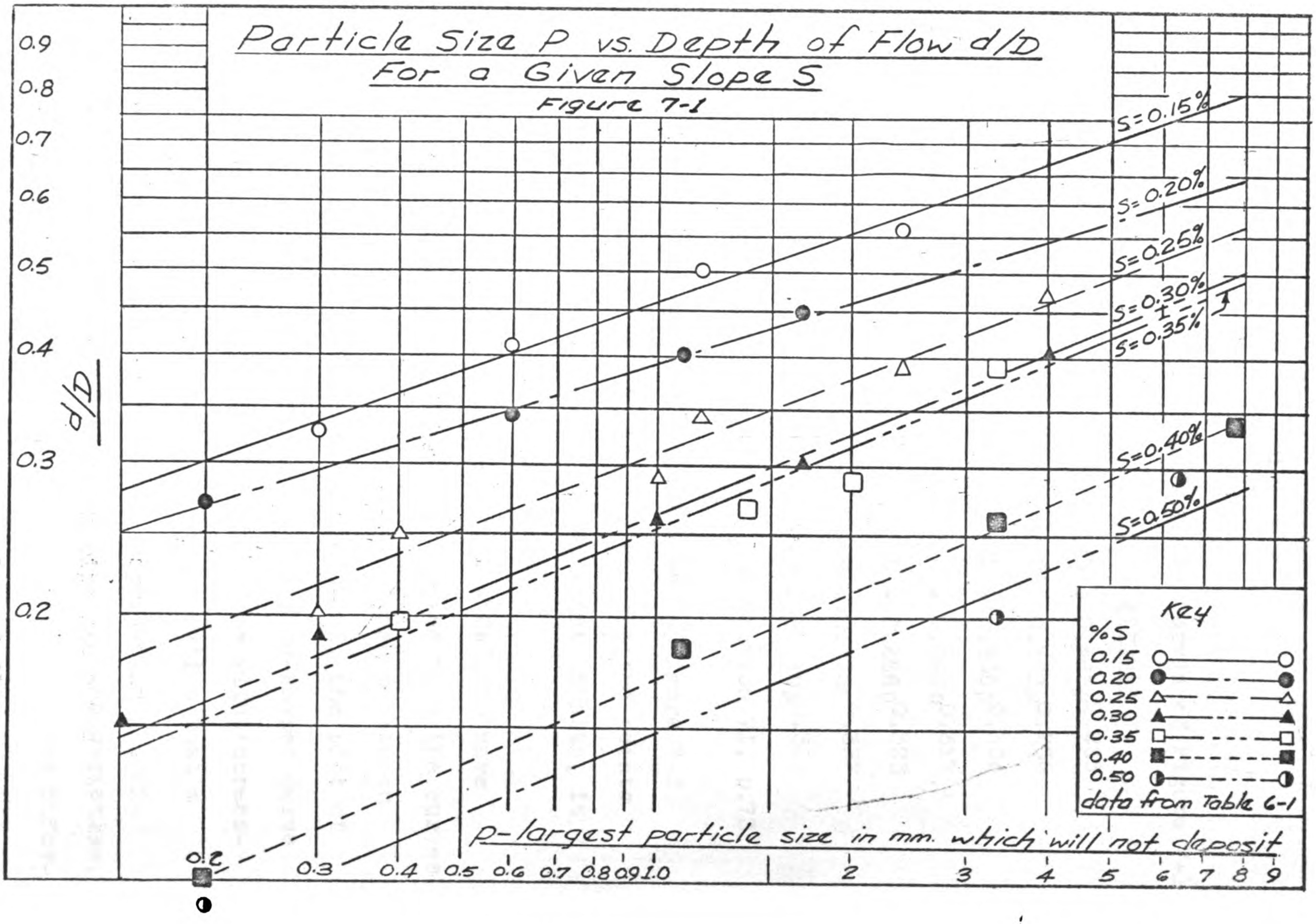
The problem to which this thesis has committed itself is one of establishing patterns or relationships which cause deposition. Briefly, the question is this: "Can the size of sand particles which are transported in a sanitary sewer be predicted for a given pipe slope and at various depths of flow?"

A preliminary coordinate paper plot of flow depth d/D against the maximum particle size p of sand which met the required "self-cleansing" criterion (2) of Section VI indicated that, for a given slope, a family of parabolic-type curves described the observed deposition. The above variables were then plotted on log-log paper as shown in Figure 7-1. The plot of Figure 7-1 shows an interesting linear pattern. In general, the curves of this figure appear to have approximately the same slope. With the exception of the $S = 0.35\%$ curve, there also seems to be a tendency for the curves to be regularly spaced.

The d/D versus p relationships of each slope of Figure 7-1 were then analyzed by the method of least squares, and an expression for each curve was determined. The empirical expressions establish the largest size particle in mm. which will successfully pass through the entire pipe line at a particular grade and any depth of flow. These relationships are presented by Table 7-1 in the form $d/D = Cp^x$.

- Variation of C and x With Slope -

An examination of Table 7-1 showed a decrease of the coefficient C with increasing slope, suggesting that some relationship may exist between C and the pipe slope. This relationship was more apparent when C was plotted against



Particle Size P vs. Depth of Flow d/D For a Given Slope S

Figure 7-1

Table 7-1.* Formulas Resulting From the Curves of Figure 7-1

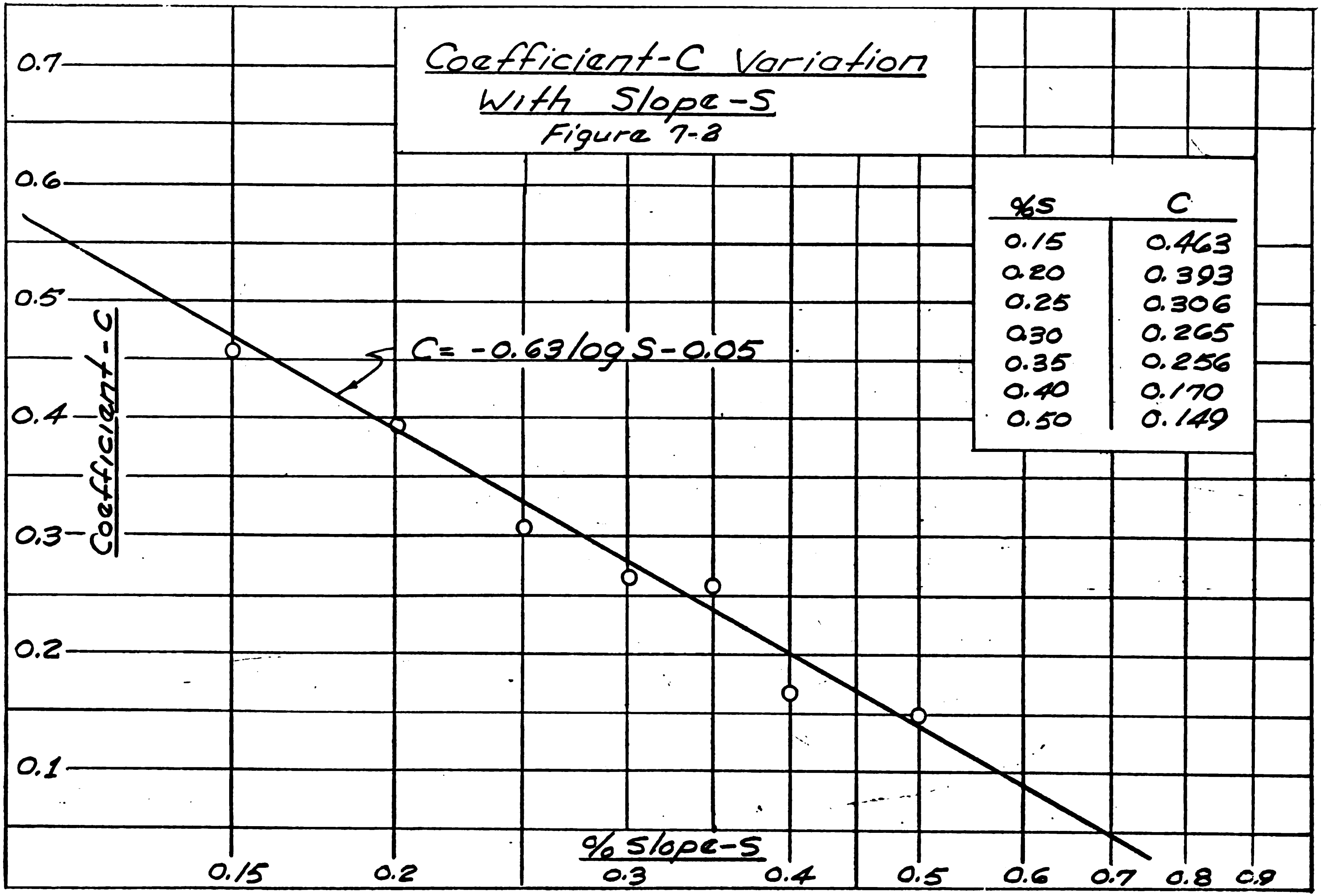
% Pipe Line Grade S	Formula
0.15%	$d/D = 0.463p^{0.265}$
0.20%	$d/D = 0.393p^{0.236}$
0.25%	$d/D = 0.306p^{0.304}$
0.30%	$d/D = 0.265p^{0.317}$
0.35%	$d/D = 0.256p^{0.323}$
0.40%	$d/D = 0.170p^{0.337}$
0.50%	$d/D = 0.149p^{0.322}$

