EXTRANEOUS FLOW IN A CITY OF WINNIPEG SEPARATE SANITARY SEWER SYSTEM

A Thesis

Presented to

The Faculty of Graduate Studies and Research

University of Manitoba

In Partial Fulfillment

of the Requirements for the Degree

Master of Science in Civil Engineering

by

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1972

ABSTRACT

The variations in sewage flow rates in a separate sanitary sewer system in the City of Winnipeg were studied during dry and wet weather conditions. It was found that massive amounts of stormwater or extraneous flow enter the sanitary sewer system during even moderate rainfall. Peak extraneous flow rates in excess of seven times the average dry weather flow were monitored. The thesis contends that the chief source of this extraneous flow is the foundation drains around the basements of houses and that poor surface drainage of house lots is a major contributing factor.

It was found that the volume of extraneous flow was relatively small, being about three percent of the total annual sewage flow, but that the flow pattern is characterized by high short-term peaks. Good statistical correlation was found between the volumes and peaks of extraneous flow and the probable frequency of occurrence of rainstorms.

The traditional design procedure for determining the flow capacity requirements of sanitary sewers is based on the projection of water consumption in the service area. It is shown that the critical factor governing the hydraulic adequacy of the sanitary sewers is the magnitude of the extraneous flow occurring in the sewers. The study concludes that this flow component must be given priority in future designs.

The thesis studies various methods of dealing with this extraneous flow problem. It is not considered likely that this flow can be completely eliminated at the source. The separate connection of foundation drains to storm sewers is shown to be costly. The simple overflow of sewage to

the receiving stream is not recommended and probable annual overflow pollution levels are calculated. The local collection sewer system must be capable of carrying the sewage flows produced while providing a reasonable degree of property protection and an allowance for extraneous flow is suggested for design purposes. It is not economical to provide flow capacity in the interceptor and treatment plant facilities for these short peak flow rates. The recommended solution is to reduce extraneous flow at the source and to provide suitable storage and/or treatment at the interface of collection and interceptor sewer systems for the unavoidable extraneous flow.

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LIST OF ABBREVIATIONS AND SYMBOLS

gal.	- gallons (unless otherwise noted, the U.S. gallon is used in this report)
g.p.a.d.	- gallons per acre per day
gpm	- gallons per minute
gphr	- gallons per hour
DWF	- dry weather flow
EFPF	- extraneous flow peak factor
in./hr.	- inches per hour
B.O.D.	- biochemical oxygen demand
S.E.	- standard error of estimate
r	- coefficient of correlation
t_L	- time lag between centre of mass of rainfall and peak extraneous flow rate
ppm	- parts per million
cfs	- cubic feet per second
ppa	- people per acre
gpcd	- gallons per capita per day

CHAPTER 1

INTRODUCTION

Pollution and sewage treatment are topics which have generated considerable public interest and discussion of late. A growing awareness of the need to protect their environment has made the public become pollution-conscious to an encouraging degree. Much of this attention with respect to sewage from urban living has been focused on the building of bigger and better sewage treatment plants and the merits of removal of nutrients such as phosphates. The conduits for the transmission of sewage have been taken for granted, even by the people involved in the field of pollution control, as being adequate for their purpose, especially the separate sanitary sewer system. In many cities it appears that the sewers themselves are in fact very much a part of the pollution problem and these problems are appearing under unexpected circumstances. The combined sewer system, designed to convey both sanitary sewage and storm water, has long been recognized as having serious short comings due to overflows in time of rainfall. Now the separate sanitary sewer system, commonly regarded as the highest standard of sewer service, has been shown to be vulnerable to rainwater intrusion, the very problem it was designed to avoid.

The first sewers in North America were designed for the purpose of conveying storm water and not sanitary sewage.[1] Initially, the connection of houses to storm drains was strictly prohibited. When these connections were permitted, these first storm sewers became combined sewers. There was no apparent need at that time for a separate storm sewer and sanitary sewer system and combined sewers were the normal method of sewer service for decades. In Canada today more than thirty-seven percent of the total population and fifty-three percent of the urban population are still served by combined sewers.[2]

When treatment of sewage became necessary, it was not feasible to construct treatment plants of sufficient capacity to cope with the entire combined storm and sanitary sewage flow during rainfall. Consequently a major portion of these wet weather flows were by-passed to the receiving water with a combined sewer system. The separate sanitary sewer system was developed to separate the domestic and industrial wastes of the community from the urban storm runoff. Although more expensive initially than the traditional combined sewer system which handled both sewage and storm water, the idea that the sewage proper could be selectively conveyed to treatment seemed an eminently logical approach to pollution abatement. This concept promised less costly transmission and treatment works and reduced pollution of the receiving waters. Instead of constructing large costly sewers and treatment plants to cope with sewage diluted by rainwater or, alternatively, discharging large quantities of diluted waste into the receiving streams, the sewage itself could be transferred to treatment free from the influence of storm water. The separate sanitary sewer system has been accepted as good practice and is now commonplace. The construction of combined sewer systems is an unusual situation in North America today. In fact, many existing combined sewer systems have been converted to a separate system at considerable cost and most cities have given serious thought of the feasibility of the separation of their existing combined sewer systems.

Recently, however, it has become apparent that the theoretical separation of storm and sanitary sewage is illusory. Large quantities of storm water are in fact entering the separate sewers and it develops that these sewers are still very much influenced by rainfall. This extraneous flow has in many cases overtaxed the capacity of the sewers resulting in flooded basements and increased transmission and treatment costs. Further, the severe flooding has often forced the local authorities to discharge raw sewage to the rivers in order to relieve the overburdened sewers, thus in part reverting to a combined sewer system.

The recognition of these weaknesses in the separate system has placed the designer in a dilemma. He is faced with the knowledge that the existing design criteria are unrealistic since they do not recognize the conditions as they exist but not enough is known about the nature and extent of this extraneous flow for the designer to adequately correct deficiencies in either the design of implementation of this design. The problem demands greater study to place the problem in its proper perspective and enable public agencies to evaluate possible solutions. Improved pollution control must start in the sewer system itself.

This study is intended to evaluate the magnitude and extent of the extraneous flow in the separate sanitary sewer system of a typical suburban area in the City of Winnipeg and to evaluate some of the causes and effects of this problem. It must be recognized that the results of this study will be applicable only to the City of Winnipeg and surrounding area due to the fact that extraneous flows are dependent upon the soil type, rainfall pattern and construction practice, etc., peculiar to the area.

CHAPTER 2

THE TYPICAL SEPARATE SANITARY SEWER SYSTEM

The introduction traced briefly the evolvement of the modern separate sanitary sewer system from the original storm sewers to the present day where this system is the accepted standard of service for new developments. The following discussion will describe the nature of the system.

A. Definition of a Separate Sanitary Sewer System

An accepted definition of a separate system is given as "a sewer system comprised exclusively of sanitary sewers which carry only sewage and to which storm water and ground water are not intentionally admitted, also referred to as a sanitary system or a separate sanitary system."[1] The main intent of the sewer system is to transfer the "fouled" water supply of the community or domestic sewage, industrial wastes, and unavoidable ground water infiltration to a selected point of treatment or final disposal.

B. The Layout of the Sewer System

For this system, waste from the individual household is conveyed via a building sewer, sometimes called a "house sewer" or "house connection" to the sanitary lateral sewer fronting the property. In the

Winnipeg area, it is common house-building practice to install foundation drains or weeping tiles around the basement foundation which are also connected to the building sewer. It should be noted that this is a contravention of the definition stated earlier. However, the connection of weeping tiles from basement foundations to the sanitary sewer, while discouraged in many city ordinances, is not peculiar to the Winnipeg area alone. In Winnipeg, as in most areas, direct connections of roof and yard drainage to the sanitary sewer are strictly prohibited.

The sanitary sewers are usually located in the centre of the street at a depth of at least eight feet below street level. A single sewer thus serves both sides of the street and house connections are approximately the same length for houses on either side. A sewer having only building sewers connected to it is termed a "lateral" sewer.[2]

Manholes are located at the junctions of sewers and changes in alignment, size or grade. Often, manholes are installed at preset intervals, usually about 300 to 500 feet, to enable cleaning of the sewers. Street intersections almost invariably have manholes due to the junctions of sewers at this location.

The lateral sewers are usually a minimum of eight inch diameter and 0.4 percent slope for self-cleaning purposes and together with other lateral sewers discharge into a branch sewer which in turn discharges into a trunk sewer. The branch and lateral sewer system is sometimes called the local collection sewer system. Intercepting sewers are often located below the trunk sewers to accept gravity flow from the trunk sewers. In Winnipeg, there is little topographical relief and most trunk sewers are at elevations below the interceptors, forcing the pumping of sewage into the interceptors. The interceptors convey the collected sewage to the point of treatment or disposal.

The routes of the interceptor and trunk sewers will have a pronounced effect on the sewer planning in any given area. To a large degree, their proximity and depth determine the extent of pumping required in the tributary system. Boundaries and shape of the drainage area will have obvious influences in the planning of the sewer layout.

C. Pumping Stations

The degree to which pumping will be required in a sewer system will depend mainly on topography, geology, and location of the treatment plant or point of ultimate disposal. In Winnipeg, the flat topography, the need to avoid excessively deep cuts due to bed-rock, and the relatively long distances to the points of sewage treatment dictate considerable pumping. There are forty-one pumping stations in the portion of the city served by combined sewers and about thirty-five stations on separate systems in the suburban areas.[3,4]

The stations consist usually of two or three pumps located in a substructure, the depth being dictated by the depth of the incoming sewer. The pumps are of non-clogging design but stand-by pumping capacity for pump failure is common practice. A wet-well or sump is provided adjacent to the sewer which provides storage to equalize the incoming sewage flows to pumping rates. Liquid level in the wet-well is used to regulate the pumping intervals. If feasible, most pumping stations have an overflow to the river for use in the event of power failure or excess flow in the sewer. If the river is too distant, storm sewers are sometimes used for this relief.

D. Treatment Plant

The treatment plant is the final barrier between the collected sewage and the receiving water. In general, the purpose of the treatment of sewage is to remove sufficient contaminants from the sewage so that the treatment effluent will not interfere with the best use of the receiving streams. This treatment can be done by several methods and to varying degrees. The current acceptable method of sewage treatment is to provide secondary treatment, usually by the activated sludge process. The main treatment plant in Winnipeg is of this type. Plain sedimentation or primary treatment usually precedes the activated sludge process.

The activated sludge process is a biological contact process where living aerobic organisms are brought into contact with organic solids in the sewage in an environment deliberately maintained so as to be favorable for the aerobic decomposition of the solids.[5] Adequate dissolved oxygen must be maintained at all times and the mixture must be continually seeded with vast additional numbers of organisms in order to accomplish efficient reduction of the pollutional characteristics of the sewage. This seed is termed "activated sludge", a brownish floc-like substance teeming with bacteria and other micro-organisms, which is recycled into the process to retain biomass. The control of the biological phenomena in this process is a delicate matter. The preliminary sedimentation, quantity of air, aeration time and proper amount of activated sludge returned are vital control parameters and are all greatly influenced by the flow rate and volume. Physically the plant components are determined mainly by the peak flow rates anticipated. It is not the intention to discuss the details of the operation of a treatment plant but rather to provide a brief glimpse of the nature of the process since it will be shown to be a factor in the deliberations as to how best to cope with extraneous flow.

CHAPTER 3

PRESENT DESIGN AND CONSTRUCTION PRACTICE

The design and construction of sewer systems is a critical factor in municipal engineering economics and efficiency. The finished product is hidden from public view and, when functioning properly, is usually taken for granted.

The purpose of the sanitary sewer is to convey waste to a point of treatment or disposal. The design of a sewer system then involves a determination of the waste flows that will be generated from the area and the selection of the optimum size, slope and depth of the sewers to accomodate these flows. The construction phase is the physical implementation of the design. The theory of the design must, in order to be effective, take cognizance of the realities of the construction phase. The following discussion will deal with the typical current approach to the design and construction of separate sanitary sewer systems.

A. Estimate of Sewage Flows

Of paramount importance in the sewer system design is the required hydraulic capacity or design flow rate. Some of the factors involved in this determination of this flow rate are outlined below.

1. Design Period

A projection of estimated flows used for determining sewer capacity must be made for the design period. The design period is usually selected according to the local situation. Lateral sewers are usually designed for the saturation development expected in that area but the larger trunk and interceptor sewers are usually designed for the nature and extent of development anticipated over a twenty-five to fifty year period to reflect the half-life of the structure. Locally, the fifty-year period is common for interceptor sewers.

2. Population and Land Use

The population and the character of the land use are important factors in the total sewage flow generated from a given area. There are many mathematical and logical methods used for predicting population, all of which are subject to the judgement of the forecasters. Usually, local agencies will have developed population estimates for the entire city and also for sub-sections of the city. The local planning and zoning authorities will have regulations and predicted land usage projected in the tributary areas. This information will include permissible population densities, industrial areas, park areas, etc. With this information and the knowledge of the population projection for the overall city area, a population projection for the tributary area can readily be made for the design period.

3. Average Per Capita Domestic Sewage Flows

The domestic or residential sewage is estimated by a projection of per capita sewage flow. This projection is often made from the water consumption records of the city. Sewage flow will usually be less than the water usage due to uses such as lawn watering, leakage and "unaccounted-for-water" (hydrant flushing, street sweeping, meter inaccuracies, etc.).

The trend, both locally and nationally, has been for increased water consumption per person.[2] This factor must be given due consideration in the projection of sewage flows. The amount that the sewage flow will be less than water consumption will vary with the area and local records must be used for this estimate. The average domestic sewage flow may range from fifteen to seventy gallons per capita per day.[3]

4. Commercial and Industrial Sewage Flows

The sewage flow generated from commercial and industrial establishments is usually estimated on a flow per acre basis. Where sufficient detail is known, certain establishments may be assigned specific flow rates. Otherwise the areas are given overall unit flows per acre based on the area anticipated for development and the nature of the zoning placed in the area.

In the Winnipeg area, typical design rates are 1500 - 5000 gallons per acre per day and 2500 - 8000 gallons per acre per day for commercial and industrial usage, respectively.[4]

5. Infiltration

In addition to domestic, commercial and industrial sewage flows, some infiltration of ground water into the sewer will occur. The amount will depend on the type of pipe and joints, type of soil, groundwater conditions, size of sewer, condition of pipes and manholes, and especially the workmanship in the installation of lateral sewers and building sewer connections. In a given sewer system, the sizes up to twelve inch diameter inclusive may be ninety-five percent of the total sewer length. Infiltration into house connections has been found to be up to ninety percent of the total infiltration for a system.[5] The designer must provide capacity for the infiltration rate judged to be reasonable.

There are various methods of allowing for infiltration. It is becoming more common to use an allowance in terms of gallons per day per inch-diameter per mile rather than the traditional gallons per acre per day basis. The amount of infiltration permissible is typically specified in the construction specifications. Normally, the specifications range from 4500 - 6000 gallons per mile per day for twelve inch diameter pipe.[6] Locally, a figure of 200 gallons per acre per day has been used frequently in design and found to be appropriate for winter dry-weather conditions.[7]

Sometimes a nominal allowance is made for storm water intrusion into separate sewers. The typical manual recommends some consideration be given to this possibility but the amount of the capacity to be assigned to this flow component is not given due to inadequate information.[8] Locally, some designers, aware that this flow intrusion does exist, are making some arbitrary allowance for this factor.

6. Variations in Sewage Flow

The flow rates in a sewer vary continuously throughout the day. The degree of fluctuation is influenced by the season, size of area, land use and population. Usually, the domestic sewage flow will be at a minimum from 2:00 to 6:00 A.M. with peak flows occuring during the latter part of the morning.[9] A widely-used emperical formula for predicting domestic or residential sewage ratios of peak to average flow rates is the so-called Harmon formula, which is described by the equation $1 + \frac{14}{4 + \sqrt{P}}$ where P is the population is thousands.[10] This ratio, or dry-weather peak factor, is used to obtain the "peak hour" flow rate from domestic sewage. This formula results in peak factors ranging from about 1.0 to 4.0.

In addition to the above peak factors, a factor for "uncertain development" is sometimes used to allow for greater than expected development in a particular area, this allowance ranging from zero to thirty percent, depending on the size of the design area.[11]

The variations in commercial and industrial sewage flows are less severe and are usually assigned peak factors based on local experience. These peak factors are in the order of 1.5 to 3.75 for commercial and 1.1 to 2.75 for heavy industrial sites.[12]

The various flow components, namely, the domestic, commercial and industrial sewage flows, multiplied by the appropriate peak factors are added to the infiltration allowance to produce the design peak capacity requirements. The infiltration component is assumed to be constant during intervals of wet soil conditions.

B. Hydraulics

The hydraulics of most sewer systems are straight-forward with most of the sewers being designed as open-channel conduits according to traditional formulae.

The total available energy for gravity sewage flow is severely limited in most sewer systems. At the upper end, the depth of the basements of the dwellings will dictate the elevation of the sewer and the interceptor sewer will usually govern the downstream elevation. Intermediate pumping is expensive and normally avoided if possible. The designer, by selection of size and slope of sewers, attempts to use the available energy in the optimum manner to conduct peak flows from inlet to outlet. A somewhat conflicting requirement is the need to provide for self-cleansing velocities, usually 2.0 feet per second, in the sewer at minimum flows. The need to conserve energy to discharge peak flows is usually given first priority at the expense of some compromise in desirable minimum velocities.

The sanitary sewers are usually placed at a depth of about three feet below the basement floor of the houses at the top of the street.[13] Locally, this means the sanitary sewers are at least 8.5 feet deep in the streets. House connections are usually laid on a slope of two percent or one-quarter inch per foot. The slopes of the lateral sewers are kept parallel to the ground surface, if possible, to minimize excavation.

C. Types of Sewer Pipes and Joints

1. Types of Pipe

There are numerous types of sewer conduit available to the designer and almost all have demonstrated that the material can be made effectively free from infiltration of water and can meet typical specifications for strength and corrosion resistance. The most common types of sewer pipe in use locally are probably concrete pipe and asbestos-cement pipe. The concrete pipe is available both in the plain or reinforced concrete design and is usually the specified material where high strength is required. Pipe lengths are four to six feet, thus dictating numerous joints. The asbestos-cement pipe is much lighter and easier to handle than concrete. The typical laying length of asbestos-cement pipe is thirteen feet. For building sewers or other special situations, cast iron or ductile iron pipe may be supplied. The various plastic and polyethylene pipes are gaining in use. Locally, several river crossings for sewers have been installed using polyethylene pipe with apparently good results. The intervals between joints are up to forty feet. This long laying length minimizes the number of joints in the sewer system with consequent reduction in infiltration.

2. Pipe Joints

In terms of preventing the exfiltration or leakage of the sewage from the pipe or infiltration of ground water into the sewer, the adequacy of the joint becomes very significant. A common axiom is that no sewer system is better than the joints in the system. There are numerous types of joints available today.

Cement mortar joints are rarely recommended any more. These joints are rigid, tend to shrink and crack, and will not adjust to even minor displacement of the pipe. Asphaltic or butuminous joints are superior to the cement mortar joints in that they are more flexible. However, considerable care is necessary to obtain a permanent joint. Compression gasket joints use a natural rubber or other similar gaskets. They are used on asbestos-cement, concrete and plastic sewer pipe. These joints allow considerable deflection of the pipe without loss of effectiveness of the joint and are considered to be the most effective against infiltration.[14] Chemical or heat-welded joints are limited to the plastic pipes and provide a water-tight seal but the long-term stability of this technique has not been established as yet.

D. Construction Practise

The effectiveness of the sewer system in meeting its intended purpose will often be determined by the physical act of construction and

and in many aspects, such as infiltration, the construction phase is by far the most critical factor in determining the final worth of the system.

Most sewer construction is by the open-trench method, where the sewer is laid in an excavated trench open to the surface. The trench is kept as narrow as is possible but still will permit adequate working and inspection room. The trench width is a direct factor in determining pipe strength requirements.[15] The contract specifications will usually stipulate a maximum permissible trench width, length of time the excavation can remain open, dry working conditions etc. After excavation of the trench, a bed of free-flowing graded sand is prepared on the trench bottom. The pipe lengths are jointed in the trench, the joints inspected, and the pipe backfilled with granular material to the depth specified, usually at least to the mid-point of the sewer. Backfill above the pipe is native soil unless specified otherwise. The physical room in the trench for adequate inspection of the joints is limited and the frequency of the joints is usually high, making proper inspection of joints difficult. Faulty pipe joints are the source of most pipe infiltration.

In order to reduce infiltration, manholes are usually specified to have rubber-gasketted joints, if they are of the pre-cast barrel type, although proper installation of gaskets is often not realized. Manhole covers are sometimes specified to be solid for the same reason. Many manhole covers have several holes for ventilation purposes. It has been tested that leakage through manhole covers may be twenty to seventy gallons per minute if the depth of water is one inch over the cover.[16]

Infiltration specifications usually give a maximum limit in the order of 500 gallons per inch of diameter per mile of sewer. Locally, the former Metropolitan Corporation of Greater Winnipeg specified 0.1 gallons per inch of pipe diameter per foot per day.[17] The specifications stipulate that the actual infiltration shall be measured by the contractor prior to acceptance.

The construction of a complete typical sewer system will usually cross the jurisdiction of several authorities. The local situation is perhaps somewhat more complicated than most areas but is an example of this occurrence. The individual houses will have the plumbing extended from the house by a "stub-out", an eight foot extension of the house sewer. The extension of this "stub-out" is usually done under building by-laws and inspected by the building inspectors. From the "stub-out" to the lateral sewer, the municipal sewer by-laws and regulations apply and inspection is under the municipal authority. This construction is at least partly on private property and may not receive its proper share of attention. The fronting sewer is also usually under the local municipal authority. In the past, trunk or interceptor sewer was most often under the jurisdiction of the former Metropolitan Corporation of Greater Winnipeg. The split jurisididction did not favour a consistent standard of inspection. It is also true that the larger sewers are often given greater attention with respect to inspection in spite of the fact that by far the greatest length of sewer in the system is of the smaller size. In fact, the total building sewer length alone may be equal to the length of all the remaining sewers.[18]

E. House Lot Drainage

The original intent of the separate sewer system was to eliminate the storm water from entering the sanitary sewers. Most areas have regulations prohibiting the drainage of roof run-off into the sanitary sewers. The direct connection of downspouts from the roof into the sanitary system is usually prohibited by law. The early thinking with respect to roof drainage obviously considered that once the roof run-off was directed onto the ground surface of the lot, it would eventually find its way into the storm drainage system. The Winnipeg By-laws reflect this approach. Run-off from a roof is prohibited from draining into the sanitary sewer, but weeping tiles are permitted as a connection to the sanitary sewer provided storm water from the roofs is not discharged on the ground closer than four feet to the foundation wall.[19,20] The lot grading is stipulated to provide adequate surface drainage over the entire area.[21] It is not stipulated that the direction of drainage should be away from the house.

A survey of a subdivision in St. Vital in 1965 indicated that over eighty percent of the houses had roof downspouts discharging less than four feet from the foundation walls and also eighty percent of the lots drained towards the house. These conditions were considered representative of local subdivisions.[22]

Locally downspouts from the roofs of new houses are usually extended the necessary four feet from the building initially but the extensions are often quickly removed by the first owner because of the inconvenience or obstruction to walkways. Foundation drains are almost invariably connected to the sanitary sewer. The local clays often expand when wet and the foundation drains help maintain a uniform moisture content. The weeping tiles are essential to prevent heaving of the basement floor and the seepage of groundwater into the basement.

Basement floors are built within the walls and foundation as shown on Figure 1. The only resistance to flotation is the weight of the basement floor which is relatively light, being four to six inches thick. One foot piezometic head caused by groundwater above the basement floor would thus tend to heave the floor or cause cracking to relieve water pressure. The alternative is to relieve the groundwater by a ring of perforated pipe, or weeping tile, laid on crushed rock. This drainage must be taken to a point of discharge lower than the basement. If a storm sewer does exist in the front of the house, it would likely be too high to enable drainage of the weeping tile. Therefore, the simplest alternative is to allow this connection to the sanitary sewer. The problem is aggravated by the admission of window-well drainage into the weeping tiles. These window-wells, usually corrugated metal walls around the basement windows with a bottom lined with crushed rock, are often beside sidewalks which settle and bring surface drainage into the window-wells and directly into the weeping tiles.

Construction of the basement for the house is typically done by pushing the soil material towards the back of the lot. The excavation is fairly close to the basement dimensions laterally but longitudionally the hole is much longer than required due to the nature of the excavation method. The backfill around the house is rarely compacted to any degree and much debris finds its way into this backfill. As a result, the area around the house settles and a significant portion of roof drainage, even if extended four feet from the foundation wall, is likely to drain back to the house and ultimately to the weeping tiles. Many new developments

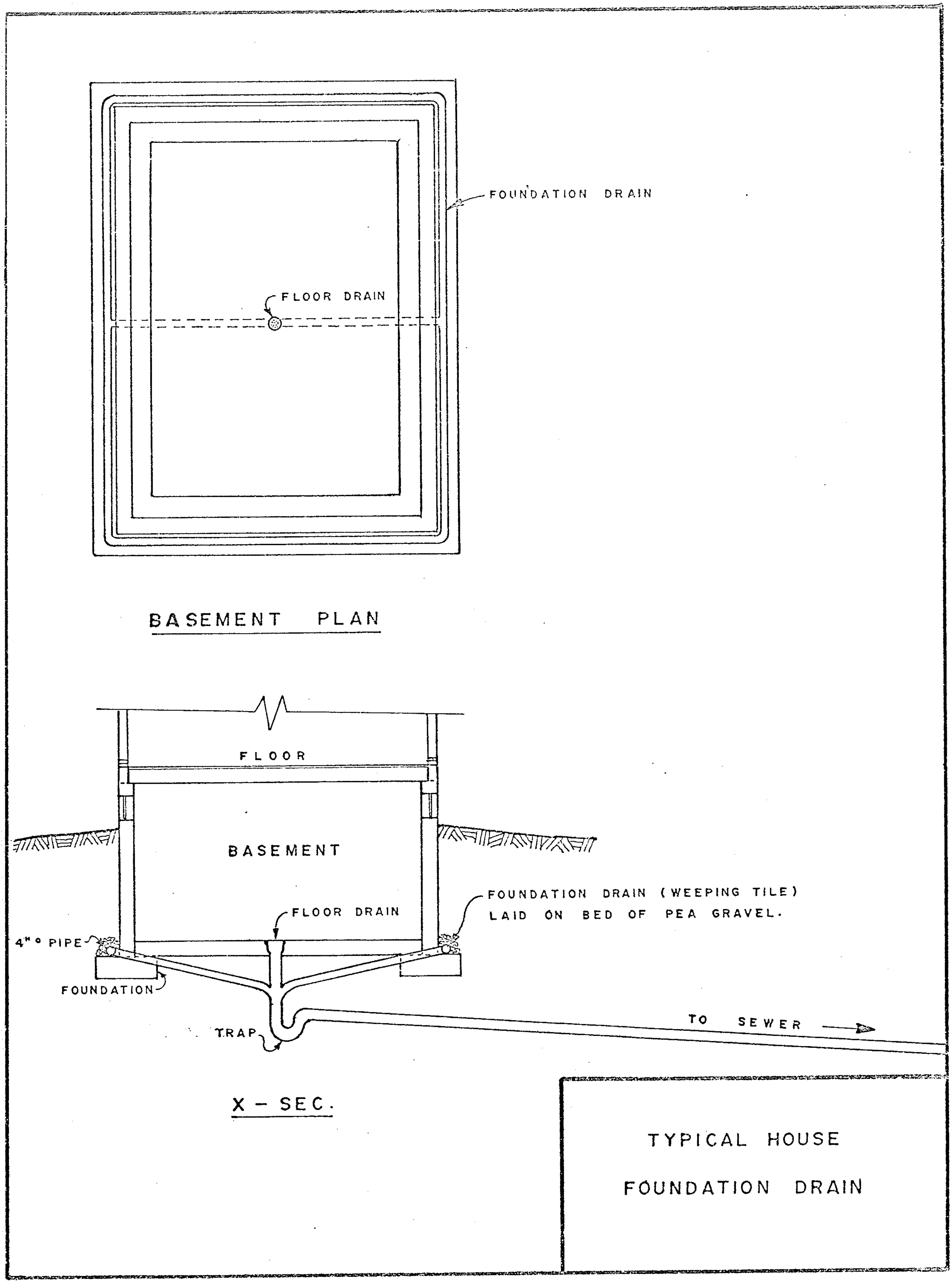

FOUNDATION DRAIN
FLOOR DRAIN
BASEMENT PLAN
FLOOR
BASEMENT
FOUNDATION DRAIN (WEEPING TILE)
LAID ON BED OF PEA GRAVEL.
FLOOR DRAIN
4" ø PIPE
FOUNDATION
TRAP
TO SEWER
X - SEC.
TYPICAL HOUSE
FOUNDATION DRAIN

FIG. I

do not have service lanes and lot grading is from the back lot-line to the street. This means all surface run-off must drain past the house and again it is likely that some portion will be intercepted by the settled depression around the house proper and ultimately be drained to the weeping tile.

In some local municipalities, recognition of these problems has resulted in increased attention being given to surface drainage of lots. Swales or shallow surface drains have been constructed in the back of the lots in some areas to provide drainage parallel to the streets. Lot drainage is then split with the house being the high point in the lot. Municipalities are also applying tighter control on the grades and elevations. The municipality will stipulate the elevation of the four corners of the house and a small deposit is held to encourage conformity to the regulations. To guarantee conformity would require individual inspection and the limited personnel does not allow this in many cases. There is no continued inspection to ensure that the lot grading is maintained. In fact, most homeowners tend to alter the original landscaping.

F. Is the Present Practice Realistic?

A brief glance at a few case histories amply answers to the above question. In September 1969, the City of Winnipeg monitored sewage flows in the separate sewer system in the Rosser Area of North West Winnipeg.[23] Relative to the water consumption of the area, the sewage flow rates were 5:1 for average dry weather conditions, 19.4:1 for maximum rainfall during the summer, and 153:1 for spring run-off conditions. Transcona, a municipality of about 6000 homes in the City of Winnipeg, referred to 300 homes experiencing flooding in an information bulletin to the citizens.[24] Many similar reports of floodings and excessive peak factors are to be found in Winnipeg and in other areas. Peak flows in Johnson County, Kansas have been reported in excess of twenty times the normal dry-weather sewage flows.[25] The problem of rainwater intrusion into the sanitary sewer is widespread and, until recently, has not received proper recognition as a major problem and still has not received adequate study and evaluation of the causes and effects. There is little doubt that this extraneous water is exacting a considerable price, either in flooded basements, added sewer capacity or pumping capacity, treatment costs, or river pollution effects of overflows.

The basic concept that a separate sanitary sewer is essentially free from intrusion of rainwater is wrong. The connection of foundation drains to the sanitary sewer is a permitted contradiction at the outset. In Johnson City, Kansas, dye introduced into a downspout discharging onto a splash pad was found to appear in the closest sanitary sewer manhole nine minutes later.[26] The time of travel from the house itself was seven minutes, indicating a very direct connection from ground surface adjacent to the house to the weeping tile. This may be a commonplace phenomenon with foundation drains.

The temporary nature of lot grading due to settling around the house is a factor that has not been considered in design. Attention to window wells, downspouts, splash pads, grades of drainage swales, etc., are all fairly recent additions to the check list of the inspectors. The building sewer is probably still not receiving its due share of attention.

Some designers have recognized the failure of present practice to consider these aspects and have adopted rather arbitrary allowances to compensate for extraneous flows. In Kansas City, Missouri, an average of 1.25 gpm per house has been allowed for foundation drains.[27] This is certainly better than ignoring the problem but, rather than assigning arbitrary allowances to this factor, it would be much more satisfactory to obtain the necessary data to permit a thorough evaluation of the best methods of dealing with the problem. The local authorities have, in some cases, shown commendable concern and recognition of the problems but existing regulations are not entirely appropriate. However, until the entire problem is examined, it is difficult to determine which area or areas should be given priority in an attack on the problems.

The conclusion is inescapable that present design and construction practice is not consistent with the actual situation and that the problem of extraneous flow in sanitary sewers demands greater study to determine effective revisions in current practice.

CHAPTER 4

THE STUDY METHOD

It has been illustrated that the inflow of large amounts of rain water into separate sanitary sewers is far in excess of that anticipated in the design of the sewer system. In order to define the magnitude of this extraneous flow and to study the relationship of this inflow and rainfall the following procedure was adopted.

A. Procedure

A representative sub-division was chosen for test purposes. Within this area, the procedure was to monitor sewage flow from the district both in dry weather and during rainstorms. In order to be able to detect the inflow from rainwater, it was essential to establish the normal daily diurnal flow pattern during dry weather. Hydrographs of sewage flow were developed during rainstorms and, when super-imposed on the dry weather hydrograph, the effect of the intrusion of rainwater could be readily seen.

To permit the correlation of stormwater in the sanitary sewers to the magnitude of rainfall, it was necessary to ensure accurate continuous monitoring of rainfall. The measurement of account and duration of rainfall would then allow classification of rainstorms as to probable frequency of occurrence.

With respect to the selection of a test area, some of the factors to be considered were the size of area, nature of development, sewer system and adaptability to flow measurement. The area should be typical of relatively new subdivision developments, that is, mainly residential in character with single-family dwellings, fairly large lot sizes and should be fully developed. The area should be fairly small to facilitate the collection of rainfall data that will be accurate for the entire tributary area. The sewer drainage area should be well-defined and have one location where all sewage flow can be suitably measured.

B. The Study Area

The Pulberry Sewer District in the Municipality of St. Vital, was selected for testing. Figure 2 defines the limits of the drainage area. This district is almost fully developed with single family dwellings. The area began development in 1958-59 and is typical of residential subdivisions of the past decade.[1] The houses are well kept and set on a street pattern of bays and loops. There is no commercial or industrial development in the district. This area has been subject to recurrent basement flooding with particularly severe flooding in 1968.[2] The district is about 120 acres in size. Originally, the sewer system was designed to service an additional ninety-five acres but this area was drained elsewhere partly because of flooding.[3]

The actual population of the district is not available. The population of the Pulberry district including a small area to the north and south of the drainage limits was reported by the Planning Division of the Metropolitan Corporation of Greater Winnipeg in December 1968.[4]

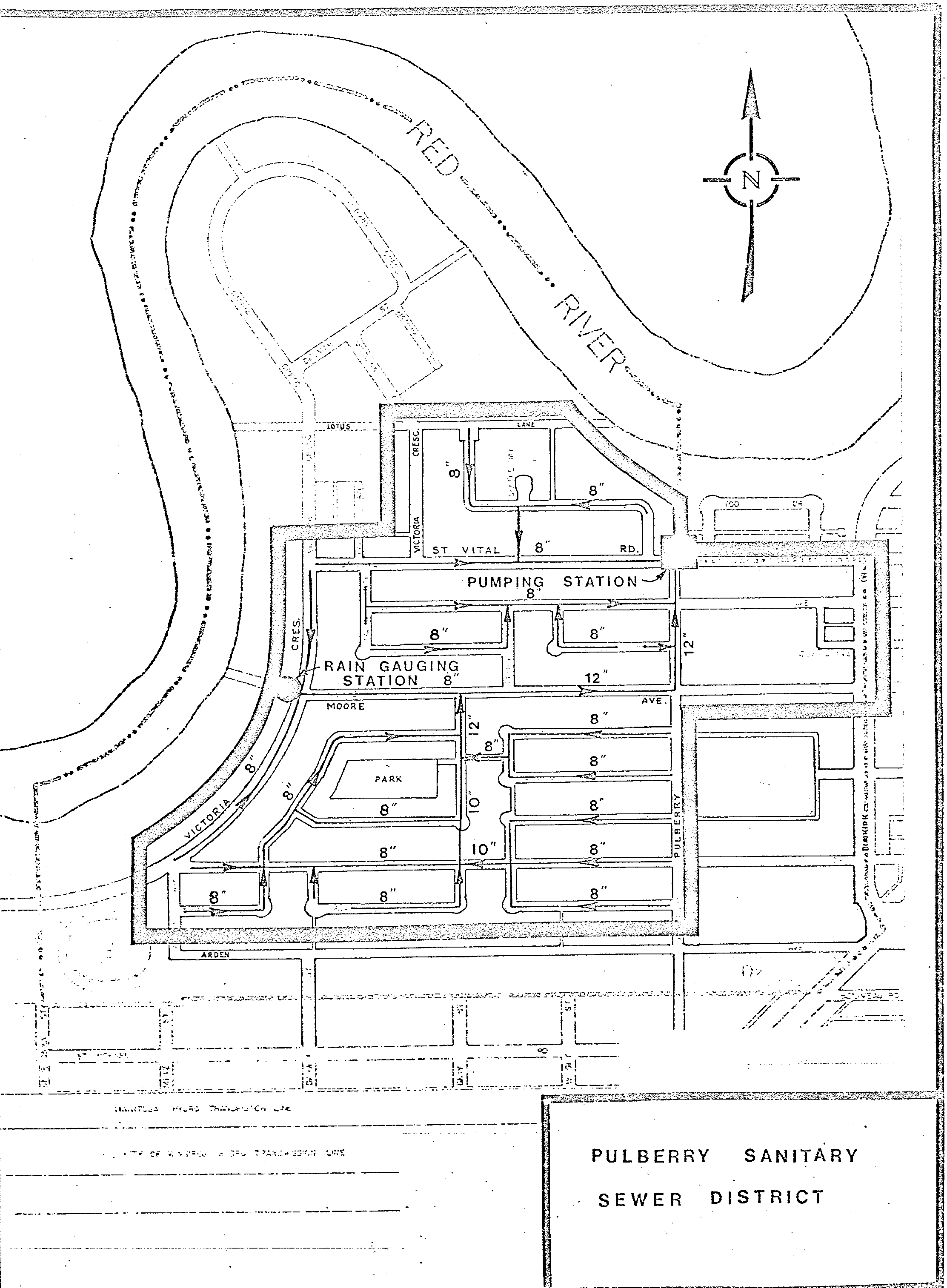

RED RIVER
N
LOTUS
LANE
CRESC.
VICTORIA
ST VITAL
RD.
8"
PUMPING STATION
RAIN GAUGING
STATION
CRES.
MOORE
AVE.
12"
10"
PARK
VICTORIA
PULBERRY
DUNKIRK
ARDEN
PULBERRY SANITARY
SEWER DISTRICT

FIG. 2

The area was almost fully developed at that time. Using this reported population and making a calculated deduction for the number of homes and people beyond the drainage limits of the Pulberry district, the population of the tributary area was estimated at 2760 and the number of dwellings was similarly calculated to be 700.

The sewer system for the Pulberry district is of the separate sanitary sewer design. Sewage is collected in the sanitary sewers and conveyed to a pumping station at Pulberry Street and St. Vital Road where the sewage is lifted to a branch sewer from where it ultimately flows to a treatment plant. Storm sewers have been provided in the district and discharged into the Red River at Moore Avenue. The sewers are basically of the concrete and reinforced concrete type with rubber gasketted joints.

C. Rainfall Data Collection

The intersection of Moore Avenue and River Road was chosen as the location for the collection of rainfall data. See Figure 2. At this location, it was possible to mount the rain gauges on the roof of a utility trailer. The location is central with respect to the drainage area and thus rainfall data should definitely be representative of the tributary area. A recording rain gauge and a standard rain gauge were used to measure rainfall.

The standard gauge has an eight inch diameter circular receiver from which water passes to a cylindrical measuring tube. This tube is 2.53 inches in diameter which gives it an area one-tenth that of the receiver. When one inch of rain falls in the larger receiver the measuring tube fills to a depth of ten inches.[5] This permits simple and accurate measurement of total precipitation.

The recording gauge was of the tipping bucket type. This type of gauge collects the rainfall in a twelve inch funnel.[6] The funnel discharges the water through an opening above a pair of buckets mounted on a pivot bar. When 0.01 inches of precipitation is collected in one bucket, it tips on the pivot emptying the water into a storage can and also bringing the other bucket below the point of discharge. This process is repeated as the rain continues. The tipping of the bucket also energizes an electrical circuit which results in a pen marking a notch on a strip-recorder chart which is clock-driven. The water in the storage reservoir is drawn off and measured as is done with a standard gauge. The recording gauge gives the intensity or rate of rainfall as well as the total rainfall.

The standard gauge is generally considered to be more accurate than the recording gauge. Consequently the standard gauge reading of total rainfall was checked after each storm against the total rainfall recorded on the recording gauge. If the totals differed, the individual recording gauge values were adjusted by a correction factor based on the standard gauge measurement.

Both the gauges were obtained through the cooperation of the Environment Canada, Atmosphere Environment Service, Government of Canada and the Waterworks and Waste Disposal Branch of the former Metropolitan Corporation of Greater Winnipeg. The gauges were read from June 1 to September 30, 1971 inclusively.

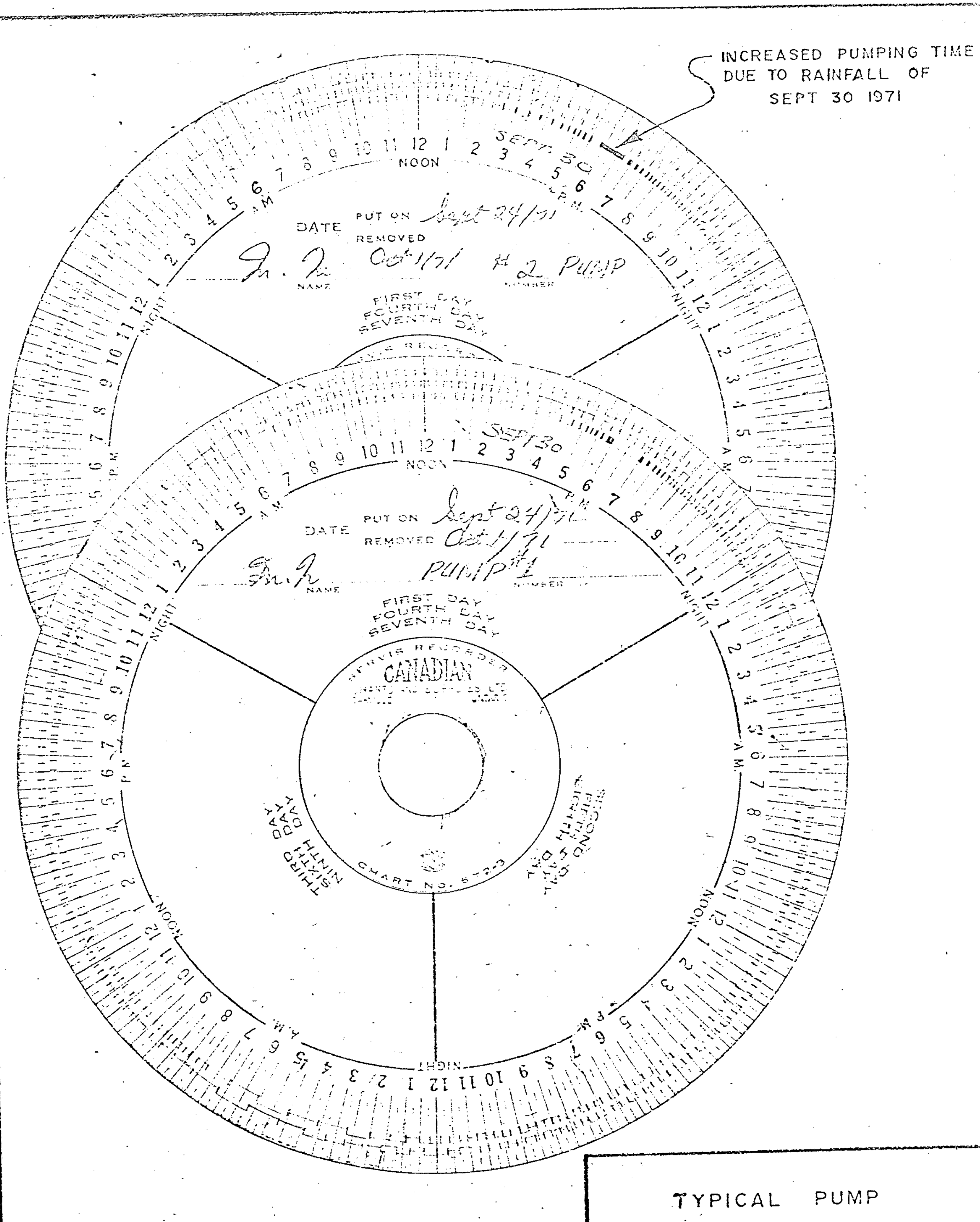
D. Sewage Flow Monitoring

The entire sanitary sewage flow generated in the Pulberry sewer district is conveyed to a pumping station at St. Vital Road and Pulberry

Street. This station has two pumps, normally in single operation rather than in parallel. The pumps lift the sewage via a short discharge line to a branch sewer where flow is by gravity, eventually to treatment. The operating time of the pumps was recorded by continuously monitoring power consumption of the drive motors. This was done by using existing power recorders at the station. These recorders have a clock-driven circular time chart and, anytime a pump unit is drawing current, a stylis pen is activated to mark a line on the chart. The marking process is continuous throughout the interval that the pump circuit is operating. A copy of a chart is shown in Figure 3.

In this way, the interval and the actual chronological time of pumping is readily obtained. When the pump is not operating, the chart advances but no line is marked. Thus a continuous record of pumping operation was kept. The charts were changed weekly.

Having obtained the chronological times of pumping and pumping intervals in the above manner, the pump performance curves and the piping system head curves were used to calculate sewage flows. To enlarge on this method, the pumps in the station are Chicago Pump Company, type VOS-L56, eight inch by six inch centrifugal pumps with eleven inch impellers and rated at a speed of 1150 revolutions per minute. Performance curves were obtained on these pumps.[7] The curves define the total head delivered by this pump at any given capacity. Information on the elevations of the incoming sewage flow and the free water level in the discharge sewer was obtained.[8] This provided the static head that the pump was required to deliver. The friction losses in the piping system were calculated using the Hazen-Williams friction formula and are shown in Table 1. From these data, the system head curve was plotted showing



TYPICAL PUMP MONITORING CHARTS

(PULBERRY DISTRICT)

FIG. 3

TABLE 1

SYSTEM HEAD CURVE CALCULATIONS

Static Lift

Elevation at point of discharge	-	753.50
Elevation of free water surface in wet well	-	733.50
Static Lift	=	20.0 feet

Equivalent Piping

Note: All piping is cast-iron and losses are calculated according to the Hazen-Williams formula with a "C" value of 100.