* For an explanation of the symbols see Section VI, p.73

slopes for each formula of Table 7-1 as in Figure 7-2. A straight line was interpreted for the plotted points of Figure 7-2; applying the method of least squares, it was found $C = (-0.63 \log S - 0.05)$.

For the general relationship, $d/D = Cp^x$, to have meaning, the exponent, x, must be evaluated from the curves of Figure 7-1. To test whether a relationship between slope S and exponent x could be established, the plot of Figure 7-3 was prepared. The resulting sine-shaped curve of Figure 7-3 indicated that x did increase with increasing slope. However, there was no demonstration that a clear-cut relationship exists.

Since the data of Table 6-1 pertains to the variables, slope, depth of flow and particle size, insufficient information is available to make any definite statement concerning causes of the variation of the value x.


Coefficient-C Variation
With Slope-S
Figure 7-8
%S | C
0.15 | 0.463
0.20 | 0.393
0.25 | 0.306
0.30 | 0.265
0.35 | 0.256
0.40 | 0.170
0.50 | 0.149
C= -0.63/log S-0.05
Coefficient-C
0.7
0.6
0.5
0.4
0.3
0.2
0.1
% Slope-S
0.15
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9

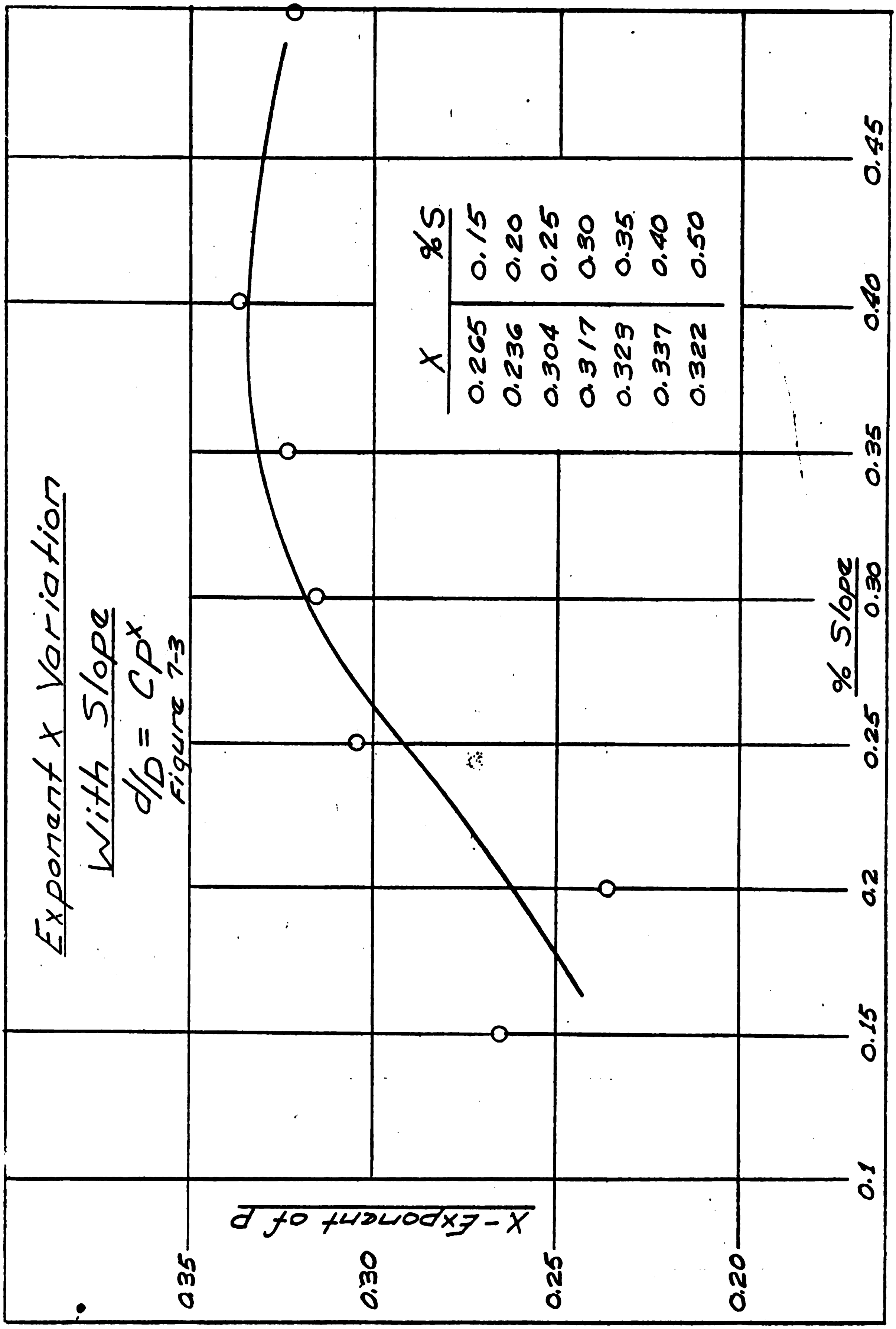

Exponent X Variation
With Slope
d/D = Cp^x
X | %S
0.265 | 0.15
0.236 | 0.20
0.304 | 0.25
0.317 | 0.30
0.323 | 0.35
0.337 | 0.40
0.322 | 0.50
X - Exponent of p
0.35
0.30
0.25
0.20
0.1
0.15
0.2
0.25
0.30
0.35
0.40
0.45
% Slope

Figure 7-3

The range in x is not extreme (0.236 to 0.337) and the median of the exponent values of Table 7-1 is approximately 0.32. Hence an x of 1/3 was selected for this initial limited investigation. The general expression for the depth of flow required to prevent a given sized sand particle from depositing in the pipe line at varying slopes could then be defined as:

$$d/D = (-0.63 \log S - 0.05)p^{1/3} \qquad (1)$$

- Solution by Nomograph -

The above final expression for deposition is conveniently expressed graphically for rapid solution. Such a nomograph is shown in Figure 7-4. A solution to the above expression for deposition in an 8-inch clay pipe can be obtained from the nomograph if two of three variables, depth of flow (d/D), slope S or particle size p are known. The unknown quantity is found in each case at the intersection of its scale with a straight line connecting the other two known variables.