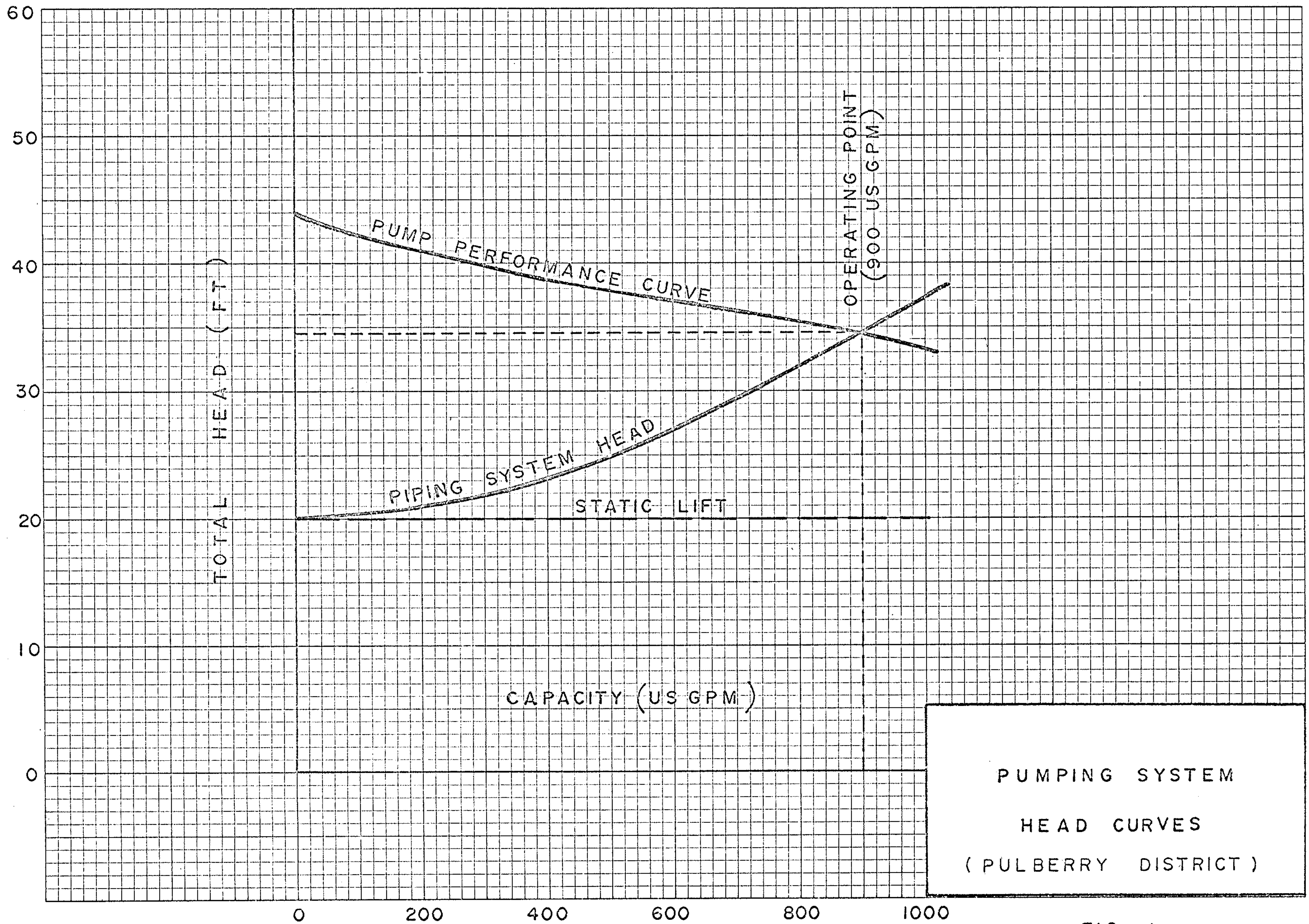
Pipe or Fitting	Equivalent Length of 8"ø Pipe
2 - 6"ø Gate Valves	46 Feet
1 - 6"ø Check Valves	160
2 - 90° x 6"ø Elbows	140
24' - 6"ø Pipe	96
1 - 90° x 8"ø Elbow	22
1 - 45° x 8"ø Elbow	10
115' - 8"ø Pipe	115
Total Equivalent Length	589 Feet

Hydraulic Losses

Flow (USgpm)	Unit Friction Loss ('/100')	Friction Head (')	Static Head (')	Total Head (')
400	0.54	3.2	20.0	23.2
600	1.14	6.7	20.0	26.7
800	1.97	11.6	20.0	31.6
1000	2.97	17.5	20.0	37.5

the total head requirements of the installed piping system at varying capacities. The combination of this curve with the pump performance curve indicates the actual pumping rate as shown in Figure 4.

Knowledge of the pumping rate and the chronological interval of pumping allows the calculation of the flow rate and volume pumped in a given time interval from which the hydrograph of sewage flow can be produced. Table 2 shows a typical calculation for a daily hydrograph of dry weather flow. The actual pumping times were measured on several occasions and compared to the chart readings to improve interpretation of pump running times.



PUMPING SYSTEM

HEAD CURVES

(PULBERRY DISTRICT)

FIG. 4

TABLE 2

CALCULATION OF SEWAGE FLOW RATES

Date - September 28, 1971

Time Interval	Pump #1	Pump #2	Total	Sewage Flows* (U.S. gal. per hr.)
12 - 1	4.5	4.5	9.0	8100
1 - 2	3.0	1.5	4.5	4050
2 - 3	1.5	3.0	4.5	4050
3 - 4	1.5	1.5	3.0	2700
4 - 5	1.5	1.5	3.0	2700
5 - 6	1.5	1.5	3.0	2700
6 - 7	1.5	1.5	3.0	2700
7 - 8	4.5	4.5	9.0	8100
8 - 9	7.5	7.5	15.0	13500
9 - 10	6.0	6.0	12.0	10800
10 - 11	6.0	6.0	12.0	10800
11 - 12	4.5	4.5	9.0	8100
12 - 1	6.0	6.0	12.0	10800
1 - 2	4.5	4.5	9.0	8100
2 - 3	4.5	4.5	9.0	8100
3 - 4	4.5	4.5	9.0	8100
4 - 5	3.0	3.0	6.0	5400
5 - 6	4.5	4.5	9.0	8100
6 - 7	6.0	4.5	10.5	9450
7 - 8	6.0	7.5	13.5	12150
8 - 9	7.5	6.0	13.5	12150
9 - 10	4.5	6.0	10.5	9450
10 - 11	6.0	6.0	12.0	10800
11 - 12	6.0	4.5	10.5	9450
			Average	= 7930 US gphr

* Flow = Pumping Time x Flow Rate = Time (min/hr.) x 900 USgpm
= US gphr.

CHAPTER 5

DATA ANALYSIS

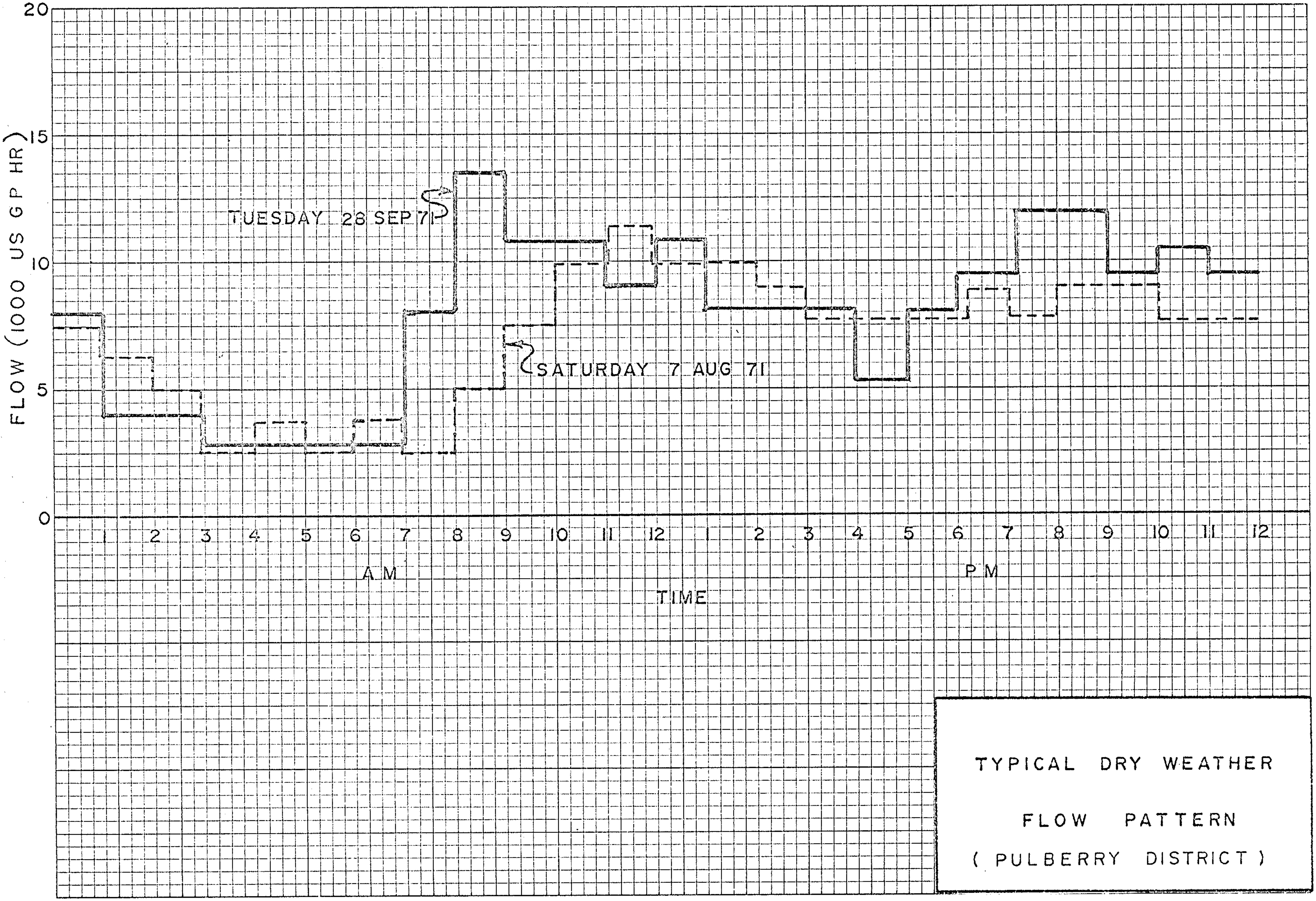
A. Dry Weather Flow Conditions

The dry weather flow pattern must be established to enable measurement of the effect of rainwater intrusion into the sewers. In waterworks and sewage works design it is common to base peak factors on the average annual flow, as was discussed in Chapter 3. Therefore the annual average daily flow was required even though the daily flow hydrographs for summer were a prime interest for this study.

1. Diurnal Flow Pattern for Dry Weather Conditions

The daily sewage flow hydrographs were calculated for winter conditions prior to snow-melt and for summer conditions without rain. The daily flow pattern was calculated on an hourly basis similar to Table 2. From these calculations, flow hydrographs for that particular day were developed. It was found that the daily flow pattern was remarkably similar for Monday to Friday during either the summer or winter season. There was a distinct difference in the flow pattern for a week-day as opposed to a week-end day. A typical diurnal flow pattern for a week-day and a week-end are shown in Figure 5.

Both curves show a period of minimum flow in the 3:00 - 6:00 A.M. range. The week-day pattern consistently shows the maximum flow occurring



TYPICAL DRY WEATHER

FLOW PATTERN

(PULBERRY DISTRICT)

FIG. 5

in the 7:00 - 9:00 A.M. range whereas the week-end daily pattern typically shows a maximum flow in the 10:00 - 12:00 A.M. range. The week-day pattern usually also showed a lesser peak in the 7:00 - 9:00 P.M. time range which was not always evident in the week-end pattern. These flow patterns are very uniform, with the week-day morning peak being particularly consistent and pronounced.

This illustrates the uniformity of suburban living. The week-end shows the effect of most people not being required to report to work on Saturday and Sunday and many choosing to start their daily activities at a somewhat later time. As a result, the peaks become more diffused, even though the total flow is not significantly changed.

The diurnal flow pattern is particularly important to this study because it is used as a base for measuring incremental flow in the sewer due to rainwater intrusion, which may occur at any time in the day. Table 3 illustrates the numerical values of some of the pertinent parameters in the days used for determining dry weather flow patterns. The consistency of the flow pattern, with only minor variations in the magnitudes of maximum and minimum values, indicates that a generalized flow pattern can be used for comparison with wet weather flows.

2. Average Sewage Flows

The annual average sewage flow is the most common base for maximum or minimum design factors in sewer design. The average annual flow was calculated from observations as shown in Table 3. The average dry weather flow found was 5535 US gphr. in winter and 7820 US gphr. in summer. It has been found in this area that the annual water consumption is similarly

TABLE 3

DRY WEATHER FLOW CHARACTERISTICS

Date	Minimum Flow (US gphr)	Maximum Flow (US gphr)	Average Flow (US gphr)
Dec. 8, 1970	1300	11,700	6200
15	1300	10,200	5850
16	1300	12,700	5850
Jan. 9, 1971	1300	9,000	5070
26	1300	7,600	5200
Feb. 13, 1971	1300	7,600	4600
16	1300	10,200	6270
Mar. 9, 1971	1300	7,600	4900
16	1300	9,000	5850
	Winter Average - 5535 US gphr		
June 4, 1971	2600	12,800	8450
7	2600	10,400	8900
July 5, 1971	3900	14,200	9300
Aug. 3, 1971	2600	10,200	7400
7	2600	11,500	7250
24	2600	10,200	7150
31	2600	11,800	7550
Sept. 9, 1971	1300	11,800	7550
28	2600	11,800	7930
	Summer Average - 7820 US gphr		

characterized by these two distinct seasons. The summer season with respect to water use is considered five months long and the winter season seven months long.[1] Accordingly, the annual average dry weather flow (DWF) for the test area was calculated as follows:

$$\text{Average annual D.W.F.} = \frac{(\text{winter DWF x 7 months}) + (\text{summer DWF x 5 months})}{12 \text{ months}}$$

$$= \frac{(5535 \times 7) + (7820 \times 5)}{12}$$

$$= 6500 \text{ US gphr}$$

To determine if this average sewage flow was reasonable, a calculation of the theoretical sewage flow from this area was made. This was done by taking the average water consumption and making suitable deductions for water use that will not contribute to sewage flow and an allowance for ground water infiltration. The average per-capita water consumption in the Municipality of St. Vital was fifty-four U.S. gallons per day in 1971.[2] This should apply very closely to the test area since the entire municipality is residential in character. The domestic sewage can be estimated from the water consumption by deducting for water not reaching the sanitary sewers, such as lawn sprinkling and unaccounted-for-water. Unaccounted-for-water is a term in waterworks used to describe an allowance for water meter inaccuracies, fire fighting water use, leakage in water mains, street cleaning and hydrant flushing. The amount allowed for these losses were taken from actual design data for the general area and are 2.4 U.S. gallons per capita per day (g.p.c.d.) for lawn sprinkling and 10.8 U.S. gallons per capita per day for unaccounted-for-losses.[3]

Using population estimates as given in Chapter 4, the amount of residential or domestic sewage becomes; population x (per capita use - lawn sprinkling - unaccounted-for-losses) or 2760 people (54 g.p.c.d. - 2.4 g.p.c.d. - 10.8 g.p.c.d.) = 2760 x 40.8 = 112,600 US gal/day = 4685 US gphr. To this must be added an allowance for ground-water infiltration which again was taken from actual design and taken as 240 US gallons per acre per day.[4] Infiltration is then 120 acres x $\frac{240}{24}$ g.p.c.d. = 1200 US gphr. The total sewage flow expected from the test area is then 4685 + 1200 = 5885 US gphr. The observed flow was 6500 US gphr. or about ten percent higher. From the daily flow pattern it can be seen that the minimum flow, which should consist almost entirely of infiltration, was always higher than the theoretical allowance of 1200 US gphr. (See Table 3). Considering this factor, the computed theoretical sewage flow is very close to the observed average flow and confirms that the observed value can be used for a base for further evaluations.

3. Peak Dry Weather Flow Factors

The peak hourly dry weather flow observed was 14,200 US gphr which is a peak factor of 2.15, that is, 2.15 x the average annual sewage flow. Peak factors found for various days ranged from 1.2 to 2.15. This is similar to peak factors observed for service population of this size in the Winnipeg area.[5]

4. Infiltration of Ground Water

The actual infiltration of ground water occuring in the system can be taken as the minimum flow observed in the 3:00 A.M. to 6:00 A.M. time range. At this time, the sewage produced from a residential community is negligible and, during dry weather, the measured flow must be that of ground water infiltration. This flow is not to be confused with the inflow of rain water. The observed infiltration ranged from 1300 to 3900 US gphr as shown in Table 3. The infiltration during the winter was significantly less than during the summer namely, an average of 1300 compared to a summer average of 2900 US gphr. The average infiltration over the year was calculated to be 1970 US gphr.

B. Wet Weather Flow Conditions

The flow patterns under dry weather conditions have been presented. The conditions existing under rainfall conditions will now be examined.

1. Rainfall

There was a total of forty-six occurrences of rainfall in the Pulberry area in the June to October interval. This includes all rainfall in excess of 0.01 inches. The total rainfall during this period was 10.15 inches compared to twenty-four year average of forty-three periods of rainfall and an average total of 10.84 inches during this interval.[6,7] For the purposes of this study, rainfalls of less than 0.04 inches were ignored since it was considered that these amounts were insignificant in terms of the problem under investigation. This was verified in the sewage flow monitoring. With this consideration a total of thirty rainstorms were studied in detail in the study area. Each storm was identified as to intensity and duration and then classified as to its frequency of occurrence or, as it is often called, the average return period of a storm of this magnitude or more. This classification was done using data recorded by Environment Canada for this area over an eleven year period of record, as shown in Table 4. Table 5 illustrates the characteristics of the various storms and the frequency classification.

TABLE 4

PROBABLE RAINFALL FREQUENCIES

Number of Days Per Year of 11 Year Record With a Rainfall Equal to Or Greater Than the Specified Value in the Specified Duration

Rainfall (in.)	5 Min.	10 Min.	15 Min.	30 Min.	60 Min.	2 Hrs.	6 Hrs.	12 Hrs.
0.02	26.1	31.0	36.4	37.4	41.5	42.4	42.8	44.3
.04	14.6	20.5	25.6	28.7	34.7	36.2	36.5	37.9
.06	10.6	14.5	19.2	23.1	29.2	31.3	32.5	34.5
.08	8.3	11.5	14.3	19.5	25.2	27.5	29.0	31.2
.10	6.8	9.7	11.8	16.1	21.6	24.5	26.9	28.9
.12	5.0	8.5	10.4	14.4	19.2	22.1	24.5	26.8
.14	4.2	7.1	9.4	11.9	16.3	19.8	22.5	24.7
.16	3.8	5.9	7.6	10.1	14.8	17.7	20.9	23.1
.18	3.5	5.0	6.45	9.0	13.2	16.0	18.5	20.9
.20	2.9	4.4	5.6	7.8	11.2	14.2	17.4	19.0
.22	1.9	3.8	4.9	6.5	10.0	12.6	16.3	18.1
.24	1.5	3.2	3.9	5.6	8.5	11.6	15.2	16.7
.26	1.1	3.1	3.5	4.7	7.5	10.6	14.5	16.0
.28	0.9	2.6	3.2	4.1	6.5	9.8	13.4	14.7
.30	.8	2.3	3.0	3.7	5.6	9.2	12.6	13.7
.32	.8	2.1	2.7	3.4	5.2	8.4	12.0	13.2
.34	.8	1.6	2.5	3.0	4.5	7.1	11.2	12.6
.36	.5	1.4	2.1	2.7	4.2	6.3	10.2	11.8
.38	.5	1.4	1.9	2.7	4.1	6.0	9.7	11.3
.40	.2	1.1	1.5	2.5	3.9	5.8	9.3	10.8
.42	.1	1.0	1.5	2.4	3.5	5.4	8.2	10.2
.44	.1	0.9	1.4	1.9	3.1	4.8	7.5	9.2
.46	.0	0.7	1.1	1.8	2.7	4.1	7.1	8.2
.48		0.6	1.0	1.7	2.7	3.9	7.0	8.0
.50		0.5	1.0	1.5	2.3	3.5	6.4	7.6
.52		.5	.9	1.4	2.1	3.3	6.0	7.1
.54		.5	.9	1.0	2.1	3.0	5.5	6.7
.56		.5	.9	1.0	2.0	2.7	5.3	6.3
.58		.3	.8	.9	1.9	2.3	5.0	6.0
.60		.3	.7	.9	1.8	2.2	4.9	5.7
.62		.1	.7	.9	1.6	2.0	4.5	5.4
.64		.1	.7	.9	1.5	1.8	4.1	4.8
.66		.1	.6	.9	1.4	1.7	3.6	4.3
.68		.1	.5	.9	1.4	1.7	3.5	4.2
.70		.1	.4	.7	1.4	1.5	3.1	3.6
.72		.1	.4	.7	1.4	1.5	3.0	3.5
.74		.1	.3	.6	1.3	1.4	3.0	3.5
.76		.1	.2	.6	1.1	1.4	2.9	3.3
.78		.1	.1	.5	1.0	1.3	2.6	3.1
.80		.1	.1	.5	1.0	1.2	2.5	3.1
.82		.1	.1	.5	.9	1.1	2.3	3.0
.84		.1	.1	.5	.8	1.1	2.2	2.9

TABLE 5

RAIN GAUGE DATA

Date		Time Rain-fall Began	Greatest Rainfall In Interval (in.)								Duration (hr.)	Probable Occurrence (Times per Year)
			Minutes				Hours					
			5	10	15	30	1	2	3	4		
June	4	1545	0.04								0.5	28.7
	5	0155	0.13	0.20	0.30	0.31	0.32	0.50			2.0	3.5
	6	1535	0.05	0.10	0.10	0.10	0.10	0.15	0.16	0.20	4.0	16.0
	10	0315	0.03	0.05	0.07	0.15	0.22	0.25			2.0	10.0
	11	2115	0.02	0.04	0.06	0.07	0.10	0.15			2.0	19.0
	11	1450	0.13	0.18	0.19	0.21					1.0	10.0
	15	2345	0.03	0.04	0.04	0.05	0.05	0.05	0.10		3.0	25.0
	19	1015	0.02	0.04	0.05	0.10	0.14	0.23	0.29		3.0	13.0
	19	1535	0.11	0.23	0.29	0.33					0.5	3.0
	26	0325	0.02	0.04	0.05	0.11	0.22	0.27			2.0	10.0
	30	0935	0.03	0.05	0.06	0.06					0.5	23.0
July	3	0610	0.18	0.28	0.29	0.33					0.5	3.2
	3	1820	0.14	0.23	0.26	0.26					0.5	4.7
	7	0020	0.15	0.22	0.25	0.27	0.39	0.66	0.34		3.0	1.5
	10	0010	0.02	0.04	0.07	0.09	0.16	0.21	0.28	0.35	4.0	8.7
	15	1950	0.02	0.04	0.05	0.07					0.5	21.0
	17	2020	0.03	0.03	0.04	0.05	0.07	0.11	0.14		3.0	20.4
	19	0750	0.03	0.03	0.03	0.04	0.08	0.10	0.13		3.0	21.8
	24	0205	0.03	0.06								23.0
	25	0515	0.02	0.03	0.04	0.08	0.13	0.21	0.29		3.0	10.3
	27	0535	0.01	0.02	0.02	0.03	0.04	0.07	0.12	0.14	4.0	21.0
	28	1235	0.03	0.03	0.05	0.09					0.5	17.8
	28	1915	0.05	0.09	0.12	0.15	0.25	0.34	0.39		3.0	6.8
	31	1710	0.02	0.03	0.06	0.03					1.0	21.6
Aug.	16	2025	0.02	0.03	0.05	0.09	0.17	0.27	0.38		3.0	8.7
	19	0530	0.02	0.03	0.03	0.05	0.10	0.12	0.15		3.0	19.6
Sept.	1	1125	0.05	0.08	0.09						0.05	16.0
	5	0530	0.22	0.43	0.64	0.75	0.85	0.90			2.0	0.8
	26	1940	0.03	0.04	0.04	0.06	0.08				1.0	25.2
	30	1430	0.09	0.15	0.17	0.25	0.45	0.79	0.87	0.96	4.0	1.2

Table 6 compares the number of occurrences of rainfall in 1971 to the twenty-four year average.

TABLE 6

OCCURRENCES OF RAINFALL IN 1971

(considering all rainfalls in excess of 0.01 inches)

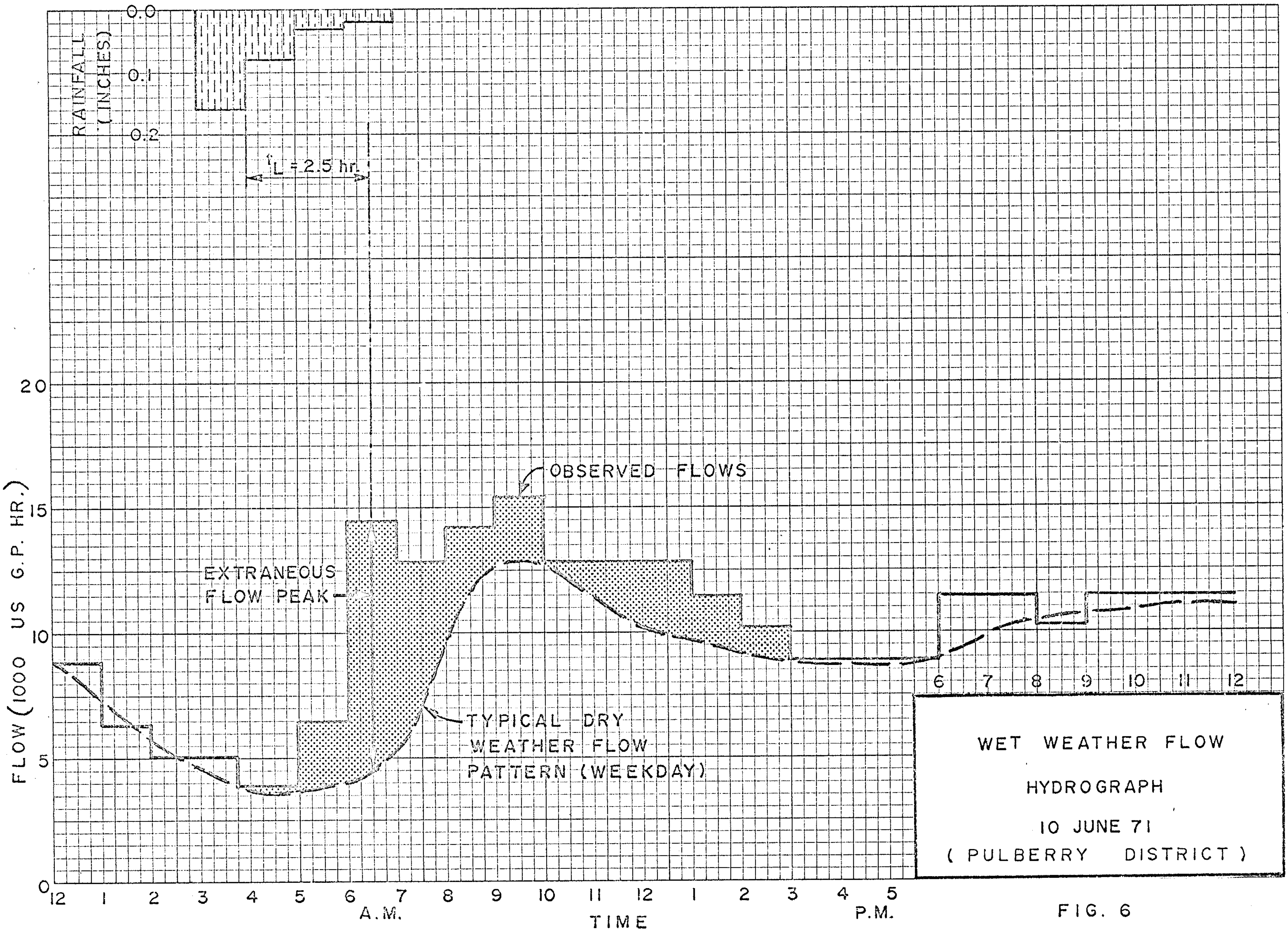
Month	1971	Normal
June	14	12
July	17	11
August	5	10
September	10	10
TOTAL	46	43

The months of June and July provided considerable data with eleven and thirteen significant rainstorms, respectively. By comparison, the months of August and September only produced two and four significant rainstorms, respectively. The two most severe storms experienced in the study area however, occurred in September.

While rain fell a little more often in June and July and less often in August than normal, the overall impression is that 1971 is fairly representative of the normal in terms of total occurrences and amount of rainfall.

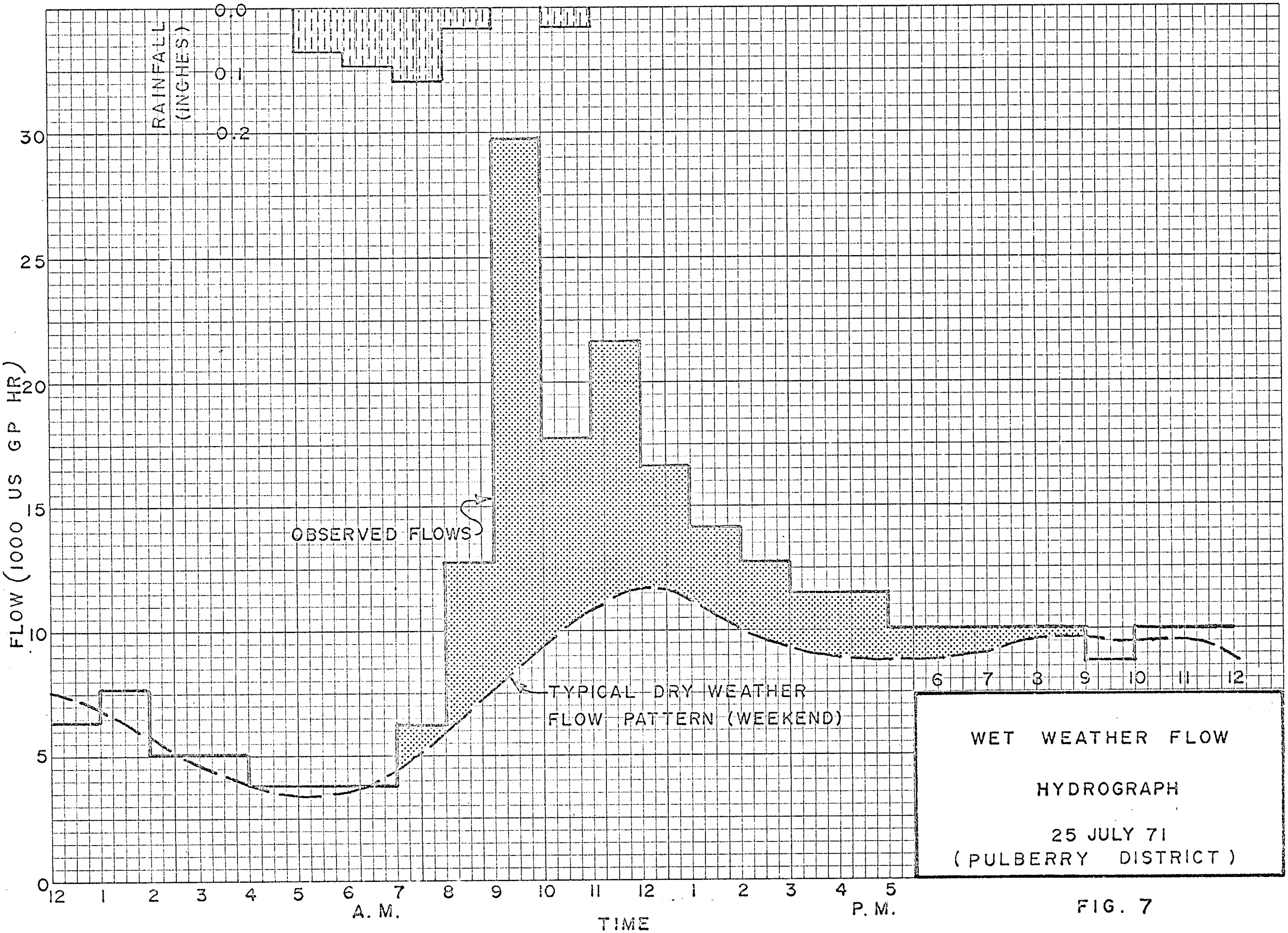
2. Wet Weather Hydrographs

Hydrographs for each of the days on which significant rainfall fell were plotted. Typical examples are shown on Figure 6 and 7. Figure 7 illustrates the rainstorm of July 25, when a total of 0.34 inches of rain fell in a period of six hours. The typical dry weather flow pattern is shown super-imposed on the actual hydrograph obtained, showing clearly



WET WEATHER FLOW HYDROGRAPH
10 JUNE 71
(PULBERRY DISTRICT)

FIG. 6



WET WEATHER FLOW

HYDROGRAPH

25 JULY 71
(PULBERRY DISTRICT)

FIG. 7

the dramatic response of the flow in the sanitary sewer to the rainfall. The magnitude and duration of the reaction leave no doubt that this particular system was receiving very significant extraneous flow in the sanitary sewer. A glance at the hydrograph and the incremental flow due to rainfall is sufficient to appreciate why sanitary sewers have in many cases proved incapable of discharging these inflated rates of flow and have instead flooded basements.

A summary of the characteristics of each of the wet hydrographs is given in Table 7. These characteristics will be discussed in greater detail.

(a) Extraneous Flow Peak Factors

The key factor in determining the capacity of the sewers is the peak flow. As explained earlier, it is common practice to relate maximum flows in a sewer to the average annual dry weather flow by a peak factor. A similar approach was taken with the extraneous flow present in the sewer. The flow in the sewer judged to be due to extraneous flow or incremental to the normal dry weather pattern was divided by the average annual dry weather flow to obtain an extraneous flow peak factor (E.F.P.F.). For the July 25, rainstorms, as shown on Figure 6, the peak extraneous flow occurred at 9:00 - 10:00 A.M. and was 21,500 gphr resulting in an E.F.P.F. of $\frac{21,300}{6500} = 3.28$. This factor thus describes the extraneous flow measured over and above the normal dry weather flow in terms of multiples of the average annual dry weather flow. This extraneous flow peak factor can readily be compared to the design peak factor since they are calculated on the same base. It is important to recognize that this E.F.P.F. describes flow in addition to the dry weather flow occurring at a given time.

TABLE 7

CHARACTERISTICS OF WET WEATHER HYDROGRAPHS

Date			Rainfall Frequency (Probable Occurrence Per Year)	Peak Extraneous Flow Factor	Peak Extraneous Flow (US gphr)	Time of Peak Extraneous Flow	Time Lag. t_L (hrs)	Volume of Extraneous Flow (US gal.)
Fri.	June	4	28.7	0	0	0	0	0
Sat.		5	3.5	2.23	14,500	5-6 AM	2.5	45,500
		5	16.0	0.85	5,500	8-9 PM	2.6	39,500
Thurs.		10	10.0	1.53	10,000	6-7 AM	2.5	35,000
Fri.		11	19.0	1.16	7,500	9-10 AM	2.0	20,000
		11	10.0	1.01	6,500	5-6 PM	1.5	20,000
Wed.		16	25.0	0.54	3,500	5-6 AM	3.7	13,000
Sat.		19	13.0	3.10	20,000	2-3 PM	2.5	54,000
		19	3.0	2.15	14,000	5-6 PM	2.0	54,000
Sat.		26	10.0	1.62	10,500	6-7 AM	2.0	27,000
Wed.		30	23.0	0.54	3,500	1-2 PM	2.5	13,000
Sat.	July	3	3.2	1.62	10,500	8-9 AM	2.0	35,000
		3	4.7	1.30	8,500	9-10 PM	2.5	20,000
Wed.		7	1.5	7.10	46,000	4-5 AM	2.7	118,000
Sat.		10	8.7	1.78	11,500	6-7 AM	4.2	50,000
Thurs.		15	21.0	0.85	5,500	11-12 PM	3.8	10,000
Sat.		17	20.0	0.47	3,000	11-12 PM	2.5	10,000
Mon.		19	21.8	0.87	5,500	1-2 PM	4.5	14,000
Sat.		24	23.0	0.54	3,500	5-6 AM	3.0	15,000
Sun.		25	10.3	3.28	21,300	9-10 AM	2.5	67,000
Tues.		27	21.0	0.47	3,000	10-11 AM	2.8	8,000
Wed.		28	17.8	0.23	1,500	3-4 PM	2.0	3,000
		28	6.8	1.62	10,500	10-11 PM	2.0	37,000
Sat.		31	21.6	0.47	3,000	11-12 AM	2.0	9,000
Mon.	Aug.	16	8.7	0.47	3,000	1-2 AM	3.5	10,000
Thurs.		19	19.6	0.47	3,000	10-11 AM	4.0	9,000
Wed.	Sept.	1	16.0	0.39	2,500	1-2 PM	2.0	8,000
Sun.		5	0.8	5.70	37,000	6-7 AM	1.0	81,000
Sun.		26	25.2	0.65	42,000	10-11 PM	2.0	12,000
Thurs.		30	1.2	5.80	37,600	6-7 PM	2.0	105,000

The extraneous flow peak factors observed ranged from 0 to 7.1. It is interesting to note that the extraneous flow coincided with the peak flow period of the normal dry weather pattern on only about twenty-five percent of the occasions. (See Table 7) In fact, only on fifty percent of the occasions did the extraneous flow occur at times when the dry weather flow at that particular time was in excess of the average flow. This is important when considering the probability of extraneous flow and dry weather peak flows being coincident.

The maximum observed extraneous flow rate was 46,000 gphr on July 7. If it is assumed for the moment that all this extraneous flow originated from the weeping tile around basement foundations, the peak flow per house would be $\frac{46{,}000 \text{ gal/hr.}}{700 \text{ houses} \times 60 \text{ min./hr.}} = 1.1$ gpm. On July 7, the maximum rain in a one hour period was 0.4 inches. It is widely thought that much of the weeping tile flow comes indirectly from roof drainage. On a 1200 square feet roof area, this rainfall would have produced a flow of about 5.0 gpm meaning that an average of twenty percent of the roof water was finding its way into the sewer. Around many homes, the lot grading is such that a much larger area than the roof surface may be contributing flow to the weeping tiles. However, assuming that all the extraneous flow comes solely from weeping tiles, the average peak flow from an individual house is not a large flow rate but yet can account for a significant portion of the rainfall on the total roofed area.

The maximum extraneous flow of 46,000 gphr. or 1.7 cfs. on July 7 is about nine percent of the total estimated storm water runoff of 19.2 cfs from the entire drainage area from this storm. The total runoff was calculated on the basis of the rational formula $Q = CiA$ where Q = runoff (cfs), i = intensity of rainfall (in./hr.), and A = area (acres).[8] The intensity was taken as 0.4 inches per hour.

(b) <u>Time Lag Between Peak Extraneous Flow and Rainfall</u>

An attempt was made to evaluate the time lag between rainfall and the peak flow of rainwater in the sewer. The time from the centre of mass or centroid of the actual rainfall to the time of peak extraneous flow, termed "t_L", was calculated for each wet weather hydrograph. See Table 7.

The time lag, t_L, was found to be remarkably consistent. Values ranged from 1.0 to 4.2 hours with an average of 2.6 hours. There were only six occasions when t_L was outside the two to three hour range and on most of these occasions the rainfall was an extended light drizzle.

The time of travel for a particle of sewage from Triton Bay and River Road, an extremity of the sewer system, to the Pulberry pumping station, the point of flow measurement, was calculated to be about forty-five minutes. The calculation was based on the distance of travel and an average calculated velocity in the sewer of about two feet per second. The average t_L of 2.6 hours compared to a transit time in the sewer of a maximum of 0.75 hours, indicates the inflow of stormwater is a slow phenomenon in this system. It suggests that the bulk of the extraneous flow is not likely to originate from direct connection of rainwater into the sewer, such as connection of street catch basins into the sanitary sewer.

This type of direct connection would yield a much swifter response in the sewage flow. Instead, the implication of the long t_L is that the sources of extraneous flow are many and probably not of great individual magnitude as, for example, numerous weeping tiles all contributing nominal amounts of flow.

This t_L can be compared to the "time of concentration" used in storm sewer design. The time of concentration is defined as the time required for water to flow from the remotest part of the drainage area to the point in question.[9] The time for overland flow to reach the point of inlet into the sewer system plus the travel time is the time of concentration. The inlet time is often taken at ten to fifteen minutes for storm sewer design.[10] By comparison the observed inlet time for the extraneous flow would be an average of 2.6 hours minus the travel time of 0.75 hours of 1.85 hours. The inflow of extraneous flow into a sanitary sewer should of course not be unimpeded and the preceding illustrates that the sanitary system is not suffering from a handful of illicit connections but rather a much more difficult situation of diffused sources.

(c) Volume of Extraneous Flow

The volume of extraneous flow is an important consideration in that operating costs of pumping and treatment facilities are directly influenced by volume. If relief overflow to the river occurs, the volume discharged to the stream will be a key factor in the pollution load on this body of water and in the sizing of any detention basins. The volume of extraneous flow associated with each wet weather hydrograph was calculated and the summary is shown in Table 7.

The maximum volume observed was 118,000 gallons from the rainstorm of July 7. On the basis of 700 homes in the service area, this is a contribution of about 170 gallons per house, if all extraneous flow is assumed to originate from foundation drainage. A total of 0.84 inches of rain fell on July 7. On a typical 1200 square feet roof drainage area, the total runoff from the roof would be about 640 gallons at a 100 percent runoff. The extraneous flow to the sewer might then be about twenty-five percent of the rainfall on the roofed area. Thus on an overall average basis, it is possible that the weeping tiles may be accepting a significant portion of the total rainfall. Considering an overall runoff coefficient of 0.4, the total runoff from 0.84 inches on the entire area of 120 acres would be 1,100,000 US gallons. The measured volume of extraneous flow was about ten percent of the entire runoff.

The total volume of extraneous flow measured was about 950,000 gallons. This was observed in the period of June 1 to September 30 when a total of 10.15 inches of rain fell compared to a normal of 15.4 inches over the summer.[11] To obtain the extraneous flow over the entire year, the volume of extraneous flow actually measured was adjusted by the ratio of measured rainfall to the normal rainfall for the summer as follows: $950{,}000 \text{ gallons} \times \frac{15.4}{10.15} = 1{,}500{,}000 \text{ gallons.}$

This extraneous flow can be compared to the annual dry weather flow from the area.

$$\text{Annual dry weather flow} = \text{Average flow} \times 24 \text{ hr/day} \times 365 \text{ days}$$
$$= 6500 \text{ gal/hr.} \times 24 \times 365 = 57{,}000{,}000 \text{ gals.}$$

The extraneous flow is then $\frac{1{,}500{,}000}{57{,}000{,}000}$ or about 2.7 percent of the total dry weather flow.

The extraneous flow is thus a small percentage of the total sewage generated from the area but it is the concentration of this flow into relatively short time periods that results in hydraulic problems.

(d) Quality of Extraneous Flow

It is important to realize that the extraneous flow from rainfall is of good quality upon arrival at the sanitary sewer but, once admitted into the sewer, it becomes intimately mixed with the sewage and the total flow is then polluted water. It is further significant that the pollutional strength of this flow is not diluted by the admission of large amounts of rainwater. In fact, the opposite is true. The monitoring of the Biochemical Oxygen Demand (B.O.D.) of total flows in sanitary sewers in Fort Garry and Charleswood during twelve storms in 1970 showed that average strength of the total flow during the rainfall period was eighty percent greater than the average dry weather sewage strength.[12] It was found that the pattern of B.O.D. values roughly paralleled sewage flow rate. This increase in oxygen-consuming pollutional load is very likely due to the flushing action of high flows in the sewers. This is similar to experience reported for combined sewers where pollutional load is high during the initial surge of high runoff in the combined sewers.[13] Deposition of sewage solids in the sewer occurs during periods of relatively low flows. As was explained earlier, it is common for the designer to have to compromise desirable self-cleansing velocities and low flows in order to conserve hydraulic head for the discharge of peak flows. At high flows, these depositions are dislodged and are reflected in increased B.O.D. and solids concentrations. Thus, at a time when the sewer system is most likely to require hydraulic relief to the receiving stream, the wastewater is likely to be at its maximum pollutional potential.

3. Correlation of Wet Weather Hydrographs to Rainfall

It is evident from the hydrographs of sewage flow that there exists some relationship between extraneous flow and rainfall. A statistical analysis of the data on several characteristics of the extraneous flow was made to define the nature and degree of this relationship.

(a) Extraneous Flow Peak Factors and Rainfall Frequency

The method of least squares was used to determine the curve of best fit to these data. This method is a common statistical tool for determining from a given set of data that curve that has the property of minimizing the sum of the square of the deviation of all points from this calculated curve.

The extraneous flow peak factors and rainfall frequencies, as given in Table 7, were studied in this manner. The scatter of points suggested a form of geometric curve as being most descriptive of the relationship between the two variables. This type of curve will have the generalized form $Y = aX^b$ or $\log Y = \log a + b \log X$. The latter equation, will plot as a straight line on a log-log graph paper. The constants "a" and "b" can be determined from the formulas given below.[14]

$$\log a = \frac{\Sigma \log Y \, \Sigma (\log X)^2 - \Sigma \log X \log Y \, \Sigma \log X}{n \, \Sigma (\log X)^2 - (\Sigma \log X)^2}$$

and

$$\log b \quad \frac{n \, \Sigma \log X \log Y - \Sigma \log Y \, \Sigma \log X}{n \, (\log X)^2 - (\Sigma \log X)^2} = \frac{Z}{D}$$

where n is the number of pairs of data.