- General Discussion of Data -

The previous discussion on deposition was devoted entirely to examining the data relative to 8-inch diameter sewer pipes. However, it is desirable to study the data in more general terms so that the results of the investigation can be applied to sewer pipes of other diameters.

One method of projecting this research to different sewer pipe diameters is to examine the data in terms of a function that is common to all sewer pipes. This examination can be accomplished by calculating the non-joint boundary shear, τ_c , as concerned with sand particle deposition. The non-joint boundary shear can be calculated from equation (2), Section IV.

Figure 7-5 represents Table 7-2 plotted on log-log paper where it can be seen that τ_c does not have a constant value, but gradually increases with the maximum particle size which will not deposit. This non-constant value of τ_c may be attributed to:

1) The calculation of τ_c at a non-joint section since deposition occurred at the joints which indicates that τ_c is some function of the true critical boundary shear,

2) The effects of turbulence, and the magnitudes and the directions of the eddy velocities at the joints.

While the above method places the data in a more general nature, it does not explicitly determine what causes τ_c to vary. Nevertheless, it does make it possible to extend the data to other sized sewer pipes pending further examination.

Pipe Joints and Deposition

It was mentioned in Section VI that deposition occurred primarily at pipe joints. The tendency to deposit at the joints can partly be explained from observations made on the surface disturbances at the various observation points as shown by Figure 6-2 and Figure 7-6. The surface of the

SLOPE AND DEPTH OF FLOW (SEWAGE) REQUIRED TO PREVENT DEPOSITION IN A NEW 8" CLAY SEWER PIPE

Figure 7-4

$$d/D = p^{1/3}[-0.63 \log S - 0.05]$$

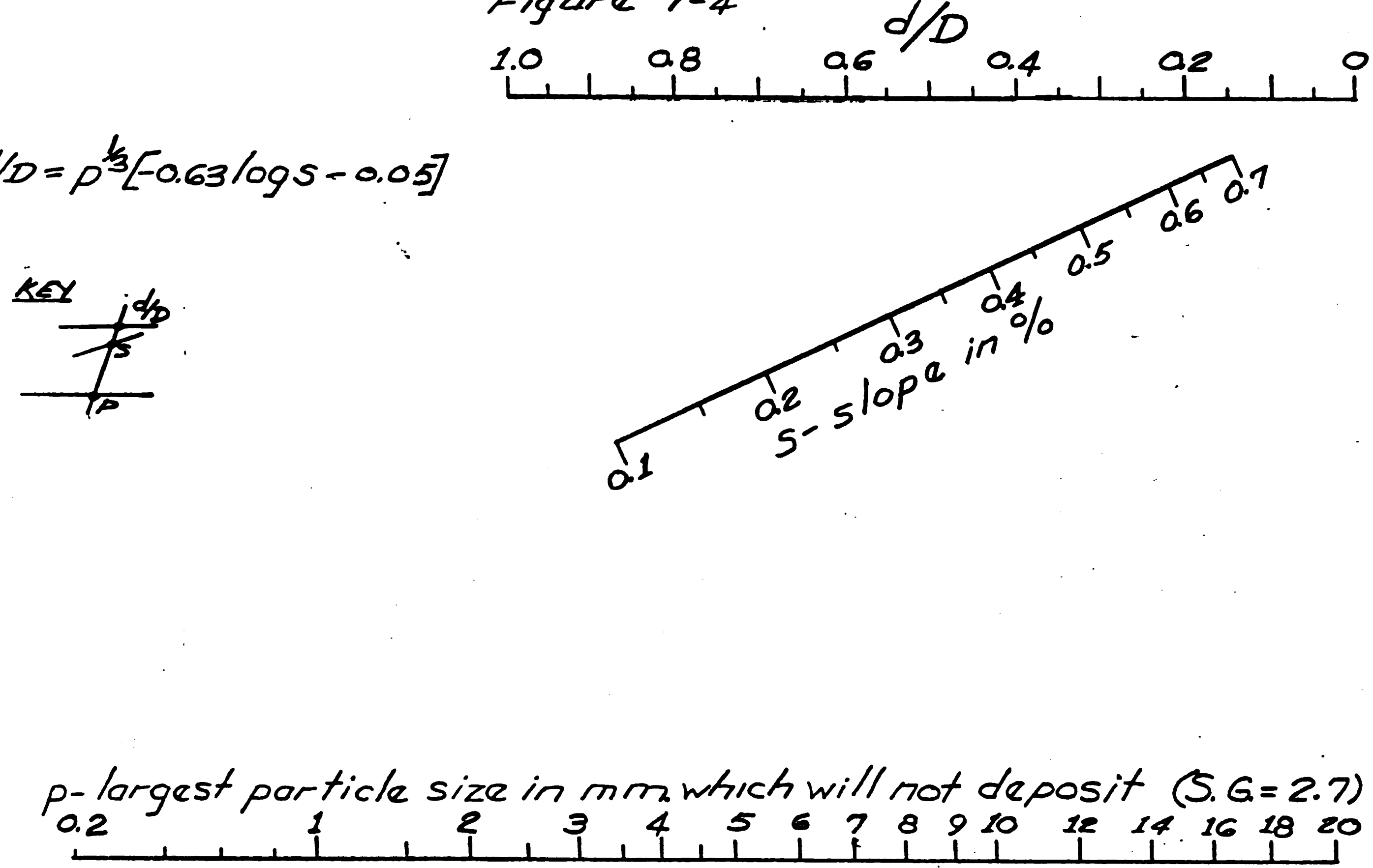
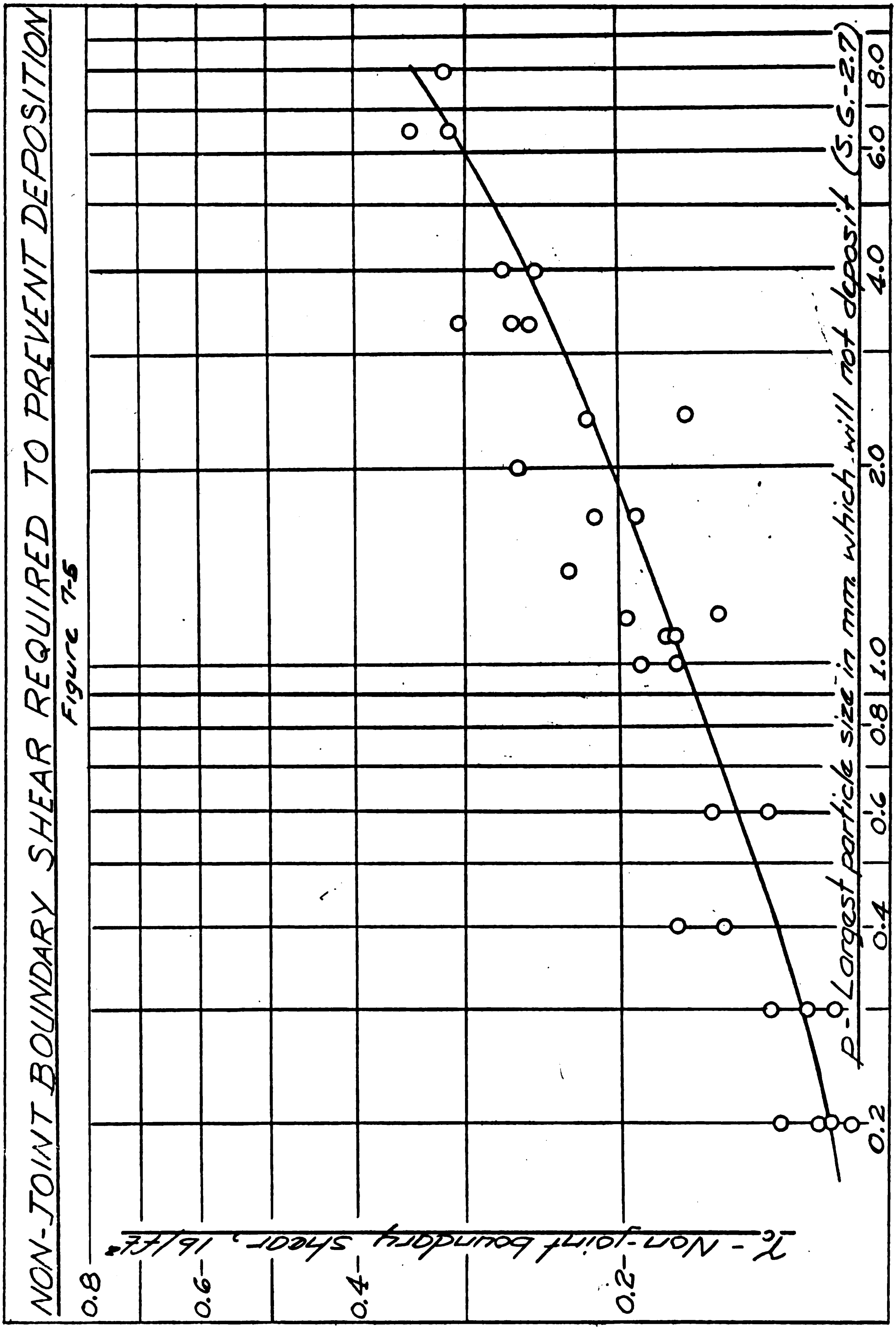