If Y is used to describe the EFPF and X is the rainfall frequency, the above analysis will provide a non-linear regression curve for EFPF on rainfall frequency. The analysis can be extended to quantify the degree of relationship or correlation between these variables and the standard error of estimate.[15]

The standard error of estimate of Y or X is given by

$$S.E. \; (\log Y \log X) = \sqrt{\frac{\Sigma (\log Y - \log a + b \log X)^2}{n}}$$

This standard error of estimate is useful in that it provides insight into the expected distribution of points from a large population of data about the regression curve. For example, lines parallel to the regression line of Y on X and at a vertical distance of S.E. (log Y log X) above and below the regression line would include sixty-eight percent of all the points in a sample of data. Similarly, parallel lines vertically offset from the regression line by twice the standard error of estimate would contain ninty-five percent of the points in the samples.[16]

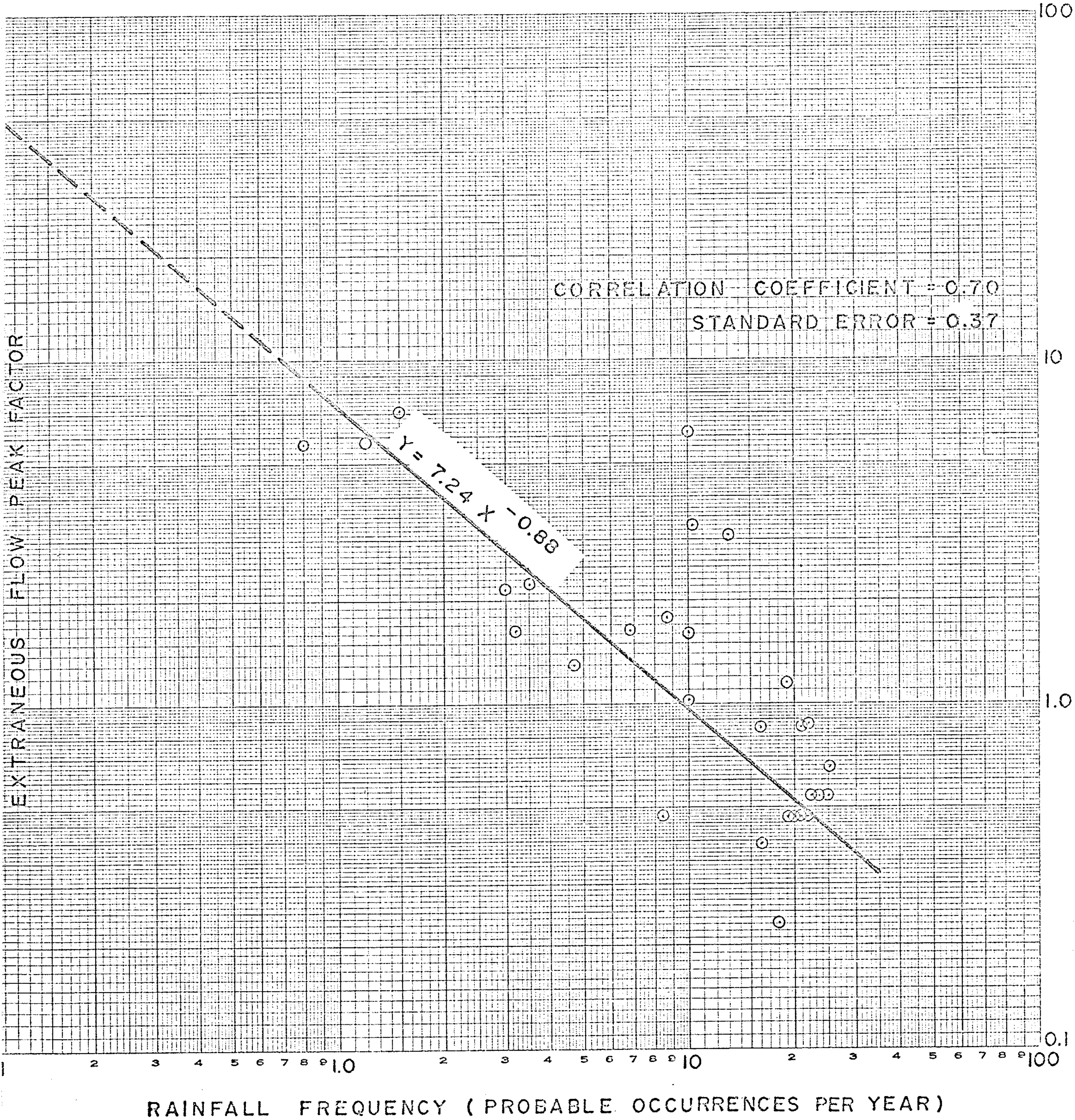
The coefficient of correlation, termed r , is calculated by the following equation.[17]

$$r(\log Y \log X) = \frac{Z}{\sqrt{D \quad (n \sum (\log Y)^2 - (\sum \log Y)^2}}$$

Z & D are given from the earlier equation. This coefficient of correlation gives the square root of the ratio of the explained variation to the total variation. The explained variation is the total deviation of Y estimates from the mean Y. The ratio r, which preferably should be close to unity, then provides a measure of how well the regression line accounts for the total variation in the data of the sample.

An Olivetti-Underwood Programma 101 computer was used to analyze the data. The plot of the peak factors and rainfall frequencies observed and the regression line determined from the statistical analysis is shown on Figure 8. The regression curve is described by the equation $Y = 7.24\ X^{-0.88}$ where Y is EFPF and X is the rainfall frequency in average occurrences per year. The standard error of estimate of the E.F.P.F. was found to be 0.37. The coefficient of correlation was 0.70 which indicates that there is significant correlation between the magnitude of peak extraneous flow and rainfall. Extraneous flow in the sewer is undoubtedly influenced by other variables such as antecedent moisture conditions, types of rainfall and storage in the sewer to name a few. In view of this, the coefficient of correlation shows a strong relationship between the EFPF and probable rainfall frequency.

A statistical analysis was also done on the set of data using a linear regression analysis in order to verify that a non-linear curve was indeed the best description for the relationships of peak extraneous flow to rainfall. The analysis was very similar to that



CORRELATION OF EXTRANEOUS FLOW PEAK FACTOR AND RAINFALL FREQUENCY (PULBERRY DISTRICT)

FIG. 8

described for the non-linear regression. It was found that the coefficent of correlation was marginally below that found for the non-linear analysis. As well, the non-linear curve intuitively is more logical since, as in most hydrological systems, it would be expected that the reaction of the sewer system to extreme rainfall would be much more pronounced than for lesser rainfalls. Accordingly, the non-linear curve was selected as most appropriate for this relationship.

The regression curve shows that high peak factors occur with relatively light rainfall. For example, a rainfall that would occur once a year is likely to result in an EFPF of 7.2. Similarly, a rainfall of a magnitude that is probable ten times a year is likely to result in an EFPF of about 1.0 or about equal to the average dry weather flow. It should be appreciated that extrapolation of the curve into the frequency range of less than once per year is risky in that data on storms of this size was not obtained. In order to observe storms of these magnitudes or greater, a test program of several years duration would be required.

The EFPF determined from the regression curve on Figure 8 for a given frequency of rainfall is additive or incremental to the dry weather flow in the sewer at the time of rainfall. For purposes of design of sewers, it is important to recall that the peak extraneous flow only coincided with an above-average dry weather flow on fifty percent of the occasions. In other words, on half the occasions the peak extraneous flow could be expected to occur when the dry weather flow was below the annual average.

It is evident that, even for a rainfall frequency of once per year, the sewer would be almost entirely designed for rainwater. For the once per year rainstorm, the EFPF is 7.2 according to Figure 8 and the hydraulic capacity of the sewer might then be designed for a total peak factor of 7.2 + 1.0 (average annual dry weather flow) = 8.2. Thus about ninety percent of the hydraulic capacity of the sewer would be assigned to extraneous flow and ten percent to the sewage flow proper. For more severe storms, such as one in five or ten year storms, the EFPF is so large that the dry weather sewage flow is insignificant in comparison. Thus, it is evident that, under rainfall conditions, the sanitary sewers are at many times operating as storm water sewers to a far greater extent than as sanitary sewers.

(b) Volume of Extraneous Flow and Rainfall Frequency

The statistical analysis for this correlation was similar to that described in the previous section. It was found that a linear regression had a far superior correlation coefficient than the non-linear curve. The linear curve had the equation $Y = -2690 X + 68,200$

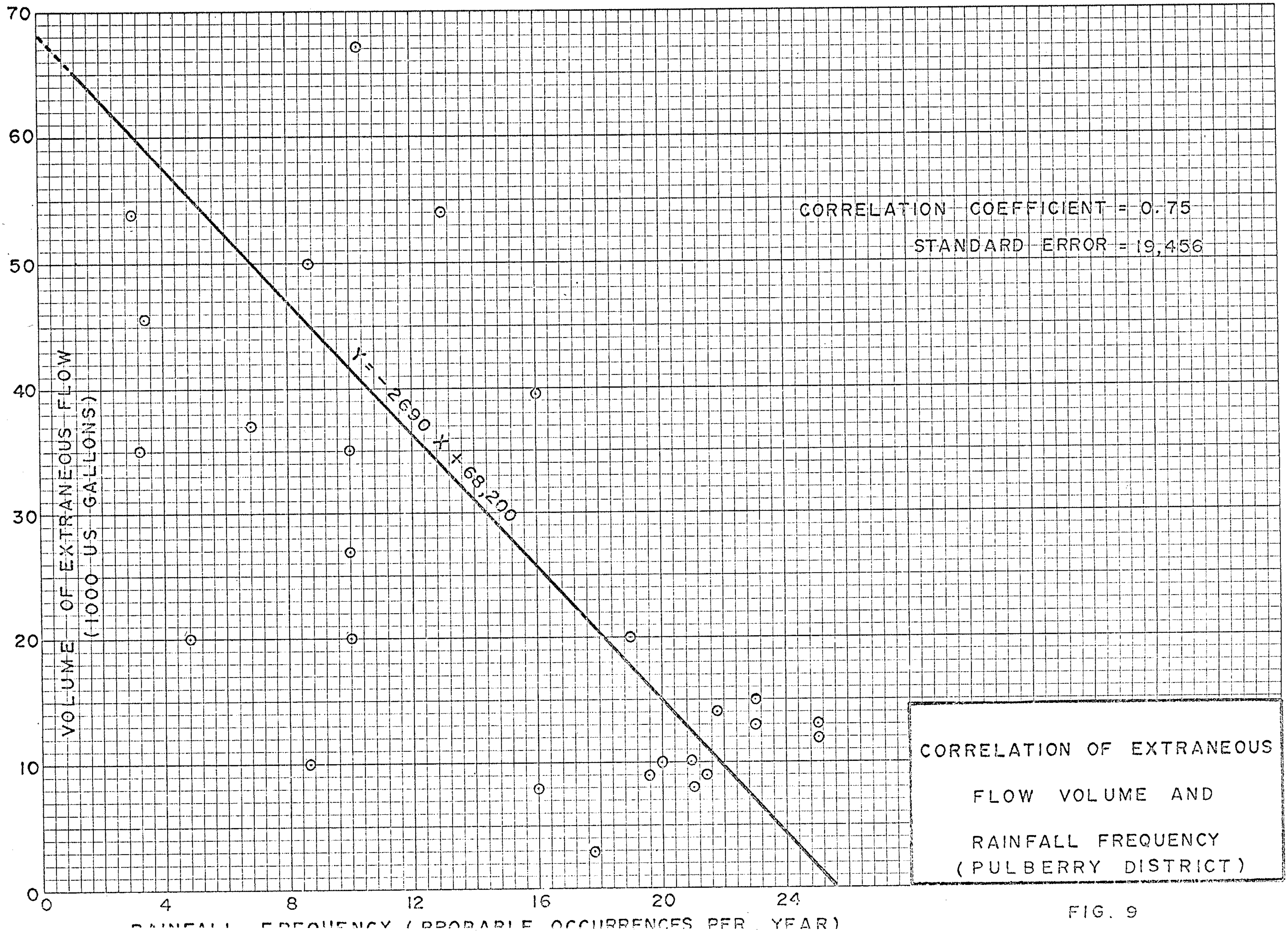
where Y = volume of extraneous flow in US gallons and

X = rainfall frequency in average occurrences per year.

This is shown on Figure 9. The correlation coefficient was 0.75 which indicates significant correlation between volume of extraneous flow and frequency of rainfall.

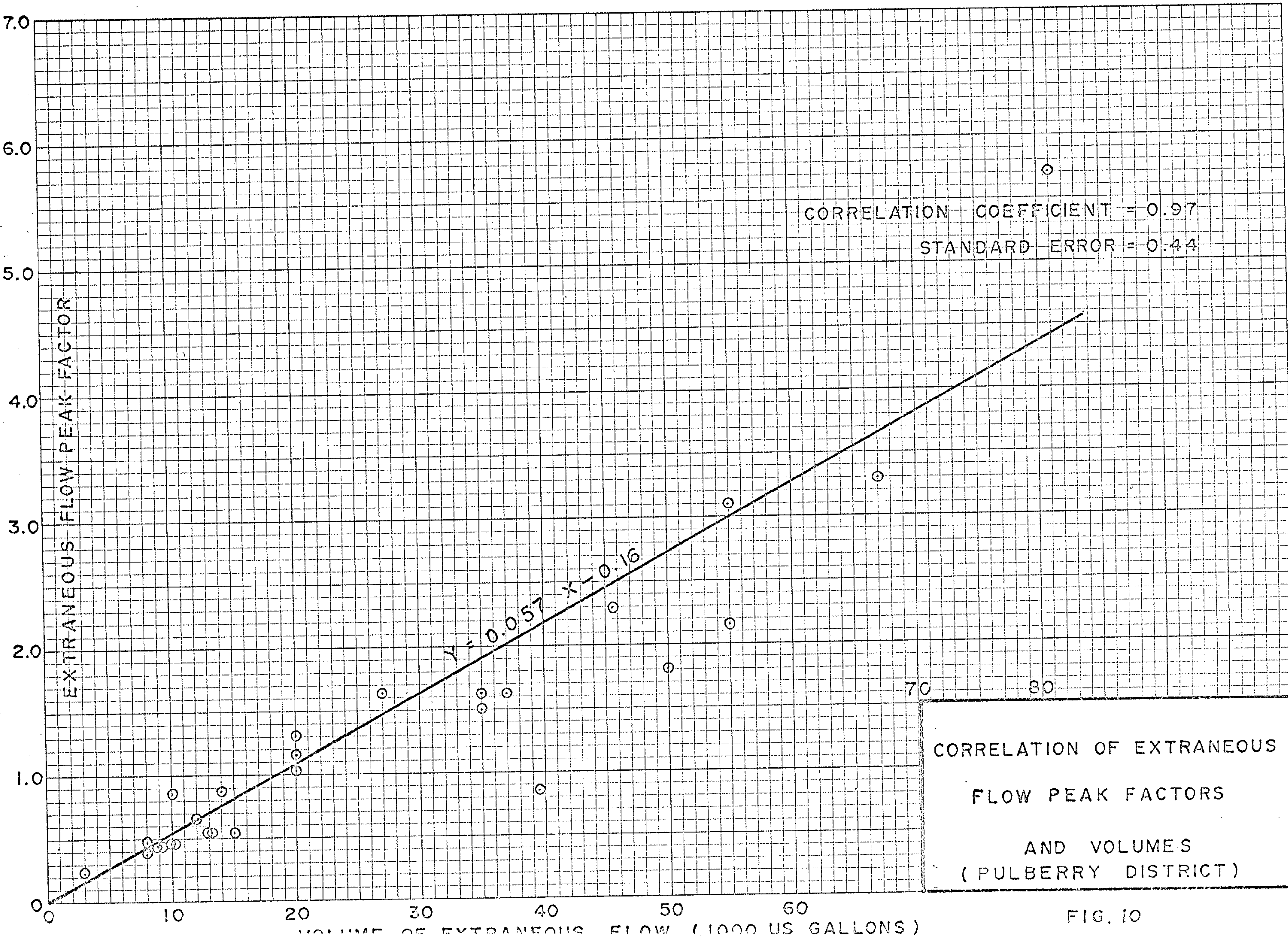
(c) Extraneous Flow Peak Factors and Volume

The peak factors and volumes of extraneous flow were subjected to regression analysis. A linear expression of $Y = 0.057 X - 0.16$, where Y is the EFPF and X is the volume of extraneous flow in 1000 US gallons, was found to provide a coefficient of correlation of 0.97, as shown on Figure 10. This indicates an unusually high degree of correlation and is probably due to two modifying factors.



CORRELATION OF EXTRANEOUS FLOW VOLUME AND RAINFALL FREQUENCY (PULBERRY DISTRICT)

FIG. 9



CORRELATION OF EXTRANEOUS
FLOW PEAK FACTORS
AND VOLUMES
(PULBERRY DISTRICT)

FIG. 10

The steepness of the rising limb of the wet weather hydrograph is influenced greatly by the time of inlet of rainwater into the sewer. The time of inlet was very uniform as evidenced by t_L times for the various rainstorms. The falling limb of the hydrograph represents water releasing from storage. Storage in a sewer is not greatly influenced by depth of flow once the depth of flow in a sewer is much deeper than mid-depth, as it would be for the higher peak factors. For this reason, storage in the system would be fairly constant. These two factors both tend to make the shape of the wet weather hydrograph similar. As a result, the peak factors relate very well to total volume of rainwater.

(d) Time Lag (t_L) and Rainfall

An analysis was made to determine if t_L, the time lag between the centre of mass of the rainfall and the peak flow, was correlated to rainfall frequency. The correlation coefficients for linear and non-linear regressions were very low, both being well below 0.5. This indicates little relationship between t_L and rainfall magnitude. This corresponds to the earlier discussion regarding Table 7 when it was noted that t_L values were very uniform. This time lag thus appears to be identified as a characteristic of the particular sewer system, or perhaps the overall inlet time for lot surface drainage, rather than to rainfall magnitude.

(e) Use of t_L for Effective Storm Duration

As outlined earlier in this chapter, the rainstorms were classified as to frequency on the basis of duration of at least thirty minutes. The t_L was found to be an average of 2.6 hours. In storm sewer design, the effective duration of a rainstorm is chosen as the time required for water to flow from the remotest part of the drainage area to the point in question.[18]

A similar approach was used in a further analysis of EFPF and rainfall frequency data on a non-linear least squares method. This time, the rainstorm frequencies on short duration storms were determined as though the actual rain fell over a duration of a minimum of 2.0 hours since the actual observed t_L was only less than 2.0 hours on two occasions. The regression line in this case had a correlation coefficient of 0.76 compared to the $r = 0.70$ found previously. The entire analysis was then repeated calculating the frequency of each storm on the basis of the rainfall that fell in the actual observed t_L for that storm. The regression line obtained had an $r = 0.75$.

It is apparent that the effective duration of the rainfall used for frequency classification is a factor in the relationship of EFPF and storm magnitude. Using an effective duration that approximates the effective total time of travel of the rainwater, including the inlet time, improves the relationship.

CHAPTER 6

COMPARISON OF RESULTS WITH PRESENT DESIGN PARAMETERS

The observed values of various significant parameters of sewage flow in a typical sanitary sewer system have now been presented as well as the accepted design practice in predicting these values. A comparison will now be made between estimated and observed results.

A. Dry Weather Flow Parameters

1. Average Domestic Sewage Flow

The calculated domestic sewage, that is, the total dry weather sewage flow minus infiltration, was calculated to be 4685 USgpm compared to an observed value of 4530 USgpm. The estimate thus compares very well with the observed value and confirms that the method of using water consumption less appropriate deductions is a suitable means of arriving at average domestic sewage flow.

2. Infiltration

The estimated calculated infiltration based on 240 US gallons per acre per day was 1200 gphr but observed infiltration ranged from 1300 to 3900 gphr with an annual average of 1970 gphr. As indicated earlier, it is generally accepted that present sewer pipe and joint materials make it possible to keep infiltration down to levels less than the design allowance but vigilant inspection of construction is required. The observed values indicate that maximum infiltration is about three times the design value which suggests that there must be leaking joints or manhole barrels. This infiltration is not contributing inordinately to the peak flows that the sewer system is required to handle. Relative to extraneous flow, the peak infiltration is not of concern with respect to capacity. None-the-less, the average infiltration is fifty percent higher than the design allowance and this increment of infiltration over the design allowance is over ten percent of the total average flow on a year-round basis. Operating costs of a treatment plant could thus be considerably higher than necessary if all tributary areas functioned similarly.

3. Peak Dry Weather Flows

The maximum dry weather flow observed had a peak factor of 2.15. The design peak factor, as estimated from the Harmon formula, is equal to $1 + \frac{14}{4 + \sqrt{P}}$ where P is the population in thousands. The design peak factor is then $1 + \frac{14}{4 + \sqrt{2.76}} = 3.38$.

The design peak factor is substantially higher than the observed conditions in the field. This has been found previously in the Winnipeg area where peak factors from sewage districts of similar size were about 2.0. In fact, the Harmon formula was shown to consistently produce factors substantially higher than recorded in the field.[1] The fact that the commonly-used Harmon formula overestimates the peak flows has undoubtedly been a factor in reducing the flooding due to extraneous flow. The capacity reserved for peak dry weather flows which have not occurred has instead been fully utilized for carrying extraneous flow.

B. Wet Weather Flow

In normal sanitary system design, little or no allowance would be made for the entry of rainwater into the sewers since, traditionally, the sanitary sewer system has been considered as being independent of rainfall. The test intervals did not include any unusually heavy rains yet, on several occasions, the sanitary sewer was actually carrying extraneous rain water at a rate of over seven times the average dry weather flow. In other-words, over eighty percent of the total flow was fouled rainwater. The rate of inflow of extraneous water on a unit area basis was found to be up to 4000 gpad. This is far in excess of the typical infiltration allowance of about 200 gpad. It is doubtful that a designer, even aware of possible rainwater intrusion and making nominal allowance for extraneous flow, would use such high allowances. It is apparent from these results that the careful design of sanitary sewers for dry weather conditions is irrelevant to the actual governing situation in the field. Dry weather flow conditions do not govern hydraulic design of the sanitary sewers subject to extraneous flow.

When reviewing the observed flow data, it is remarkable that basement flooding is not more prevalent in the test area. This is probably due to several moderating factors such as storage in the sewers, lesser peak dry weather flows than expected, and the installation of very large pumping capacity on the system. In addition, this sewer system was originally intended to service the larger area than the present district limits. Storage in the sewers occurs because an eight inch diameter sewer is commonly the minimum size employed for practical reasons even though, hydraulically, this size is not required. This, coupled with the fact that the typical designs overestimate the dry weather flow peaks, means these sewers have considerable storage available even while carrying dry weather flows. To illustrate, if the 20,000 feet of eight inch diameter sewer in the test area were to be assumed as running half full, the storage available in the remaining half of the pipe is approximately 25,000 US gallons. Some hydraulic surcharging of the sewer would be necessary for all this storage to be utilized, which is the case prior to flooding. This is a significant volume considering that more than half of the volumes per individual rainstorms were less than 20,000 US gallons. It is natural for the sewer system to store extraneous water temporarily due to the nature of the servicing layout, which must be over-designed.

The existing pump capacity in the study area is certainly greatly in excess of standard practice. The usual station design would place the pumping capacity slightly in excess of the maximum expected dry weather flow in the next five to ten years. Rather, the pumping capacity is 54,000 USgphr, or more than eight times the average DWF. This undoubtedly aids in avoiding upstream flooding with the pumps able to keep the hydraulic gradient down in the lower end of the sewer system. If the upstream sewers were to be slightly surcharged, the effective hydraulic gradient is increased and capcity can increase significantly thereby.

It was shown that up to ten percent of the total runoff from the entire area may be conveyed to the sanitary sewer. It is ironic that the storm sewer system is usually of such a large size that a ten percent increase or decrease in flow is not critical in its design yet this same amount is so large relative to the sanitary sewer dry weather flow that its addition to the pipe is intolerable. For example, the design storm runoff for the study area might be about fifty cfs. considering a four percent runoff from a one inch per hour rain. This would require a sewer in the order of fifty-four inch diameter yet the sanitary sewage is only 6500 USgpm or about 0.3 cfs and an eight inch diameter sewer provides ample capacity. The sanitary sewer, due to its relatively small capacity, is much more sensitive to fluctuations in flow.

It is worth repeating that this extraneous flow is in addition to the infiltration allowance for unavoidable joint leakage, etc. From where, then, does all this flow originate? The connection of the foundation drains is under increasing suspicion as being the prime source of extraneous flow. The nature of the reaction of flow in the sewer to rainfall suggests a large number of individual contributions dispersed throughout the system, such as weeping tiles, as the most logical source of extraneous flow. Referring again to a survey in the test area conducted in 1965, about eighty-three percent of the houses had roof downspouts discharging less than four feet from the foundation walls and fifty percent of the houses had no splash pads to direct roof runoff away from the foundation walls.[2] Moreover, about eighty percent of the lots of these houses drained towards the house due to settlement. These findings tend to confirm that foundation drains are a major source of extraneous flow and that inadequate lot drainage is a major contributing factor in directing roof runoff to the foundation.

In Oakland County, Michigan, peak flows of 2.6 to 5.2 times design flow were recorded in areas with foundation drains while parallel observations in an area without the connection of foundation drains showed a peak factor of only 1.3.[3] These tests indicated there was a definite connection between runoff from roofs which was discharged adjacent to foundations and the flow into the sanitary sewer from weeping tiles. Locally, a flow rate of about one gpm was measured during a moderate rainfall from the foundation drains of a typical modern house on a separate sanitary sewer system.[4] All indications point to the foundation drains as being the most significant source of extraneous flow.

It has been shown that the sanitary sewer is at times operating much like a combined sewer with the domestic sewage comprising only a minor portion of the total flow. The meticulous design of sanitary sewers for dry weather conditions is incompatible with respect to the total flow generated from this drainage system. It is abundantly clear that the existing practice does not adequately recognize the actual field conditions.

CHAPTER 7

METHODS OF COPING WITH THE EXTRANEOUS FLOW PROBLEM

Recognition of the fact that extraneous stormwater in the sanitary sewers is reducing the degree of property protection of the homeowner, as well as the degree of pollution control intended for the sewer system, demands that consideration be given to improved methods of coping with the problem. As is evident from the foregoing discussions, the problem is not yet completely defined. More information is required before the problem can be fully understood and rational solutions can be fully evaluated. Some of the areas of this problem requiring further testing will be discussed later. However, in spite of limited knowledge in certain aspects of the phenomena, the nature and extent of the problem has been sufficiently defined to enable some preliminary analysis of possible methods of approaching the extraneous flow problem.

There are two fundamentally different approaches that can be taken to deal with extraneous flow. In the first approach, the problem would be attacked at the source and the intrusion of stormwater into sanitary sewers would be eliminated or at least vastly reduced. The second approach would be to accept the intrusion of stormwater in sanitary sewers and to modify design practice to provide for the most economical method of handling the total flow produced in the sanitary system. Careful study is needed to determine which philosophy, or combinations of the philosophies, can best be implemented. An overview of some possibilities will be presented.

A. Elimination of Extraneous Flow at the Source

Ideally the intrusion of storm waters into the sanitary sewers should be eliminated at the source. This storm water is basically clean, unpolluted water and, as such, does not belong to a conduit intended for polluted wastewater. But can this intrusion of stormwater be eliminated in practice? Many experienced engineers feel that it cannot be eliminated completely but perhaps the inflow can be reduced significantly. Certainly a first requirement in attacking the problem sensibly is to understand and determine the chief sources of this extraneous flow. Much more needs to be known about how this stormwater finds its way into the sanitary sewer. However, the weight of evidence both locally and elsewhere certainly indicates that the foundation drains around the basement are a major source for the intrusion of stormwater into the sanitary sewers. Also, lot drainage, or rather the lack of adequate lot drainage around the house, is a major contributing factor in providing access of the stormwater to the sanitary sewer. The sources therefore lie largely on private property, making the cooperation of the individual homeowner a prerequisite to attacking the problem at the source.

In Kansas City, a detailed study on the inflow of stormwater into a sanitary system was conducted. The test area had houses with foundation drains and was known to have an extraneous flow problem.[1] It was found that over seventy-one percent of the houses had conditions that would promote the ponding of surface water against foundations and would require perimeter earth fill around the house to correct this condition. Downspout splash pads, which would direct roof run-off away from the foundation, were missing in twenty-one percent of the houses and about twenty-nine percent of the houses had window wells which were much too low. These conditions are remarkably similar to those cited earlier for the Pulberry area. The homeowners in Kansas City were presented with the report of the survey on their houses and requested to make the necessary corrections. By coincidence, a developer in the area was able to make fill dirt available free of charge to homeowners wishing to use it. The area was subsequently resurveyed and it was found that over half of the residents made the required perimeter fill around the house. However, very few raised window wells, even when placing the perimeter earth fill with the result that the re-survey indicated an increase in the number of low window wells. Very few homeowners added splash pads. The study concluded that substantial improvement in overall surface drainage conditions around the houses in an existing area cannot be expected from individual effort, but that if the homeowners are given sufficient detailed information accompanied by field checks, a worthwhile improvement might be obtained. Locally, the communities of Transcona and St. Boniface have sent out notices to each home asking that the roof downspouts be extended further away from the house. The extent of improvement is not known but both areas still suffer from extraneous flow flooding problems.

Public education is essential to any program where the homeowner is asked to spend time and money voluntarily to make changes on his own property. The cost to the homeowner of having a landscaper correct the settled areas around his home can be substantial. To illustrate, a local landscaper has estimated that it would cost anywhere from $100.00 to $250.00

to raise and relandscape the settled area around a typical suburban house.[2] This includes lifting and replacing of the sod, providing topsoil fill and downspout extensions, splash pads and raising window wells. The lower estimate would apply if flower beds existed around the house thereby making it unnecessary to replace sod. These are substantial costs for most homeowners. At four houses per acre, and assuming an average of $150.00 per house, this cost would amount of $600.00 per acre, a very significant increase in cost of servicing a typical suburban acre. In order to motivate homeowners to spend these amounts of money, they must be made aware of the cost of extraneous flow to the community and also to the individual. Also, the citizen must be educated as to the causes, sources and effects of extraneous flow and what he can do to ease the problem. This applies to both new and existing areas.

Recognizing that not nearly every homeowner will be prepared to spend the money and effort to correct all the deficiencies around his home, it would be desirable if the homeowner could be advised as to which are the most critical defects in terms of extraneous flow intrusion, so that the available time and money can be spend most effectively. At the moment the problem has not been sufficiently defined locally to enable specific directions. The installation of concrete splash pads to convey roof drainage away from the house was found to be effective in Oakland County, Michigan.[3] The American Public Works Association has suggested a five to ten foot extension of splash pads away from the house for the purpose of reducing extraneous flow.[4] Locally one house builder has proposed fastening splash pads rigidly to the foundation wall of new houses, and extending the splash pads to a side swale which in turn would drain either to the rear of the lot or to the front street.[5] The rigid connection was proposed to enable the pad to bridge the settlement around the house and to avoid the removal of the splash pad by the homeowner. A local consulting firm has estimated that this method could reduce the flow from foundation drains by fifty to sixty percent.[6] The degree of effectiveness of this measure is debatable, but if this technique would direct most roof drainage permanently away from the foundation drain, it would be a worthwhile improvement. It is a relatively inexpensive measure that can be readily incorporated into new homes. A rough cost estimate would be about $40.00 per house, two pads per house at $20.00 per pad, or, at four houses per acre, a cost of $160.00 per acre. This is certainly a lot less expensive than having a landscaper correct the problem some years after the initial house construction. Allowing for some additional landscaping costs to convey splash pad drainage away from the house, this cost could be estimated at $200.00 per acre.

In new areas, improved lot grading would seem a likely measure to reduce the inflow of stormwater into sanitary sewers. Typically, settlement of the lot towards the house occurs and the resulting depression intercepts overland flow and, of greater significance, tends to direct roof drainage back to the foundation wall and to foundation drains. Recently, in some areas of Winnipeg subdivisions have been built with steeper slopes on lot grades and with rear lot swales. The lot drainage is both to the back and front away from the house. A real effort has been made in these areas to direct stormwater away from the house and it is reasonable that the inflow of stormwater should be reduced. The degree of success cannot be determined without field testing. Other methods to reduce extraneous flow are tight excavation of home basements to prevent settlement and better compaction around basements. Some developers are considering paved lanes to facilitate surface drainage.

All these proposed methods may reduce the intrusion of stormwater into sanitary sewers, but it is doubtful whether the extraneous flow problem would be eliminated. A representative of a prominent international consulting firm experienced in this problem stated that where foundation drains are permitted, large inflows of excess water into sanitary sewers probably cannot be avoided.[7] This speaker stated that significant reduction in stormwater intrusion is possible with a serious campaign against the sources but there is a practical limit to how much can be reduced and, after this point, there may still be overflows from a separate sanitary sewer system. In Kansas City, the location of the intensive survey discussed earlier, it was found that the prevention of extraneous flow at the source met with only a minimum of success in newer districts.[8] It was found that the area could not be policed well enough to exclude the illicit discharge of water to the system. The authorities there are attempting to lessen the problem by controlling the surface drainage around structures. The practical solution adopted in Kansas City for the problem of extraneous flow was to provide sedimentation and disinfection for the overflows before they reached the receiving stream. In the Winnipeg area, considerable care was taken in the development of the Southdale area in St. Boniface to extend roof drainage away from the foundation. Some improvement was apparently obtained but this area still has considerable extraneous flow.[9] In Thompson, Manitoba where the foundation drains are connected to the storm sewers for each house, basement flooding and extraneous flows are still a problem during heavy rains.[10]

Thus it appears that in spite of a conscientious effort to avoid the intrusion of stormwaters into sanitary sewers, prudent design dictates that some hydraulic capacity in the sewers must be allocated to this stormwater. This does not infer that the attempt to reduce extraneous flow at the source should be abandoned. On the contrary, it is only good sense to reduce the entrance of stormwater to the sanitary sewer system to the practical limits.

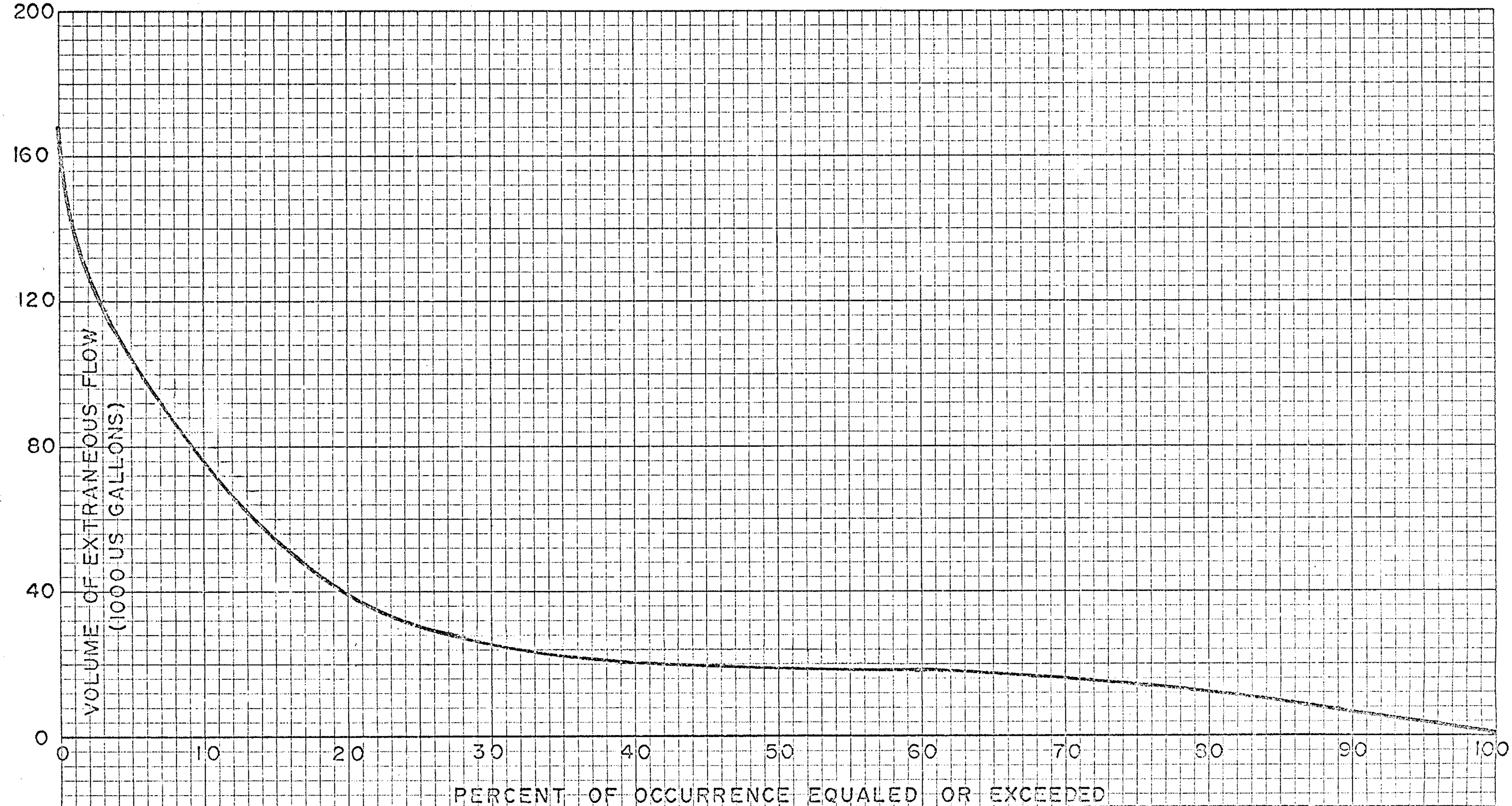
B. Overflow to Receiving Stream

One obvious solution to the problem of excessive flow in the sewer is to revert to the principle of the combined sewer, namely, to spill excess wastewater to the rivers during rainstorms. If faced with a choice between flooded basements or overflow relief to the river most homeowners would undoubtedly opt for overflow but, as an accepted policy, this practice would have serious short-comings. Perhaps of greatest significance, at a time when the public appears to be asking for improved pollution control, the dumping of raw sewage to the river during rainstorms may not meet with public approval. Also, the release of sewage in this manner during an area-wide rainstorm may in fact be a significant pollutional loading to the river. Certainly, the dumping of raw sewage to the river would not be desirable from a public health standpoint. In areas remote from the two major rivers in Winnipeg, discharge of raw sewage into small streams would be totally unacceptable.

The probable pollutional loading of the stormwater flow in the sanitary sewer system in the test area can be estimated from the data presented previously. The volumes of extraneous flow generated from the storms occurring during the test interval are shown in Table 7 and the volume of extraneous flow for the entire summer was estimated to be 1,440,000 gallons. The pollutional strength of the total sewage flow during the period when extraneous flows are occurring is higher than the average dry weather strength due primarily to the flushing or scouring of material deposited in the sewer during periods of tranquill flow. As indicated earlier, the average strength of the total flow during rainstorms can be expected to be eighty percent greater than the average dry weather sewage strength in terms of BOD. The average dry weather strength of sewage may be taken as 170 ppm BOD, which is the annual average sewage strength to the Charleswood Lagoons, a treatment facility serving primarily a residential area.[11] With these data, the pollutional potential of the extraneous flow can be calculated relative to the dry weather flow.

If no extraneous flow were to be intercepted and all this polluted stormwater were to be discharged to the river, a total of 1,440,000 gallons of sewage with an average strength of 180 percent of 170 ppm BOD would be directed to the river. This would result in a total annual load of 1,440,000 gal. x 8.3 lb/gal. x 170 ppm. BOD x 180% = 3670 lb/BOD. In comparison, the total annual BOD produced by this same area under dry weather conditions is average flow x average strength or 6500 gphr x 24 hr/day x 365 days/year x 8.3 lbs/gal. x 170 ppm. BOD = 81,000 lbs. BOD. Therefore all extraneous flow would account for $\frac{3670}{81,000}$ or about 4.5 percent of the total pollutional load generated from this system. This is similar in magnitude to the amount of pollution in the overflow from combined sewer systems. In the City of Winnipeg, the combined sewer system is based on the diversion of 2.75 x average dry weather flow during rainstorms with the remaining flow being discharged to the river via overflows.[12] This interception ratio is common with combined sewer systems and results in about five percent of the total annual pollutional load being lost to the river. Thus, in comparing the situation where all extraneous flow in a sanitary sewer were to be by-passed to the river to the older combined system, it is surprising to see that both systems are approximately equal with respect to the pollutional load captured. On this basis, the separate system does not appear to be of a worthy successor to the combined system. However, the typical sanitary sewer system does not overflow all extraneous flow. As was noted earlier, the typical design according to dry weather conditions makes an allowance for peak dry weather flows which is in excess of the dry weather peak rates actually experienced, thereby making an inadvertent allowance for handling extraneous flow. The effect of the interception of this and other portions of extraneous flow in terms of the pollutional load lost to the river can be calculated as follows.

The volume of extraneous flow associated with each individual storm, as shown in Table 7, was plotted in terms of the percent of the rainstorms that equalled or exceeded this given volume. See Figure 11.



DISTRIBUTION OF EXTRANEOUS
FLOW VOLUMES
(PULBERRY DISTRICT)

FIG. 11

This resulting curve defines the probable contribution of each storm during the year to the average volume per rainstorm. The area under the curve represents a composite of these contributions per storm, that is, the total area under the curve is the average volume of extraneous flow per storm. The numerical volume of this average can be calculated as illustrated in Table 8.

TABLE 8

AVERAGE VOLUME OF EXTRANEOUS FLOW PER STORM

Volume Per Storm (gal)	Storms Equalling or Exceeding Volume (%)	Average Volume per Storm for Interval (gal)	Average Contribution per Storm (gal)
168,000	0		
		135,000	135,000 x 5% = 6,750
102,000	5		
		91,000	91,000 x 5% = 4,550
80,000	10		
		60,000	60,000 x 10% = 6,000
40,000	20		
		31,000	31,000 x 20% = 6,200
22,000	40		
		20,500	20,500 x 20% = 4,100
19,000	60		
		15,500	15,500 x 20% = 3,100
12,000	80		
		6,000	6,000 x 20% = 1,200
0	100		
Average Volume per Storm			31,900

The average volume per storm is thus 31,900 gallons. A total of thirty significant rainstorms occurred during the test interval while about ten inches of rainfall fell. In the entire summer, about fifteen inches can be expected, as indicated previously, and therefore about forty-five significant rainstorms can be expected during the entire summer. On this basis, the annual volume would be forty-five storms x 31,900 gal/storm or 1,436,000 gallons which checks very closely to the 1,440,000 gal. calculated previously in Chapter 5.

The same procedure can be used to estimate the volume of extraneous flow intercepted at various extraneous flow peak factors. For example, the actual observed peak dry weather flow factor was 2.15; however, this system would have been designed for a peak factor of 3.4 according to normal design procedure. Therefore, the system would have a "built-in" EFPF of 3.40 - 2.15 = 1.25. Figure 10 indicates that a rainstorm that will produce extraneous flow to the extent of an EFPF of 1.25 can also be expected to have a volume of 22,000 gal. associated with this storm. Figure 11 indicates that, on the average, thirty-seven percent of the rainstorms in the year will likely produce greater volumes of extraneous flow, and conversely, sixty-three percent of the rainstorms will have lesser volumes. This means that a sanitary sewer system with a stormwater capacity corresponding to an EFPF of 1.25 will be capable of carrying all extraneous flow from sixty-three percent of the rainstorms without overflow to the river. Of the remaining thirty-seven percent of the rainstorms, this sewer system would be capable of carrying only a part of the stormwater volume produced by these rainstorms with the remaining portion being discharged as overflow. Specifically, for the example of 22,000 gal/storm, the average stormwater volume per storm for the sixty-three percent of the rainstorms that would be completely handled by the sewers is 14,000 gallons as determined graphically from Figure 11, and the total volume of extraneous flow carried by the sewers from these storms is sixty-three percent x 45 storms/year x 14,000 gal/storm = 397,000 gallons/year. Of the thirty-seven percent of the storms producing greater than 22,000 gal/storm of stormwater, the average volume per storm is 62,000 gallons; which results in a total of thirty-seven percent x 45 storms/year x 62,000 gal/storm = 1,030,000 gallons/year. Of this volume, the sewers would only overflow that volume in excess of the 22,000 gallon/storm that the sewers are capable of conveying. Therefore, thirty-seven percent x 45 storms/year x 22,000 gallons/storm - 365,000 gal/year would be carried by the sewers and 1,030,000 gallons - 365,000 gallons = 665,000 gallons/year would overflow. The total amount carried by the sewers is then 397,000 gallons + 365,000 = 762,000 gallons/year. The total of these volumes, which incidentally is also the area under the curve in Figure 11, should account for all extraneous flow. The total of 762,000 gallons carried by the sewers plus the 665,000 gallon overflowed is 1,427,000 gallons/year which checks closely with the 1,440,000 gallons determined earlier as the total annual volume of stormwater. Other similar calculations were done at various EFPF values to produce Table 9.

TABLE 9

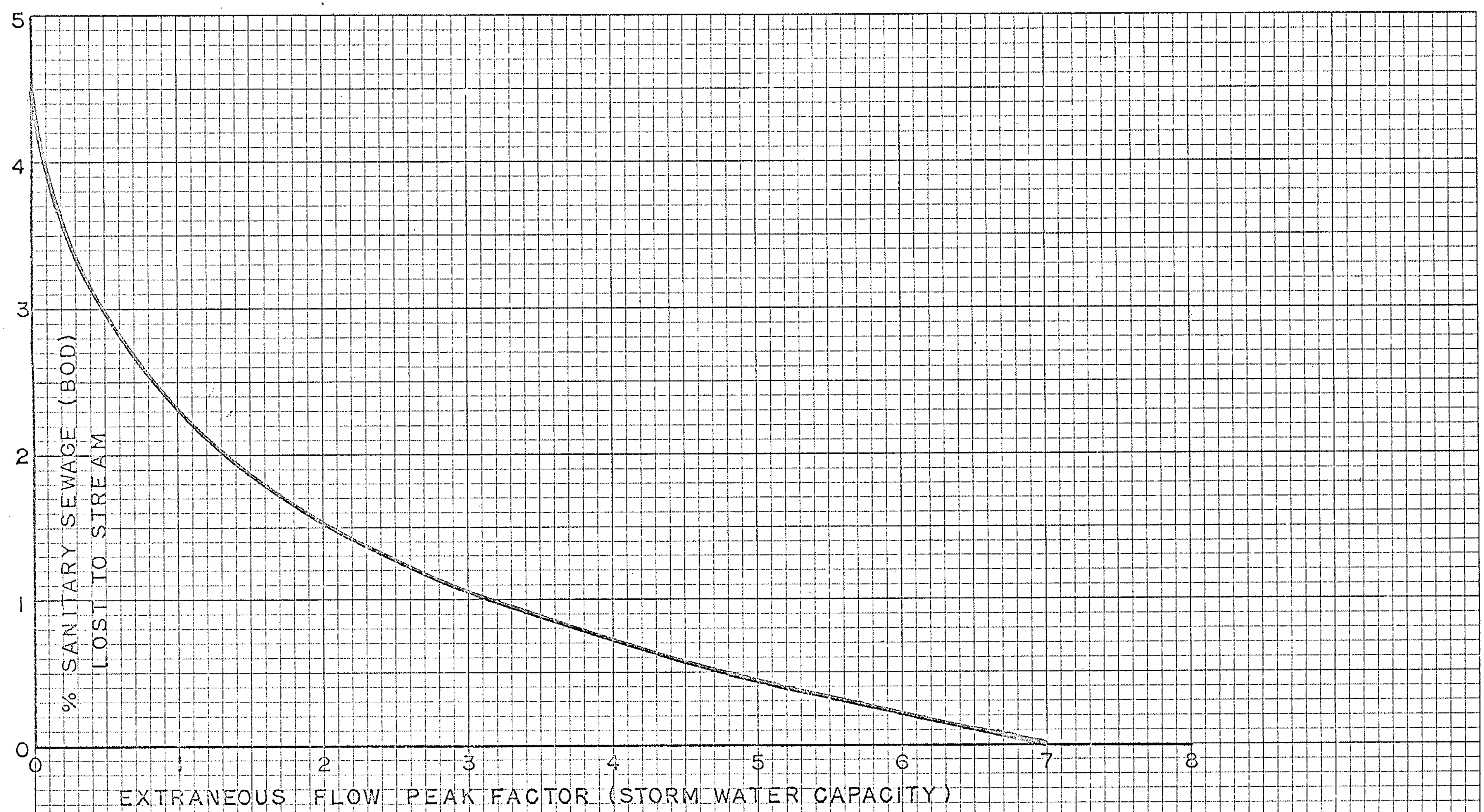
EXTRANEOUS FLOW PEAK FACTORS AND CORRESPONDING OVERFLOWS TO RIVER

E.F.P.F.	Annual Volume Overflowed (gal)	Annual Pollutional Load of Overflow lbs. BOD*	% of Annual Dry Weather Load
0	1,440,000	3,670	4.5
1.25 (normal design)	650,000	1,685	2.1
3.0	354,000	900	1.1
5.0	120,000	300	0.4
7.0	28,000	70	0.001

* based on 180 percent of average strength of dry weather flows (170 ppm. BOD)

This estimated overflow pollution is also illustrated on Figure 12. From the standpoint of intercepting pollutional load to the river, it would appear that provision of stormwater capacities in excess of an EFPF of about three could not be justified. The additional pollutional load intercepted from overflow by greater allowances is only a maximum of 900 lbs/year or, on the basis of 0.2 lbs. BOD per day per person, a population equivalent of about twelve people.[13] It would not appear justifiable to double design factors to capture this small remaining pollution.