Table 7-2. Computed Non-Joint Boundary Shear Required to Prevent Deposition*

τ_c (lb/ft^2)	p (mm)
0.0110	0.2
0.0115	0.3
0.0116	0.2
0.0119	0.2
0.0124	0.3
0.0132	0.2
0.0136	0.6
0.0137	0.3
0.0153	0.4
0.0157	1.2
0.0158	0.6
0.0169	2.4
0.0173	0.4
0.0173	1.1
0.0174	1.0
0.0179	1.1
0.0190	1.0
0.0194	1.7
0.0197	1.2
0.0214	1.7
0.0219	2.4
0.0230	1.4
0.0251	4.0
0.0254	3.3
0.0263	2.0
0.0264	3.3
0.0272	4.0
0.0307	3.3
0.0314	6.4
0.0316	7.9
0.0348	6.4

* computed from equation 2, Section IV and Table 6-1



NON-JOINT BOUNDARY SHEAR REQUIRED TO PREVENT DEPOSITION

Figure 7-5

sewage flow was markedly influenced by joints, with a resulting creation of V-shaped ripple patterns which pointed downstream. The following discussion on the cause of deposition at these ripple patterns will refer to Figure 7-6.

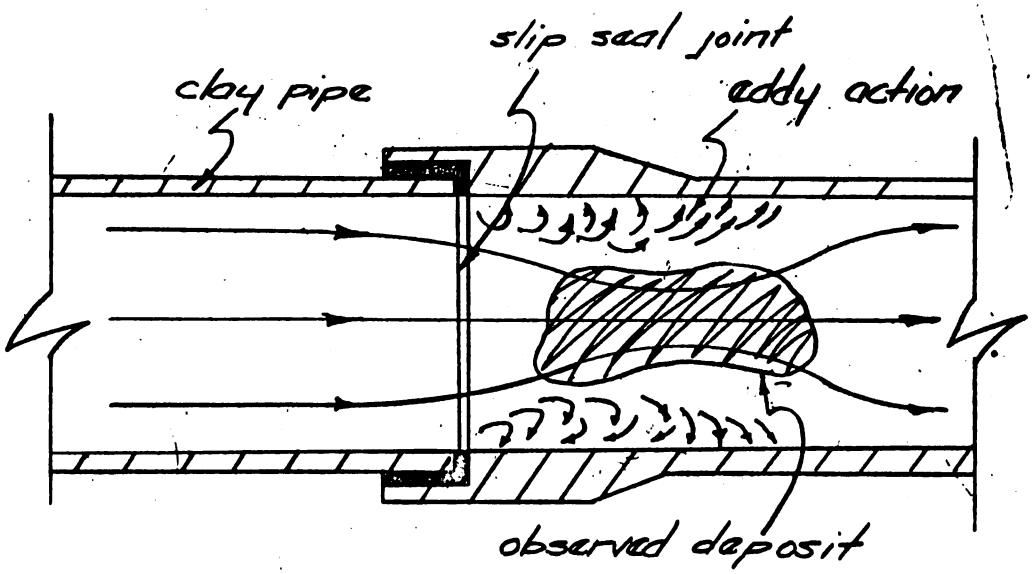


Figure 7-6. Deposition at a Joint

An explanation for the observed depositions at the pipe joints can be offered by studying the effect of turbulence and eddy action on the joint boundary shear, τ_c'. The direction and magnitude of τ_c' is dependent upon the direction and magnitude of the flow velocity at the joints. Because of the eddy action shown by Figure 7-6, the critical τ_c' can have a direction opposite that of the non-joint τ_c, which has the same direction as the flow or may decrease without changing direction from that of the

general flow.

The boundary shear in a non-joint section is adequate to prevent deposition if the shear is greater than the limiting value imposed by the flow and pipe section. At the joints, however, the boundary shear may be reduced or reversed, resulting in the observed deposition, because the boundary shear is less than the limiting value. If deposition does begin, additional sand particles of the same critical size reaching the joint will be deposited. The deposit at the joint can increase in this manner and cause further deposition of smaller sized particles due to the increased turbulence, pipe invert friction and eddy action.

Factors Affecting Deposition

Experimental data indicates that the velocity and the flow depth required to prevent deposition at the joints are related. That is, as the velocity increases, the critical no-deposit depth of flow decreases. This relationship is shown in Table 7-3 where it can be noted that V and d/D vary in this way when a given sized particle is eroded at a given slope. Table 7-3 shows quite conclusively that velocity is only one of the important variables in defining tendencies toward deposition at the joints; and, that depth of flow and pipe slope are equally important.

Apart from the influence of velocity and depth of flow on deposition of given sized sand particles, at given

Table 7-3. Velocity and Depth of Flow at Which Deposition Occurred

P (mm)	V (ft/sec)	d/D	%S
7.9	2.96	0.34	0.40
6.4	2.00	0.50	0.30
	3.27	0.29	0.50
4.0	2.17	0.47	0.25
	1.96	0.41	0.30
3.3	2.18	0.39	0.35
	2.37	0.27	0.40
	2.72	0.20	0.50
2.4	2.24	0.57	0.15
	1.78	0.39	0.25
2.0	2.12	0.32	0.35
1.7	1.96	0.45	0.20
	2.04	0.30	0.30
1.4	1.91	0.27	0.35
1.2	1.71	0.50	0.15
	1.71	0.34	0.25
1.1	1.83	0.40	0.20
	1.78	0.17	0.40
1.0	1.43	0.29	0.25
	1.66	0.26	0.30
0.6	1.33	0.41	0.15
	1.71	0.34	0.20
0.4	1.32	0.25	0.25
	1.58	0.195	0.35
0.3	1.44	0.33	0.15
	1.54	0.195	0.25
	1.50	0.18	0.30
0.2	1.24	0.27	0.20
	1.23	0.105	0.40
	1.35	0.09	0.50
0.15	0.95	0.15	0.30

pipe slopes, are the following factors which could not be considered in this initial investigation:

1) Sliming of pipe interior
2) Deposition of organic materials
3) Non-uniform flow
4) Poor pipe alignment
5) Type of sewer pipe (i.e., concrete, asbestos, etc.)
6) Pipe diameters
7) Pipe age
8) Types of sewer joints

Analysis of Shear Relationships

Analytical attempts have been made by Camp[10] and others to determine a satisfactory relationship for defining those conditions which lead to particle deposition. It will be recalled that Camp's derived expression, equation (1), Section III, contained a function β which is assumed constant; its value being determined only by the manner in which the particle was transported (along the invert or by saltation). The data collected in this experiment indicates that β does not have this constant value.