The table illustrates that for typical present sanitary sewer design, about 2.0 percent of the total pollutional load developed in the test area would be lost to the river if all flows in excess of the design



ESTIMATED OVERFLOW

POLLUTION FROM

SANITARY SEWERS
(PULBERRY DISTRICT)

FIG. 12

peak factor were diverted to the receiving stream. This does not take into consideration the times when rainfall does not coincide with peak domestic flow which would provide greater capacity for extraneous flow nor does it fully take into account the volume of storage available in the sewer system which was earlier shown to be highly significant. In fact, in the test area, the extraneous flow from over half the storms could be stored in the sewers. With these factors considered, the pollutional load lost to the river with present design practice would very likely be less than one percent of the total load generated by the system on an annual basis. This is substantially better than the typical five percent associated with the combined sewer system, indicating that the separate system does in fact reduce pollution to the river. However, it is noteworthy that the improvement in pollution control with sanitary sewers is costly. Studies in Detroit show that the cost to provide a separate sewer system in a new area may be approximately twice that of a combined sewer system, if both storm and sanitary sewers are constructed at the same time.[14] The benefits of a separate system that is vulnerable to extraneous flow are obviously not up to original expectations. This illustrates that pollution control starts in the sewer system and extraneous flow must be brought under better control if full benefit of the separate sanitary sewer system is to be achieved.

While the overflows from a sanitary sewer system may not appear large on a long-term basis, it is possible that the amount of pollution in the extraneous flow from a severe storm may be of greater significance on a short-term basis. To illustrate, the July 7 rainstorm produced a total of 118,000 gallons of extraneous flow (Table 7) in an interval of eight hours. The total pollutional load of this extraneous flow can be estimated at 118,000 gal. x 8.3 lb/gal. x 170 ppm x 180% = 300 lb. BOD. During this eight hour interval, the normal dry weather flow would have had 6500 gphr x 8 hr. x 170 ppm. BOD x 8.3 lb/gal = 70 lb/BOD. The extraneous flow thus had about four times the total pollutional potential of the normal dry weather conditions. Stated another way, overflowing all stormwater for eight hours during this rainstorm would have been roughly equivalent to complete bypass of dry weather sewage from the test area to the river for about thirty-two hours.

In areas remote from the two main rivers in Winnipeg, the discharge of such volumes of raw sewage to small streams is clearly unacceptable. It can be a argued that the discharge of raw sewage during rainstorms to the main rivers is no different than the permitted overflows from combined sewers. However, the public, who should be the final authority on such a matter, may not approve of such practice. A recent public attitude survey showed that the majority of the citizens in Winnipeg do value the local rivers and would like to make increased recreational use of them and, further, were prepared to pay more for improved pollution control.[15] In view of this, adopting new policy to allow deliberate discharge of raw sewage to the river would appear to be a backward step in pollution control.

The main arguments against overflow of this sewage, even if it is a relatively small amount, are public health protection against discharge of raw sewage and aesthetic reasons. With even minimal treatment this overflow probably would have little impact on the stream. Therefore it is concluded that the overflow of extraneous water should be given treatment for disinfection and removal of settleable and floating solids.

C. Providing Capacity for Extraneous Flow

To avoid the overflow of sanitary sewers during rainstorms, hydraulic capacity can be provided in new sewer systems to accept these flows without excessive surcharging. If the flows are to be conveyed to treatment in the same manner as the normal wastewater, the treatment facilities must also be sized to accept these peak flow rates. This method certainly would be the most convenient way of dealing with extraneous flows since no party (the homeowner, householder or contractor) would have to change his present customs. But would this method exact a premium price and is it a reasonable solution to the problem? A rough estimate of the impact of such an approach on the economics of sewage disposal can be made.

The typical sewer system for the collection of wastewater from a Winnipeg sub-division will have about 150 feet of sewers per acre.[16] This total length of sewer will have an average distribution into size ranges as shown in Table 10.[17] This sewer system would not have deliberate hydraulic allowance for extraneous flows.

TABLE 10

TYPICAL DISTRIBUTION OF SEWER SIZES

Size Range (inches Diameter)	Percent of System Length
6 - 8	75
10 - 12	14
15 - 18	6
21 - 27	4
30 - 42	1

If this same system were to be designed to allocate certain capacity to extraneous flow, the sewer sizes would increase as would the costs. An approximation of the effects on sewer sizes of increasing the design flow rate can be obtained readily by assuming that the total energy or hydraulic head consumed in a given reach of sewer will be the same for the increased design capacity and, therefore, in order to discharge the increased flow rate, the sewer size will increase. This is a valid assumption in that in most collection systems, the maximum practical available hydraulic head will be used by the designer in selecting sewer sizes. With this condition, for example, a typical street sewer, commonly an eight inch sewer laid on a 0.04 percent slope with a capacity of 0.7 cfs. would have to be ten inches in diameter in order to discharge twice the flow rate (1.4 cfs) at the same gradient or utilizing the same energy head. This technique was used to estimate the change in sewer sizes and costs for the hypothetical design for the flow rates of twice and four times the rates as given by present design practice. As discussed previously, the present design practice for a typical small area, such as the test area, allows for a peak factor of 3.4 whereas only 2.2 occurred during dry weather conditions, thereby providing an extraneous flow allowance equal to about 1.2 x average flow (EFPF = 1.2). Revising the flow design for twice the capacity means the design flow rate has a peak factor of 2 x 3.4 or 6.8 of which 6.8 - 2.2 = 4.6 is an EFPF allowance. Similarly, designing for four times the typical dry weather design capacity would provide an EFPF = (4 x 3.4) - 2.2 = 11.4. The revised distributions of sizes for these conditions are shown in Table 11 together with the capital costs associated with each system. The capital costs were based on unit construction amounts incurred locally.[18] Only twenty-five percent of the sewers in the six to eight inch diameter size range were considered to be subject to change since the major length of the street sewers are made eight inches due to a minimum size requirement for operating conditions and not for hydraulic capacity. The cost of manholes is not included in the figures of Table 11 since this item will be common to all the designs.

TABLE 11

COLLECTION SEWER COSTS FOR VARIOUS PEAK FACTORS

Sewer Size Range (in. diam)	Unit Cost (ft)	Present Design Peak Factor (EFPF = 1.2)			2 x Normal Peak Factor (EFPF = 4.6)			4 x Normal Peak Factor (EFPF = 11.4)		
		Percent Length	Length (ft)	Cost ($)	Percent Length	Length (ft)	Cost ($)	Percent Length	Length (ft)	Cost ($)
6 - 8	6	75	112	672	55	82	492	50	75	450
10 - 12	8	14	21	168	30	45	360	28	42	336
15 - 18	14	6	9	126	7	11	154	7	10	140
21 - 27	22	4	6	132	6	9	192	8	12	264
30 - 42	34	1	2	68	1	2	68	5	8	272
48 - 54	49	0	0	0	1	1	48	2	3	148
Total per Acre				$1166			$1315			$1610
Increased Cost over Present Design				0%			14%			39%

The preceding costs represent the local collection system only and to these costs must be added the cost of trunk and interceptor sewers to convey the wastewater from the local community to a central point of treatment. These costs will depend to a large extent on the location of the given subdivision relative to the treatment plant. A general local unit cost can be obtained from the actual design for trunk and interceptor systems serving areas tributary to the existing North End Treatment Plant and the proposed South End Treatment Plant in Winnipeg. One northerly interceptor system had a total estimated cost of $3,000,000 to serve 16,000 acres and the southerly system was estimated at $5,000,000 to serve 24,000 acres.[19] The unit costs are then $218.00 and $208.00 per acre or approximately $200.00 per acre. It must be noted that the interceptor systems serve industrial and commercial establishments as well as residential areas and peak factors chosen for domestic flow do not apply to industrial sewage. For example, a typical interceptor sewer may have an average design flow equally divided between domestic and non-domestic sewage with peak factors of about 3.0 and 2.0 respectively. The effective peak factor for the total flow is $(3.0 \times 1/2) + (2.0 \times 1/2) = 2.5$. Doubling the domestic peak factor results in a total effective peak factor of $(6.0 \times 1/2) + (2.0 \times 1/2) = 4.0$ or a design flow rate of $4.0/2.5 = 160$ percent of the original rate. Similarly, providing four times the original domestic peak factor results in an effective total peak factor of 7.0 or total flow of $7.0/2.5 = 280$ percent of the original design rate. Providing increased capacities of 160 percent and 280 percent of the base design flow will result in increased interceptor sewer costs of 130 percent and 190 percent of the base design cost, according to estimates made for Kansas City.[20]

The unit cost per acre for interceptor sewers would thus increase to \$200 x 130% = \$260/acre and \$200 x 190% = \$380/acre for design allowance of two times and four times the normal domestic peak factor respectively.

If the sewer system will be designed to convey these peak flows to the treatment plant, this facility must have the hydraulic capacity to accept these flows. It may well be that a large interceptor sewer system will have dampened peak flow rates due to reduced coincidence of peak flow rates from tributary areas and the larger storage volume available in these larger sewers. For the purpose of this comparison, however, it will be assumed that the treatment plant will be sized for 160 and 280 percent of the capacity when designed for two and four times, respectively, of the normal dry weather domestic peak factor. The treatment process will be assumed to provide secondary treatment and costs will be based on the conventional activated sludge system. As discussed earlier, this is a biological process and large fluctuations in flow rates tend to upset the delicate balance between organic material in the wastewater and the micro-organisms needed to convert the pollutants to inert waste. It is likely that the biological stability of the process would be severely tested under these conditions and efficiencies of treatment could easily reduce to primary treatment levels.[21] The physical plant design would also present problems, such as avoiding deposition in dry flows and yet providing adequate retention times during rainfall. However, for this cost analysis, conventional plant design will be assumed for the plant cost estimates. The United States Public Health Service studied cost data on 133 actual treatment plant projects in 1967. This information was updated by the American Public Works Association in 1970 and provides capital and operating costs for the activated sludge process on a per-capital basis.[22] The capital cost data show the typical reduction in unit cost with increasing scale and, for a plant serving a population of 200,000 people, the data shows a capital cost of $23.00 per capita, including non-contract costs, such as land and professional fees. This cost can be converted to a unit acreage cost by simply multiplying by an appropriate population density. A density of twenty-five people per acre is typical of new developments in Winnipeg.[23] The treatment costs for normal design are than 1 acre x 25 ppa x $23/person = $576.00 per acre capital costs.

Similarly, for the revised design for twice the normal domestic peak factor or 160 percent of the normal plant design flow, and using the same cost estimating curve, the unit treatment costs can be estimated at 1 acre x 25 ppa x $20/person x 160% = $800.00 per acre for capital costs. For a design using four times the normal residential peak factors, or a plant design flow of 280 percent of the normal flow rate, the capital cost is 1 acre x 25 ppa x $18/person x 280% = $1,260.00 per acre.

The capital costs of providing various levels of revised sewer design can then be summarized as follows:

TABLE 12

CAPITAL COSTS OF INCREASED SEWER SYSTEM CAPACITY

Item	All Costs in $ Per Acre		
	Normal Design Peak Factor (EFPF = 1.2)	2 x Normal Domestic Peak Factor (EFPF = 4.6)	4 x Normal Domestic Peak Factor (EFPF = 11.4)
Collection Sewers	1,115	1,315	1,610
Interceptor Sewers	200	260	380
Treatment Plant	575	800	1,260
	$1,930	$2,375	$3,250
Increase from Normal Design	0	$ 445	$1,320

It is evident that the increase in capital cost to provide for extraneous flow capacity in the entire sewer system is very significant. The extent of capacity that should be allocated to extraneous flow is debatable. This would be determined best by an economic study of the cost and benefits of providing varying degrees of protection against flooding. The required degree of protection will be determined largely by the protection needed in the service area of the collection system. The designer is then faced with the selection of an appropriate allowance for extraneous flow in the design of a sewer system. Naturally, the allowance will depend to a large degree on the care and diligence exercised during the initial construction of the service area to prevent the inflow of stormwater into the sanitary sewer system. Also, it should depend on the permanence of preventative measures. The optimum capacity allowance for stormwater in the lateral sewers is primarily a matter of providing protection against flooding of property, and against the aesthetic inconvenience of flooded basements. As an analogy, the typical storm and combined sewers are designed for storms with an average return period of five to ten years. Logically, the sanitary sewers should provide at least this degree of protection since these sewers provide a direct connection to the home as opposed to storm sewers which drain the surface water only.

The extraneous flow peak factors were correlated previously to the frequency of rainstorms. However, there are several reasons why this correlation should be applied with great care. During the relatively short test period the data did not include storms of a magnitude which normally would occur less than once per year. The extrapalation of the data into areas beyond the regions monitored is extremely risky. Since a parabolic function was selected as providing the best fit to the data, it is possible that the curve is relatively sensitive to low frequency data. Also, it was shown that the amount of storage in the sanitary sewers is a major factor in determining whether flooding will occur. As shown earlier, over one half the storms in the tested area produced an extraneous flow volume that could be stored in the sewers by utilizing half the volume of the eight inch sewers alone. This certainly adds greatly to the degree of protection afforded by the sewers. Also some hydraulic surcharging of the sanitary sewers can occur, thereby providing increased hydraulic capacity. In addition it was shown that many storms occur during such a time of the day when the dry weather flows are below average, thereby also releasing some dry weather capacity to stormwater allowance and adding to the degree of protection. These factors tend to obscure any attempt to place a selection of appropriate extraneous flow allowance on a statistical basis. Figure 8 shows that for a storm of an average return period of once in five years, or 0.2 times per year, an EFPF of thirty could be expected. This is an astounding peak factor and would be dismissed as irrational were it not for the fact that areas have reported peak factors of this order. Nevertheless, the data for this specific test area is too scanty in the low frequency range to lend sufficient confidence to use this range of the curve for design purposes.

Since the foundation drains of individual homes are suspected to be the chief cause of extraneous flow, it would be convenient to express any stormwater allowance in terms of a unit rate per house. A maximum extraneous flow rate of 46,000 gphr or an EFPF of seven occurred during the test period (Table 7). No basement flooding in the test area was reported to the responsible authorities. This peak flow rate also corresponds to a stormwater flow rate of about $\frac{46{,}000 \text{ gphr}}{700 \text{ houses}} \times 60$ min/hr. $= 1$ gpm per house. Flow rates of about 0.8 gpm were measured from the foundation drains of a typical suburban home in 1970 to add credibility to this flow rate.[24] It would appear that an allowance of about one gpm per house would be in the correct order of magnitude for an appropriate design allowance for new areas similar in size and characteristics to the test area. Washington D.C. has developed design curves for stormwater allowances in separate sanitary sewers which consider size of area and population density.[25] These curves are based on a design storm with an average return period of fifteen years. By coincidence, the suggested one gpm/house for the test area results in almost the same stormwater allowance as given by the Washington curves for the same size and density of area. The Washington curves show a significant decrease in design stormwater allowance per unit area as the service area increases. For example, the design allowance for a 2000 acre area is about one-half that of a 100 acre area, all on a unit area basis. It is logical that this trend of reducing unit allowance for extraneous flow with increased areas should exist and this should be evaluated for large areas locally.

Increased attention is being given to preventing stormwater from finding access to the sanitary sewers in new developments and reduction in the suggested one gpm per house allowance could be justified in an area where a concerted effort to reducing extraneous flow had been made. Also, the allowance may be different for multiple family units. The suggested allowance is admittedly arbitrary, but until the intrusion phenomena is fully documented, the designer will have to select an appropriate allowance based on judgement pending further information. The effect of the suggested allowance on the cost of providing sewer service for an area will be discussed shortly. It is clear that if extraneous flow rates of the present magnitude are permitted to continue in the sanitary sewer system the designer must make provisions for them or find an acceptable alternative method. It is certain that the public will not consider periodic basement flooding as an acceptable risk.

The cost to provide total sewage disposal systems designs for the suggested extraneous flow allowance of one gpm/acre (which corresponds to an EFPF of 7.0) can be estimated from Table 12. The following breakdown results.

TABLE 13

COSTS TO PROVIDE SUGGESTED EXTRANEOUS FLOW CAPACITY

	Total $/acre	Increase Over Present Design $/acre
Collection System Sewers	1,450	295
Interceptor Sewer	300	100
Treatment Plant	950	375
Totals	$ 2,700	$ 770

To provide system capacity for the suggested stormwater allowance would cost about $770.00 per acre or 40 percent more initially than present design. Considering thirty year debentures and nine percent interest, the annual cost of this increase in capital comes to about $76.00 per acre.

Even accepting that the collection system should be designed in any case to allow for extraneous flow to the extent of an EFPF of about 7.0, the remaining additional costs of $375 + $100 = $475.00 per acre for interceptor and treatment costs are substantial. When the relatively small volume of extraneous water actually collected over the year is considered, it seems that providing interceptor sewer and plant capacity for such sharp short-term peak flow rates is not the most effective method. The fact that volumes of extraneous stormwater are low suggests that storage of flows may have application in reducing system costs.

The foregoing analysis is admittedly an approximation but serves the purpose of estimating the relative magnitude of the costs involved in providing system capacity for extraneous flow peaks and demonstrates that the increase in costs would be significant to the homeowner. Also, while the costs of coping with excess flows may be hidden, the costs exist nevertheless.

D. Storage and/or Treatment of Extraneous Flows

One of the few locations where the problem of extraneous stormwater in sanitary sewers has been investigated and brought to a satisfactory solution is Johnson County, Kansas City.[26] Here it was considered that significant amounts of extraneous stormwater would occur in spite of diligent efforts to reduce this problem at the source. Also, it was concluded that it was impractical to design a biological treatment plant to cope with the extreme fluctuations in flow rates of a sewer system subject to stormwater intrusion. Under these flow conditions, the authorities at Johnson County felt that a secondary treatment plant would provide treatment only equal to primary sedimentation. The alternative of discharging raw sewage to a receiving water was not acceptable and the collection sewer system had to be designed to prevent basement flooding. The problem was resolved by providing treatment of diverted excess stormwater flows by primary sedimentation and chlorination. The design is such that when flows in the original sewer reaches approximately the capacity of these lines a signal from a control point in these sewers opens a gate valve which diverts flow to a new holding station. In the station the sewage is screened and pumped to a primary settling basin and immediately chlorinated. Here the settleable solids are removed and floating matter skimmed off the tank. All control is automatic. Sludge and skimmings are returned continuously to the nearest manhole downstream to be taken to the conventional treatment plant. Chlorine is added at ten ppm for disinfection and odour control. After the station and treatment facilities have been used for the storm, the entire structure is de-watered and washed down. A spray system is provided but final cleaning is accomplished by manual housing and brooming. A ventilation system is necessary for safety. The cost of this entire facility, including all automatic controls, land, super-structure, engineering and legal fees was approximately $395,000.00 for 20 mgd flow rate. The authorities report good experience with this facility and are constructing two similar projects in other areas of the county. A sedimentation tank is designed to provide thirty minutes retention time. The efficiency of treatment of a typical primary process is about thirty-five percent in terms of B.O.D. This method then not only reduces organic pollution substantially, but also disinfects the overflows to the stream, providing a satisfactory solution to the extraneous flow problem.

For the test area a flow rate of about 46,000 gphr was suggested as being appropriate for design allowance. This is approximately a one mgd flow rate. A holding station similar to the facilities at Johnson County could be constructed in the vicinity of the existing pumping station and operation could be automated as described. The cost of a one mgd facility could be estimated at $\frac{1 \text{ mgd}}{20 \text{ mgd}} \times \$395{,}000 \times 150\% = \$30{,}000.$

The additional fifty percent cost would allow for higher costs due to smaller scale. A rough breakdown of the components of such an installation was done to confirm that the estimate is approximately correct. On a unit area basis this cost is approximately $\frac{\$30,000}{120 \text{ acres}} = \$ 250.00$ per acre, a cost which appears to be an attractive solution relative to the previous alternatives.

In Boston, a similar approach to the method above has been used in coping with the overflows from combined sewers.[27] A stormwater detention and chlorination station was specifically constructed to accept excess overflows from combined sewers. The components of this facility are essentially the same as that described for Johnson County. There also, the cost was found to be about $20,000 per mgd of capacity. A more sophisticated system is under construction in the city of Dallas to handle extraneous flow problems. The proposed treatment plant consists of a diversion structure, pumping station, flocculation and sedimentation basins and chemical feed facilities.[28] The plant is designed to use waste lime sludge from the water treatment plant in the treatment of stormwater overflows to obtain nutrient removal. This treatment facility is estimated to cost about $38,000 per mgd and the treatment efficiency anticipated is in the order of forty percent removal of BOD and sixty percent removal of suspended solids. The treatment facility will provide an estimated thirty percent reduction in cost as compared to providing parallel sewer capacity.

In Milwaukee, a demonstration project on the treatment of combined sewer overflows using a combination of mechanical screening and dissolved air flotation has been tested. Chlorine was used for disinfection. The removals of BOD and suspended solids ranged from thirty-five to forty-eight percent without flocculating chemicals and fifty-seven to seventy-one percent with chemical aids.[29] The contact time for the chlorine was about eighteen minutes and effluent quality met the state bacteriological criteria for full body-contact waters. The capital cost of this type of treatment facility based on an actual bid was $24,000/mgd., including all land and engineering costs, for a fifty-eight mgd capacity plant.

These case histories demonstrate that while stormwater overflows from sanitary sewer systems may occur, they can be handled effectively with treatment devices at a cost that is competitive with alternative systems.