β values have been determined by substituting experimental data into an altered form of Camp's basic relationship, equation (1), Section III. This altered equation has been obtained by solving Camp's expression $s = \frac{\beta(\gamma_s - \gamma_w)d}{R\gamma_w}$ for β; this produced:

$$\beta = sR\gamma_w / d(\gamma_s - \gamma_w) \quad (2)$$

where s equals the slope, R equals the hydraulic radius, d equals the particle size, γ_s equals the specific gravity of sand and γ_w equals the specific gravity of water.

Table 7-4 represents equation (2) solved for the following conditions:

1) $\gamma_s = 2.72$

2) $\gamma_w = 1$

3) d = 6 mm. - 0.0197 ft.

4) 8" new vitrified clay pipe

5) s determined by Figure 7-4

Table 7-4. Computed β Required to Prevent Deposition of 6 mm. Sand Particle

d/D	R(ft)	%s	β
0.1	0.042	0.71	0.0088
0.2	0.80	0.56	0.0132
0.3	0.113	0.46	0.0154
0.4	0.143	0.37	0.0156
0.5	0.167	0.31	0.0157
0.6	0.185	0.25	0.0137
0.7	0.198	0.21	0.0123
0.8	0.203	0.17	0.0102
0.9	0.198	0.13	0.0079
1.0	0.167	0.11	0.0054

Table 7-4 shows that an arbitrary selection of β cannot be made relative to the mode of particle transportation. The manner of particle transportation (i.e., rolling, sliding, suspended, etc.) is determined by the compbined effects of particle size and specific gravity, pipe slope, depth of flow, flow velocity, and over all pipe roughness.

The variance of β as computed from the experimental data of this investigation is probably due to the effect of the pipe joints. Apparently, Camp's equations do not provide for any roughness other than that of the pipe walls.

Camp's β is determined from the results of Shield's work, which pertained to stream beds that obviously are different than ordinary sewer pipes. Therefore, the only conclusion that can be drawn is: Camp's equations relative to self-cleansing velocities and pipe slopes are not strictly applicable to jointed sewer pipes.

The above discussion also reflects upon the development of Section IV, where Shield's ϕ was employed. The general equations of Section IV, which are similar to Camp's relationships above, would be more applicable to engineering practice if the ϕ values were determined more realistically; that is, by investigating actual sewer pipe line conditions.

Figure 7-7 represents ϕ values calculated from Shields' entrainment function for flow conditions which initiated bed movement:

$$\frac{\tau_{critical}}{\gamma_w (S_s - 1) d} = \phi \left(\frac{d \sqrt{\tau / \rho}}{\nu} \right) \qquad (3)$$

where γ_w equals 62.4 lb/ft^3, S_s equals 2.65, d equals the particle size in ft., τ critical equals the critical boundary shear in lb/ft^2, ρ is 1.943 lb-sec^2/ft^4, ν is 1.029×10^{-5} ft^2/sec., and ϕ defines a function.

There are several apparent causes for the deviation of Shields' entrainment function for this investigation

from values prepared by Shields: 1) Shields' experiments pertained to stream beds where the data was obtained for a horizontal bed and not a semi-circular invert. This means for the same quantity of flow Shields' flow cross-section generally produced a larger hydraulic radius, a difference which strongly can influence the boundary shear. 2) Shields experiments were not related to the joint disturbances of sewer pipes.

The above comparison reinforces the previously mentioned conclusion on the applicability to jointed sewer pipe of Shields' findings, as used by Camp.

Analysis of Values of Manning's "n"

While the intent of and the equipment for this experimental work were to study deposition, an interesting aspect was the calculation of Manning's "n" from the experimental data. The calculation of "n" showed that for this experiment, "n" decreased with increasing slope. This variance can be observed from Table 7-5 and Figure 7-8 which were determined from the data of Table 6-1.

It is interesting to note that the recent work of Bloodgood and Bell[15] showed the same variance of "n" with slope as recorded by this investigation; "n" increased with decreasing slope.

The "n" variance as calculated from this study was, however, opposite to that determined from the data of

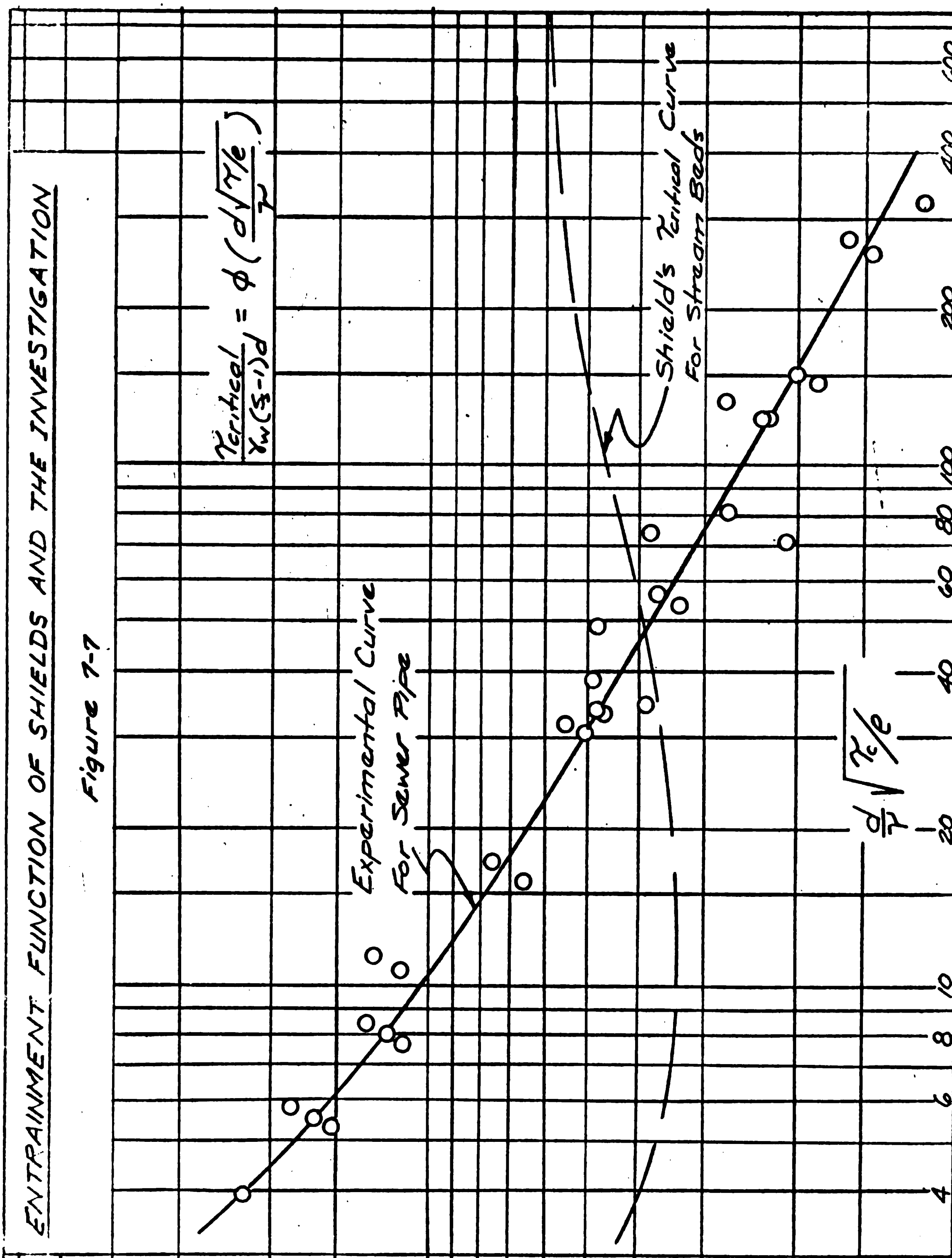

ENTRAINMENT FUNCTION OF SHIELDS AND THE INVESTIGATION
Figure 7-7
$\frac{\tau_{critical}}{\gamma_w(S_s-1)d} = \phi\left(\frac{d\sqrt{\tau/\rho}}{\nu}\right)$
Experimental Curve For Sewer Pipe
Shield's $\tau_{critical}$ Curve For Stream Beds
$\frac{d}{\nu}\sqrt{\tau_c/\rho}$
0.6
0.4
0.2
0.1
0.08
0.06
0.04
0.02
2
4
6
8
10
20
40
60
80
100
200
400
600

Table 7-5. Variation of "n" with Slope

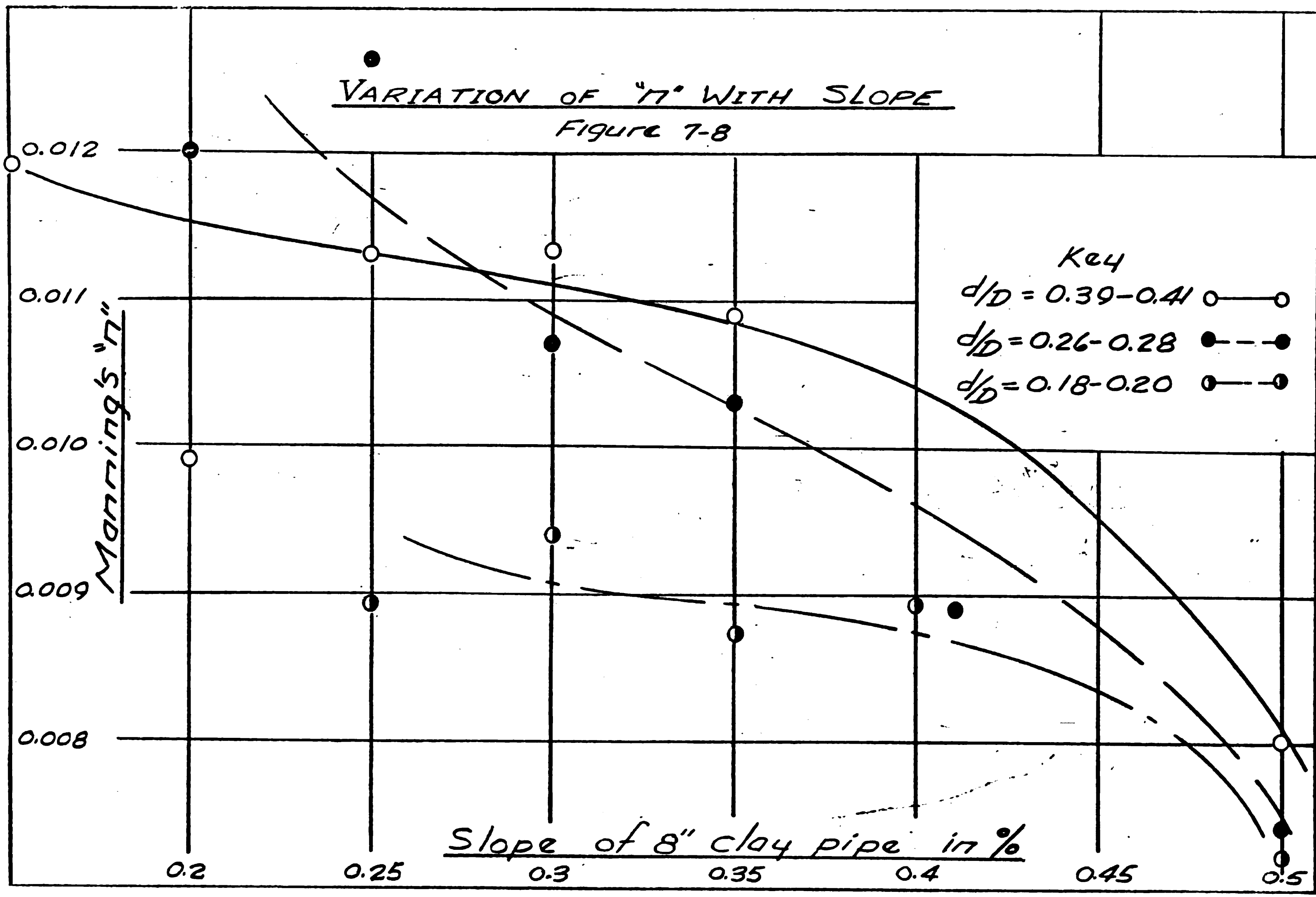
d/D	%s	"n"
0.18-0.20	0.25	0.0089
	0.30	0.0094
	0.35	0.0087
	0.40	0.0089
	0.50	0.0072
0.26-0.28	0.20	0.0120
	0.25	0.0126
	0.30	0.0107
	0.35	0.0103
	0.40	0.0089
	0.50	0.0074
0.39-0.41	0.15	0.0119
	0.20	0.0099
	0.25	0.0113
	0.30	0.0114
	0.35	0.0109
	0.50	0.0080

Yarnell and Woodward[3] which showed that "n" increased with increasing slope. Table 7-6 represents Yarnell and Woodward's data pertaining to an 8-inch clay pipe.

Table 7-6. Experimental Data by Yarnell and Woodward for an 8-inch Clay Pipe

d/D	%s	r(ft)	"n"	V(ft/sec)
0.38	0.05	0.1406	0.0120	0.658
0.40	0.10	0.1456	0.0114	1.017
0.39	0.20	0.1439	0.0138	1.159
0.41	0.30	0.1490	0.0139	1.449
0.40	0.50	0.1459	0.0143	1.730
0.24	0.05	0.0978	0.0114	0.527
0.28	0.10	0.1106	0.0114	0.851
0.24	0.20	0.0973	0.0144	0.790
0.26	0.50	0.1033	0.0149	1.250

Mean velocity increased with slope and depth of flow until it reached its maximum value at 0.77 d/D



VARIATION OF "n" WITH SLOPE

Figure 7-8

For this discussion, it is well to keep in mind three factors discussed in a recent paper by Schmidt[20] which may be responsible for the observed experimental "n" variation with slope:

1) The inability of the hydraulic radius term in the Manning Equation to adequately represent changes in the shape of the flow cross-section

2) The influence of sewer pipe joints

3) The influence of deposits in the sewers

All determinations of "n" in these experiments were made with deposits present on the pipe invert. Therefore, it seems that factors such as those proposed by Schmidt plus the inherent difficulty in obtaining a high level of hydraulic control with equipment used for these experiments caused the observed irregularities in "n" values.

Also, it should be noted that the roughness coefficient "n", which is a function of the Darcy-Weisbach friction factor, decreases with increasing Reynolds Number. For a constant d/D, an increase in slope increases the velocity and, hence, increases Reynolds Number. Therefore, "n" decreases with increasing slope in the manner indicated by the experimental results.

The above discussion concerning variations in "n" leads to the question of whether the experimental data make sense in terms of classical hydraulics. An answer to this question can be partially found by developing a Moody Diagram for the experimental data and comparing it to a standard Moody Diagram.

The Moody Diagram of Figure 7-9 was constructed from the values of Table 7-7. Table 7-7 was obtained from Table 6-1, using the relationships:

$$f = 8gRS/V^2 \quad \text{(friction factor)}$$

$$N_R = 4RV/\nu \quad \text{(Reynolds number)}$$

The value of ν was selected for water at 70°F which produced $\nu = 1.029 \times 10^{-5}$ ft²/sec. (see Table 6-2).

The calculation of Reynolds Number, N_R, and the Darcy-Weisback friction factor, f, for the construction of the Moody Diagram showed the influence of the experimental errors. The ± 0.01% error in slope, the ± 0.01 ft. error in d/D and the unknown error in determining the flow quantity required grouping the data to obtain average values for N_R and f.

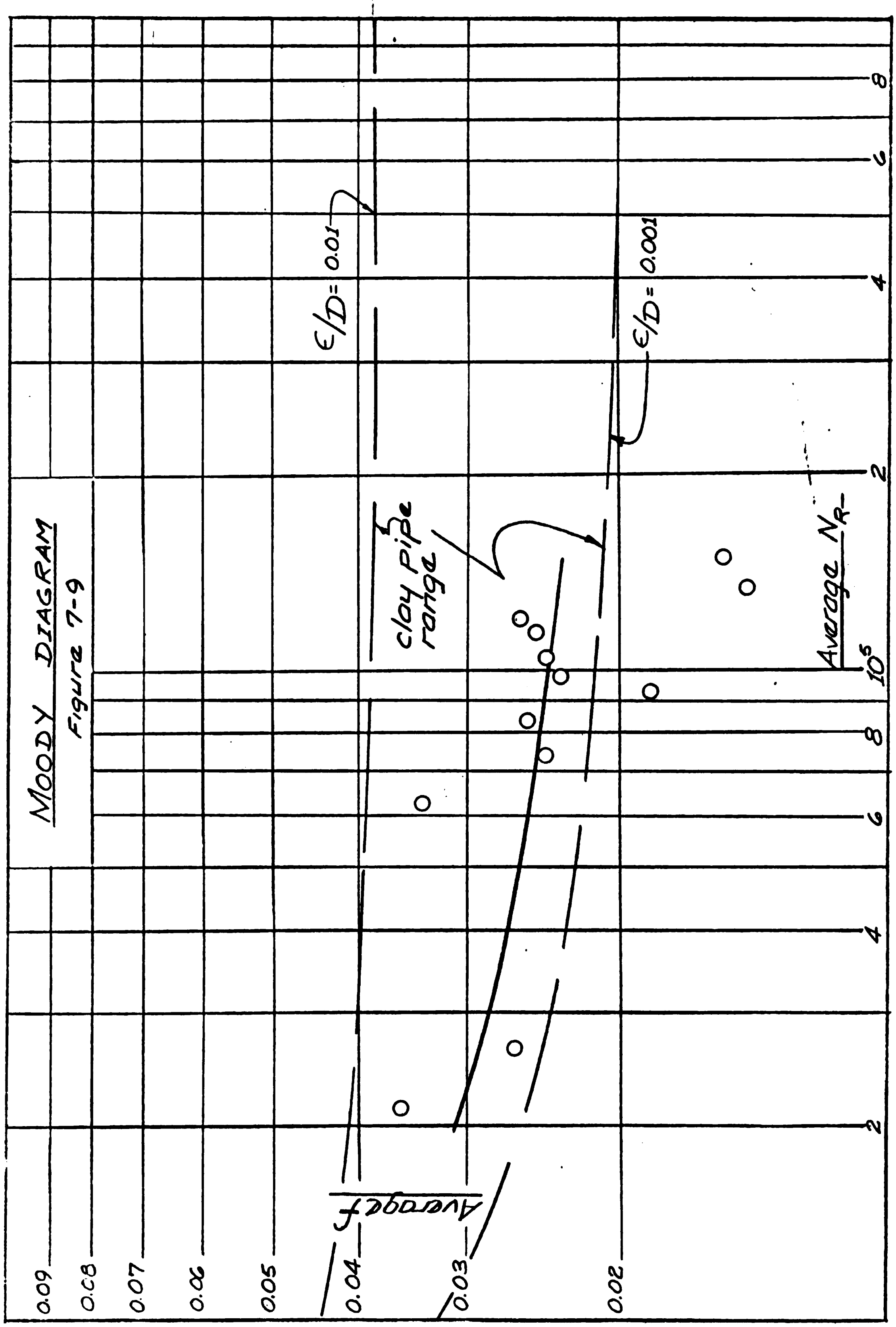
For a given ϵ/D, there can exist only one f value for a given N_R. Therefore, N_R was calculated for each experimental test run, and the data were grouped by similar N_R values. Since ϵ/D did not vary, f was also determined for each test run and an average f value was obtained for each N_R group. Figure 7-9 is a plot of the above determined N_R and f values of Table 7-7.

Figure 7-9 indicates that the majority of the calculated values fall in the general range for vitrified clay pipe on the Moody Diagram. This fact illustrates that overall the experimental data were reasonably accurate in a hydraulic sense. However, insufficient data were available to make any firm statement on the exact manner of "n" and f variance other than they probably change in the manner previously indicated.

Table 7-7. Moody Diagram Values

N_R, average	f, average
2.1×10^4	0.0366
4.9×10^4	0.0269
6.2×10^4	0.0341
7.4×10^4	0.0246
8.4×10^4	0.0259
9.1×10^4	0.0183
9.9×10^4	0.0234
1.1×10^5	0.0247
1.2×10^5	0.0252
1.3×10^5	0.0270
1.5×10^5	0.0147
1.6×10^5	0.0151

The variance of "n" also poses the following question: Should Manning's equation be used in the design of sanitary sewers? This question arises because of the importance of "n" to the Manning relationship and because sewers generally flow less than full.


MOODY DIAGRAM
Figure 7-9
ε/D = 0.01
ε/D = 0.001
clay pipe range
Average N_R
Average f
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
2
4
6
8
10^5
2
4
6
8

Sufficient data have been collected to determine the relative range of "n" values and, all studies show an "n" value of the same general magnitude. However, no two investigations can agree upon exact "n" values for sewers. The various investigations concerned with defining "n" have been conducted under <u>ideal</u> conditions; that is, with known flows, exact slope, proper alignment, etc.. The same certainly cannot be anticipated for sanitary sewers. Thus, the selection of "n" for sanitary sewer design can be, at best, only an educated guess.

It is argued that the difficulties inherent in choosing a reasonable value of "n" for design can be overcome by using a value greater than observed test values, i.e. 0.013 for clay pipe. This supposedly provides a factor of safety between 20 t0 30 per cent depending upon the actual "n". But, if investigators cannot concur on all the factors which cause "n" to vary, how can an arbitrary selection of "n" produce a satisfactory design of sanitary sewers?

It seems, then, that a closer look should be taken at Manning's equation since the "n" variance is not properly evaluated for sanitary sewer flowing at different depths. Perhaps new studies on sewer flows will reveal a better relationship for use in the design of sanitary sewers. Regardless of future "n" investigations, Manning's equation needs clarification so that the design of sanitary sewers can be placed on a more rational and economical level.

Evaluation of Investigation

Examination of the questionnaire, letter survey and the experimental results showed quite conclusively that poor joints were primarily responsible for blockages in sanitary sewers. The City Engineers of Michigan indicated the primary cause of blockage to be poor joints which could be directly attributed to poor construction practices. The experimental investigation of deposition showed that for clean pipe the joints were the chief consideration in determining the minimum grade.

In the light of the large economic factors involved, a reexamination of the entire matter of minimum grades for sanitary sewers appears to be necessary. The available evidence indicates that the minimum pipe grade, while important, is a secondary consideration relative to joints. Therefore, it seems that the design emphasis should be placed on specifications rigidly controlling proper pipe alignment, leak-proof joints and better pipe bedding.

Over the past fifty years, many 8-inch sewers have been laid at slopes considerably less than 0.4%. Reports from city engineers of various communities indicated little or no blockage resulting from these flatter slopes. Many of these sewers were laid with cement mortar joints which seriously disrupted the flow pattern at the closely spaced joints. Hence, it would appear that modern methods of pipe joining would allow for grades much less than 0.4%.

Little progress has been made since 1925 on the design considerations for sanitary sewers. Probably the greatest fallacy in present design methods is that of assuming "flowing full conditions" when selecting the slope for a given sized pipe. Furthermore, scant attention is paid to the known variance in "n" with flow depth and the resulting potential deposition problems.

A more practical method of design is one which considers the flow variation and results in a rational selection of the pipe slope with proper regard to deposition. Prior to the adoption of any design method, certain assumptions are necessary. The assumptions for the following suggested design method were:

1) That a minimum allowable sand deposit of 4 mm. should be used for design, since the questionnaire indicated that particles of 0.03 to 3 mm. have been found on the sewer inverts.

2) That sewer flow pattern based on studies by R.N. Hunter[21], and assuming that a block is composed of ten dwellings, five on each side of the street, gives a reasonable approximation of sewage flow.

3) That sand particles create blockages more readily than organic materials. Studies by K.S. Scharman[22] and others[23] indicate that garbage grinders have no serious effect on sewers.

4) That the nomograph developed in this thesis is valid (takes into consideration changing "n").

5) That an "n" = 0.011 at full flow is representative of older sewers. Tests by Cosen[13] and Bloodgood and Bell[15] show this to be a fairly liberal value.

6) That uniform flow exists. (For ease of illustration)

The following trial and error procedure was used in selecting the minimum grades of Table 7-8.

1) Assume a grade and determine the full flow with the Manning formula.

2) Determine d/D for q/Q where Q is determined by (1) and q is the flow in the sewer.

3) Determine by Figure 7-4 the d/D required to prevent a 4 mm. particle from depositing at the assumed grade of (1). If the resulting d/D value is the same as that of step (2), the assumed grade was correct.

4) Repeat steps (1) through (3) until the d/D of step (3) is the same as the d/D of step (2).

Table 7-8. Recommended Grades for an 8-inch Sanitary Sewer

Blocks (starting from the upper end)	q(Avg.) (gpm)	Suggested (S) Grade
1	8.4	0.7%
2	22.8	0.58%
3	38.0	0.51%
4	53.2	0.47%
5	68.4	0.44%
6	88.6	0.40%
7	98.6	0.37%
8	114	0.34%
9	130	0.32%
10	145	0.28%

Where the pipe flows ½-full or greater, use S = 0.25%

Table 7-8 suggests the use of an 0.25% slope where it is known that the sewer will flow ½-full or greater for a period of ½-hour or more per day.* The selection of the 0.25% slope from the experimental data agrees with the observation made by Hazelhurst and Green regarding deposition as mentioned in Section III. Also, several cities mentioned in the Questionnaire reported no problems with 0.25% slopes. For an 8-inch sewer to be laid at this slope, it is mandatory that the sewer have proper alignment, proper bedding and leak-proof joints.

Superior construction is not excessively costly, and discussions by the author with design and field engineers revealed that proper construction practices are more economical in many instances than poor practices. A particular case involved an 8-inch sewer laid for a distance of 600 feet. Upon completion of the 600 feet, it was possible to look in one end and see the other end centered. When workmen knew what was required of them, there was better teamwork resulting in increased efficiency.

The experimental investigation indicated that, in general, the derived relationship agreed with deposition observed in actual sewers. The Questionnaire revealed that deposits of 0.03 mm. to 3.0 mm. were found for sewers flowing about 1/3-full at a slope of 0.3%. For a grade of 0.3% and a d/D of 0.33, the nomograph of Figure 7-4

* The suggested 30-minute period was based on test observations (see p. 66, Section VI) where it was observed that erosion was approximately instantaneous or did not take place at all.

shows that a sand particle of 2 mm. will deposit.

It is believed that the work reported in this thesis has value for establishing relationships which predict deposition of sand particles on the inverts of new 8-inch clay sewer pipes. The proposed design procedure, page 106, is also useful since it places the design of 8-inch clay sewers on a more rational basis than present design procedures by considering the effect of the pipe joints and potential deposition problems; it also eliminates the question of how "n" varies for less than full flow for any slope.

More important yet, this research points out the unfortunate lack of understanding that exists in present design techniques. In particular, the practice of designing all 8-inch sewers with a minimum 0.4% grade is seriously in error. Both the work of Camp and the work reported here show that the upper ends of 8-inch laterals require more than 0.4% slope. On the other hand, employing this slope for flows of more than 100 gpm results in excessive and unneccessary costs.

It would be unfortunate, however, if the information presented here is either neglected or treated as a final answer to the question: how can sanitary sewers be most economically designed with full assurance of functional reliability? It has been repeatedly emphasized throughout this thesis that no consideration has been given directly to many of the other important variables which must be considered in proper modern design practice.

The need now is one of accepting the data and conclusions presented in this discussion as a basis for examining the entire question of sewer design. Too little progress has been made relative to sanitary sewer design procedures. These limitations are most apparent when comparison is made with the development of design methods in other fields.

Because of limited time and financial support, the scope of this report was restricted to a study on clean 8-inch vitrified clay sewer pipes. Within the scope mentioned just above, the work is believed to be useful and reliable, and should be considered in any new investigations or in the development of new design procedures.

Recommendations for Future Investigations

The following recommendations are based on experience obtained in conducting this investigation. All recommendations are made on the assumption that sufficient funds and time are available.

1) This experiment has indicated that the suspended solids in the sewage have little effect on deposition. Therefore, it seems sufficient to conduct any short-term experiments with clear water.

2) To eliminate pump surging, provide a positive head on the suction line.

3) Construct a stilling well on the weir box to increase accuracy of flow measurements.

4) Construct the experimental pipe line with a 3-inch opening along the top for the line's entire length, leaving the entire line open for visual inspection.

5) Set the pipe line on steel beams. The beams should be supported vertically by hydraulic jacks, centrally controlled, permitting the slope of the pipe to be changed at will to study any desired effect.

6) For the control box at the end of the line, construct a narrower sluice gate. Experience with this investigation indicates that the width of the opening should not be greater than 3 to 5 inches.

VIII CONCLUSIONS

The following conclusions were drawn from the observed experimental data and an analysis of that data:

1) Deposition was definitely influenced by the dependent variables, pipe slope, particle size and depth of flow.

2) Analysis of the data for 8-inch sewer pipe, showed the relationship between the depth of flow (d/D), the largest size particle (p) in mm. which would deposit on the invert or at the joints and the per cent slope (S) to be $d/D = (-0.63 \log S - 0.05)p^{1/3}$.

3) Solids suspended in the sewage apparently had little influence on deposition.

4) No one given velocity is a sufficient criterion for preventing deposition in clean pipes.

5) Pipe joints were the principal influence in initiating deposition.

6) Manning's "n", as calculated from the deposition data, increased with decreasing slope.

7) Further investigation of deposition relative to minimum grades and pipe joints is definitely warranted in order to place the design of sanitary sewers on a more rational and economical basis.

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