Simple storage to hold stormwater flows until peak flow rates have passed and then pumping this captured water back into the interceptor sewers and thence to treatment may be an effective method of enabling full secondary treatment of all flows. This storage need only be a supplementary larger sewer which would act as a wet-well storage for stormwater. For example, perhaps, in the test area a 500 foot length of 72 inch diameter sewer constructed at perhaps ten feet below ground level could be installed adjacent to the existent pumping station. The present pumping station could then be operated such that under normal flow conditions one pump only would operate and pump sewage to the interceptor sewer. The pumping capacity of this single pump would relate to the downstream capacity of the interceptor. As the flow increased to the point where the single pump could not keep the level in the wet-well down, as during rainstorms, a second or third pump would be activiated by liquid level controllers. These auxilliary pumps could pump the excess flow to the elevated seventy-two inch diameter sewer. When the storm had passed the captured stormwater flow could be returned to the interceptor sewer by gravity and at a rate which would not tax the downstream treatment facilities. The five hundred foot of seventy-two inch diameter sewer would provide approximately 100,000 gallons of storage, a volume which would be in the same range of protection as the flow rate allowances discussed earlier. The cost of this sewer would be about 500 ft. x $60/foot = $30,000. Adding an allowance for flushing facilities, the total cost could be in the order of $35,000 or about $300.00 per acre. This is higher than the previous methods outlined which offered primary treatment, but the above method would provide treatment to a secondary degree through the conventional plant. Constructing storage in a rectangular tank would cost approximately the same as providing the storage in the form of sewer. A storage sewer could possibly present less of a property problem.

It is possible that the judicious placement of storage at points on the interceptor system itself may be more beneficial than storage at the interface of lateral and interceptor sewers. The optimum location of storage could only be determined by a complete study of the economics of alternative systems. Regardless of the optimum location, storage with or without independent treatment appears to have considerable merit in dealing with the extraneous flow problem.

E. Connection of Foundation Drains to Storm Sewers

As discussed earlier, the connections of foundation weeping drains into the sanitary sewer is permitted in the Winnipeg area. These foundation drains are essential to relieve hydraulic surcharge on the basement floor and walls and to maintain uniform moisture content in the clay soil. If these drains are the chief source of extraneous flow into the sanitary sewers, the merits of connecting these drains to the storm sewer should be examined. Certainly the storm sewer is the intended conduit for this stormwater since much of the foundation drainage must be roof drainage which the storm sewer was designed to convey. But connecting the foundation drains into the storm sewers is not simply done. In fact, in existing areas the cost of plumbing changes on private property alone may be prohibitive. In new areas such a procedure would require a completely revised design procedure for storm sewers. Most storm sewers are designed to carry peak storm runoff at conditions of hydraulic surcharging, the hydraulic gradient being permitted to rise to within a foot or so of the street level at the upper reaches of the sewer. This means the hydraulic gradient would be well above the foundation drains since basements are typically about five feet below street level. Connections to the storm sewer under these circumstances would actually aggravate the basement drainage and flooding problem. Therefore, the re-designed storm sewers would have to be lowered some five or six feet at the upper reaches to avoid basement flooding. This represents a considerable loss in available energy to conduct storm runoff in these sewers. In addition, under present design, the storm sewers do not necessarily extend the full length of each street. Instead, the designer can use the graded streets to transfer runoff along the street to suitable street inlets and thence to the top end of the storm sewer. In this way, about one-third of the street length in a subdivision may be without storm sewers. These storm sewers would have to be extended in a design that called for individual house connections. The actual connection itself would require a separate sewer from the storm sewer to the basement floor drain including a sewer trap and vent pipe.

An approximate cost per dwelling of a separate connection of the foundation drain to the storm sewer can be estimated as follows. Consider two houses opposite each other on a typical street, each placed twenty-five feet back from the property line. Consider that the sanitary sewer is on the center line and the storm sewer on the fourteen foot line on a sixty foot public right-of-way. Each house will then have a six inch sewer connection of about thirty feet to the property line and an additional thirty feet on the street to obtain connection to the sanitary sewer. To connect to the storm sewer, one house will have a $30 + (60 - 14) = 76$ feet long connection, forty-six feet of which is common to the sanitary connection and about thirty feet of which will require augering beneath street pavement. The other house will have a forty-four feet long connection which will be in a common trench with the sanitary connection and not have to cross pavement. The cost of the two house connection is then:

Two sewer traps, 2 x $20.00 installed	$ 40.00
6" sewer pipe, 120' @ 75¢/ft.	90.00
Sewer installation - 90' common trench @ $1.00/ft.	90.00
- 30' augering under pavement @ $8.00/ft.	240.00
Connections to storm sewers, 2 @ $40.00	80.00
Vent connections, 2 @ $20.00	40.00
Engineering and contingencies, 15%	90.00
TOTAL 2 houses	$ 670.00

This represents an average cost of $335/house. To this must be added the incremental cost of revisions to the storm sewer layout. These costs can only be determined accurately if an area is designed with and without foundation drain connections. The cost can be approximated by assuming that the sewer will be deeper and larger for about half its length, thereby incurring extra costs of excavation, backfill, and pipe strength. In addition, it can be assumed that about one-third of the area, formerly using streets for storm drainage and without a street storm sewer, will now require a twelve inch diameter storm sewer. The cost for the two houses is then as follows; assuming a sixty foot width lot;

Deeper Sewer - excavation 30' @ $2.00/ft.	$ 60.00
- backfill 30' @ $0.50/ft.	15.00
- strength increase 30' @ $0.50/ft.	15.00
- size increase 30' @ $4.00/ft.	120.00
Extending Storm Sewer - 12" sewer for 60' @ $10/ft. for average of 1/3 of frontage	200.00
Engineering & Contingencies @ 15%	60.00
TOTAL 2 houses	$ 470.00

This results in an estimated cost of $235/house which, coupled with the connection cost of $335/house, provides an estimated total average cost of $570/house.

This analysis obviously is not intended to be precise but rather to give an indication of the relative cost of this concept. The estimate does check closely with more detailed estimates of costs per dwelling done by others.[31] The unit cost of $570/house would result in a cost of $2,280/acre when considering a typical density of four dwellings per acre.

Relative to the costs of alternative schemes produced earlier, this solution is costly. Further, separation of the foundation drain from the sanitary sewer may not be the ultimate solution for the extraneous flow problem. Clearly, from what is known, this procedure would eliminate the major source of extraneous flow, but that remaining fraction might still force increased capcity in sanitary sewers. Foundation drains are connected to the storm sewers in Thompson, Manitoba, but basement flooding from sanitary sewers is still reported. In Oakland County, areas without the connection of foundations to the sanitary sewer had much reduced extraneous flow rates although the stormwater intrusion was not eliminated.[32] Separate connections of the foundation drains to storm sewers would be a costly method of controlling extraneous flow. The major advantage of these connections would be the permanence of the separation of this stormwater flow from sanitary sewage. This separation, unlike any revised lot grading, would not be subject to the penchant of the homeowner to make periodic changes. Until more definitive information on the role of foundation drainage in the extraneous flow problem is obtained, the full benefits of individual connections cannot be evaluated adequately. This factor, together with the high cost associated with this procedure, indicate that individual house connections to the storm sewer is not the preferred solution for single-family dwellings.

It is possible that the cost of separate connections may be justified for multiple-unit housing where a single connection serves at least two units. The emphasis in recent construction has changed from single-family to multiple-unit housing. For example, in the period 1961 to 1971 almost two-thirds of new housing in Canada was built in some form of multiple-unit construction, such as duplexes, town-houses and apartment buildings.[33] Thie type of housing may merit special consideration with respect to separate connections.

F. Comparison of Alternatives

Some alternative methods of coping with extraneous flow and the cost estimates associated with these proposals have been presented. A comparison is facilitated by the table below.

TABLE 14

SUMMARY OF ALTERNATIVE METHODS OF HANDLING EXTRANEOUS FLOW

Method	Description	Cost $/acre	Remarks
Eliminate flow at source	- improve lot drainage in new areas	$ 200	Not sufficient in itself
	- relandscape selectively in existing areas and install splash pads	480	Requires cooperation of homeowner
Overflow of raw sewage	- simple overflow to river during rainstorm	negligible	Pollution problem, totally unacceptable in areas with small receiving streams
Storage and/or Treatment of Extraneous Flow	- storage, sedimentation and chlorination.	250	Positive method of providing primary treatment
	- simple storage and repumping	300	
Providing capacity in new sewage disposal system	- collection sewer system	295	Capacity is essential
	- interceptor system	100	
	- treatment plant	375	Effective treatment may be impractical
Separate Foundation Drain Connections to Storm Sewer	- practical in new areas only	2,200	Costly and not proven totally effective

In designing for new areas, it would appear that the best approach is a combination of methods, namely, attempt to reduce access of stormwater to the sewer by improved lot drainage and splash pads, provide hydraulic capacity in the collection sewer system, and provide treatment of extraneous flow in excess of downstream interceptor sewer design flows. The attempt to reduce extraneous flow at the source needs no justification. Permanent splash pads and improved long term lot drainage away from the house are considered good approaches. The collection system must be capable of accepting the flows generated by the local area with a reasonable degree of protection. A design of one gpm/house is proposed as being a reasonable judgement allowance for typical small areas with normal building practice. Further, it is cheaper to store and/or treat extraneous flow ($250/acre) at the interface of the collection sewer system and the interceptor sewer system than to provide hydraulic capacity in the interceptor and main treatment plant ($100 + 385 = $485/acre). This means that the collection sewer system would be designed for extraneous flow but the interceptor sewer system would not be so designed. Instead, a storage or treatment device would accept these excess flows. The cost of these revisions in a new area would then be in the order of:

Efforts to reduce inflow at source	$200/acre
Hydraulic capacity in collection system	295/acre
Storage and/or treatment	250/acre
	$745/acre

The approaches of reducing the inflow at the source and yet providing capacity in the collection sewer system for stormwater are somewhat contradictory but, until it has been demonstrated that extraneous flow can be reduced effectively and permanently at the source, it is felt that the problem must be approached in this manner.

The estimated increase in cost of $745/acre represents an approximate increase in sewer service cost of thirty-eight percent in new areas. As experience is gained with control of inflow at the source, the extent of design allowance in the collection system and overflow treatment could be reduced. Nevertheless, control of extraneous flow will be costly. Of course, if overflow of sewage to the river is acceptable, then the cost becomes considerably less.

In existing areas, the solutions are more restricted but a similar two-pronged approach is also recommended. Reduction of inflow at the source in existing areas will be a tedious, unpopular task but could prove highly beneficial. Knowledge of the critical factors that contribute to inflow of stormwater is essential so that those houses with these deficiencies could be selectively corrected. Public education could be the key element in the success or failure of voluntary corrections on private property. If eighty percent of the homes are considered to require re-landscaping the cost would be about eighty percent x $600 = $480/acre. Here again overflow treatment at about $250/acre would be necessary at least where interceptor sewer capacity is limiting, until such time as these extraneous stormwaters are reduced to acceptable levels. The cost estimate to handle extraneous flow in existing areas is then $480 + $250 = $730/acre. Treatment costs could in many cases be reduced because extra interceptor capacity exists at present. This cost estimate is similar to that for new areas. In new areas, the concept of reducing inflow at the source is not so difficult to enact since adequate precautions could be negotiated with the developer rather than with individual homeowners.

The foregoing is not meant to be an exhaustive analysis of all available methods of controlling extraneous flow in sanitary sewer systems. There are undoubtedly numerous other methods bearing further study and the alternatives presented require refinement in details. The analysis is meant to provide some direction as to where succeeding analysis and field study should be concentrated. The study demonstrates that it is not likely extraneous flow will be eliminated completely at the source. Separate connection of foundation drains to the storm sewers in new areas was shown to be very costly. The simple overflow of excess flows to the receiving streams is not recommended. It is apparent that the collection system itself must be capable of accepting the flows generated from the local service area to prevent basement flooding and yet it has been shown it is not practical economically to provide hydraulic capacity in the interceptor sewer and main treatment facilities for extraneous flow peaks. The solution then resolves into reducing the amounts of extraneous flow and determining the best method of storage or treatment of extraneous flow at the interface of the collection and interceptor sewers.

G. Additional Testing Requirements

A review of the available information on the nature and extent of the extraneous flow in sanitary sewers and an attempt to evaluate methods of controlling this flow focuses attention on the need for additional testing. Quite simply, not enough is known to analyze the problem properly. It has been found that the characteristics of the native soil is a major factor in determining the extent to which foundation drains will intercept rainwater percolation in the soil. Therefore local study in this specific area is necessary to understand the particular local circumstances of the problem.

A first priority in a test program would be to determine the extraneous flow from individual foundation drains. This could be accomplished by the installation of calibrated pumps in the trap of the basement drains of individual homes and monitoring the pumping time during rainstorms. Also, it would be necessary to determine what factors contribute to this foundation drain flow. Dye tests could be used to confirm that roof drainage returning to the foundation walls due to poor lot drainage is a major problem. Related areas requiring investigation are the effectiveness of splash pad extensions, side swales, rear lot drainage, and the quality of workmanship in the installation of sewers.

It would be very informative if a recent development, where a sincere effort to reduce extraneous flow had been made, were to be monitored in a similar manner to the test area studied herein. This would provide valuable data on the benefits of general area wide measures of improving surface drainage. The costs of these additional measures could then be assessed against their associated benefits. It would be useful if the study area were to be monitored for extraneous flow peaks for an extended period of time in order to collect data on low frequency rainstorms. A point on an interceptor sewer serving a large separate area should be monitored for peak flows to establish the extent of the dampening of peak flows in a large area. The effect of multiple-unit housing on the matter of extraneous flow also bears further examination. In addition, the quality characteristics of the flow in sewers during periods of rainstorms needs further study in order to evaluate better the effect of spills to the river.

All these studies would require considerable time and money, but the problem cannot be ignored. New construction will not mark time pending a leisurely examination of the alternatives. Simply designing around the problem in an arbitrary fashion may involve expenditures on the least effective solutions. The cost of a sound investigation of the problem would be money well spent if it permitted an intelligent selection of the optimum means of solving the problem. Much data can be collected which does not involve a great deal of money. Basement flooding records should be kept faithfully in all areas. The extent of damage during flooding would be very helpful. Observations on the nature of lot drainage during rainstorms likewise could be accumulated. Closer inspection of building sewer construction may be very beneficial. To illustrate, if each house is considered to have an average sixty foot connection to the street lateral sewer, there are approximately 42,000 feet of building sewers in the test area compared to a total of about 26,000 feet of lateral sewers. The quality of the building sewer construction can therefore be a critical factor.

A point which cannot be over-emphasized is the matter of public education. The responsibile authorities should not miss any opportunity to explain the problem of extraneous flow and the cost of this problem to the community and to the individual. Any campaign to reduce extraneous flow at the source will involve the homeowner as a partner with the authority. An informed homeowner is an essential prerequisite to obtaining his cooperation. The same comments apply to the housebuilders or developers. These people have a vested interest in the extraneous flow problem and could contribute greatly in a cooperate effort. A concerted team effort is required to develop the best solution to the extraneous flow problem.

CHAPTER 8

SUMMARY AND CONCLUSIONS

It has been recognized in recent years that significant inflow of storm water can occur in separate sanitary sewer systems resulting in flooded basements, overflow of raw sewage and increased operating costs as some of the consequences. Current design practice estimates the required hydraulic capacity of sanitary sewers primarily on the basis of water use and does not consider adequately these extraneous flows. The dry weather and wet weather flow conditions in a typical modern subdivision in the City of Winnipeg were studied in conjunction with rainfall monitoring to evaluate the magnitude and extent of influence of rainfall on the flow of sewage in sanitary sewers. These results were compared with current design practice. On the basis of the results from analysis of this test area, some general conclusions can be drawn. The findings of this study are considered representative of conditions in the City of Winnipeg and should be realized that these conclusions may not apply directly to other areas as the local characteristics of soil type, construction practice, the amount and the duration of rainfall effect the amount of extraneous flow in the sanitary sewers.

Dry Weather Flow

The average dry weather sewage flow in a typical residential area can be estimated with good accuracy on the basis of the per-capita water use in the area with an appropriate deduction for consumptive uses and adding a suitable allowance for ground water infiltration. The observed infiltration in the test area was very close to the local design allowance of 240 US gpad or 0.1 gallons per inch diameter per foot per day in the winter season but the observed filtration in summer, even excluding those days of actual rainfall, was typically three times the design allowance. This infiltration causes unnecessary operating costs but has little effect on the hydraulic capacity of the sewers compared to effect of the extraneous flow during rainfall. The average dry weather flow in the summer period from a typical residential area is about twenty percent higher than the annual average flow due primarily to increased infiltration of ground water.

Peak dry weather flow factors as given by conventional design formula, such as the Harmon equation, overestimate the peak dry weather flow from residential areas. For the test area the actual maximum peak factor observed was 2.2 as compared with theoretical peak factor of 3.4 given by the Harmon equation. This is an important factor in existing design in that it provides an inadvertent allowance for extraneous flow.

A very uniform diurnal pattern of dry weather sewage flow was found to exist in the test area for week-days with a different, but also uniform pattern for the week-end days. The effect of extraneous flow in the sewer could be seen readily by comparison of the hydrograph during rainfall with the appropriate dry weather hydrograph.

Wet Weather Flow

The rainfall during the test interval was similar to the average of a twenty-three year period of record in Winnipeg. Rainfalls of less than 0.04 inches were ignored since no extraneous flow resulted from this small precipitation. Rainfall records permitted classification of storms as to probable frequency of occurrence based on a combination of the amount and the duration of precipitation. An effective duration of at least thirty minutes was used for the classification of rainfall frequencies in order to make allowance for travel time in the sewers.

The wet weather hydrographs show clearly that the sanitary sewer system in the test area was subject to massive intrusion of storm water during even moderate rainfalls. This extraneous storm water was observed at peak flow rates of more than seven times the average dry weather flow rate. In other words, the flow in the sanitary sewer at times consisted of ninety percent stormwater. The extraneous flow in the test area is considered representative of typical residential subdivisions in the City of Winnipeg. The extent of extraneous flow intrusion in the sanitary sewers is far in excess of any previous design allowances and leaves no doubt that the present design of sanitary sewers for dry weather flow conditions is completely incompatible with the actual field conditions. The governing flow factor is the inflow of stormwater. The hydraulic adequacy of the sanitary sewers will be determined by the amount of extraneous flow occurring and the amount of capacity allocated to this flow component. The actual peak flows will have little relationship to water use. Accordingly, in order to achieve an effective sewer system, this extraneous flow must either be prevented from entering the sewers or provisions must be incorporated into the system to cope with this stormwater.

The time lag between the centre of mass of rainfall and the peak extraneous flow in the test area was an average of 2.5 hours. This time was very consistent and did not depend on rainfall. It is probably related to the inlet time characteristics of extraneous flow for this particular system. Comparison of this lengthy time lag to the relatively short transit time of sewage in the sewers itself indicates that the sources of extraneous flow are probably diffused throughout the system and not of great individual magnitude. The evidence both locally and elsewhere strongly indicates that foundation drains are the chief source of extraneous flow and that poor lot drainage is a major contributing factor in providing access of roof runoff to the foundation drain and thence to the sanitary sewers.

The volume of extraneous flow was found to be relatively low. The total volume of extraneous flow was estimated to be about three percent of the total annual dry weather flow produced from the test area. Thus the problem of extraneous flow is a matter of many sources of extraneous flow all discharging flow in a concentrated time interval resulting in extremely high peak flow rates for short time periods. The low volumes suggest that the judicious use of storage in the sewer system could be a viable method of flow control.

Once storm water is admitted into the sanitary sewer, it becomes polluted and the resulting mixture is actually higher in pollutional load than the dry weather sewage because the higher velocities cause scouring and erosion of organic deposition in a sanitary sewer. Thus when overflow to the river is most likely to occur, the quality of the flow in the sewers is at its worst.

Basement flooding in the test area is less than might be expected due partly to the fact that eight inch sewers are the minimum size used in the City of Winnipeg. This provides a significant amount of storage in many parts of the system and is an important factor in reducing hydraulic surcharge of the sewers. The storage in the test area may be higher than normal due to the fact that the actual drainage area is considerably less than originally anticipated in design. It was found that the extraneous flow from at least half the rain storms could be stored in the sewer system itself. In addition, the exceptionally high capacity, relative to the dry weather flow, of the pumping station at the downstream end of the test area helps to keep hydraulic surcharging down although this high-rate pumping merely passes on the problem to the interceptor sewer system. As well, the over-estimate of the peak dry weather flow which governs in typical sewer design has produced an unintentional allowance for extraneous flow. These same factors also mean that basement flooding cannot be correlated directly to the statistical relationships of peak extraneous flows and the probable frequency of rainfall.

Statistical Correlations

Peak factors of extraneous flow showed significant statistical correlation to the probable frequency of occurrence of rain storms using regressional analysis on a parabolic function. For classification of storms as to probable frequency, an effective duration of at least thirty minutes was necessary to allow for travel time of flow in the sewer. The correlation of extraneous flow peak factors to rainfall frequency improved when frequency classifications of storms were based on a duration equal to the average time lag from the centre of mass of rainfall to the time of peak extraneous flow at the downstream end of the sewer system. The volume of extraneous flow was found to be correlated to rainstorm frequency and showed a particularly strong correlation with extraneous flow peak factors.

The risk of basement flooding cannot be correctly assessed by the direct use of the statistical correlations with probable rainfall frequencies for the reasons outlined earlier.

Methods of Coping with Extraneous Flow

The effort to eliminate extraneous flow by improved surfaced drainage around houses is not likely to be completely successful. Some revision in design appears necessary even if diligent care is taken to prevent the access of storm water to sanitary sewers. However the attempt to reduce extraneous flow at the source appears very worthwhile, particularly in new areas, by such measures as improved landscaping, permanent splash pads, extended roof downspouts, so that future allowances for extraneous flow can be reduced. In existing areas, public education is necessary to enlist the help of the homeowners to reduce extraneous flow to acceptable limits.

The total flow in sanitary sewers during rainstorms is of higher pollutional strength than the dry weather flow. The discharge of this raw sewage to receiving streams could be both a public health and aesthetic problem in certain areas. In these days of growing public demand for improved pollution control, the deliberate discharge of raw sewage to the rivers could be against the public will. Further, unless corrective steps are taken to control extraneous flow in sanitary sewers, the full value of separating storm and sanitary sewage will not be gained.

The local collection sewer system must be capable of accepting the total flow produced from the service area while providing a reasonable degree of protection against flooding. It was found that the probability of basement flooding could not be related directly to rainstorm frequency. On the basis of the observed data, it is suggested that an extraneous flow allowance of 1 gpm per single family dwelling be used for future collection system design until it is demonstrated that improvements in surface drainage are likely to be successful on a long-term basis in reducing extraneous flow. Design allowances could be less for multiple-family units. Further it appears that the least expensive method of coping with the extraneous flow collected by the local sewer system is to provide storage and/or treatment at the interface of the collection and interceptor sewers. This is considerably less costly and in fact more practical than providing hydraulic capacity in the entire sewage disposal system for the short and extreme peak extraneous flow rates. The storage could be in the form of holding tanks for subsequent repumping of captured storm water to the main treatment plant or the treatment facility in the form of detention tanks providing sedimentation and disinfection prior to overflow of treated effluent to the receiving stream.

In new areas, these measures could result in an estimated increase of about thirty-eight percent in the cost of providing sewer service to a given area. This estimate is broken down into $200 per acre for improved surface drainage, $295 per acre for provision of hydraulic capacity for stormwater in the collection sewer system, and $250 per acre for storage and/or treatment at the interface of the collection and interceptor sewers for extraneous flow. The approach of reducing the inflow at the source and yet providing capacity in the collection sewer system is somewhat contradictory but until it has been demonstrated that extraneous flow can be reduced effectively at the source, it is felt that the problem must at least temporarily be approached in this manner. If the discharge of raw sewage to the river were to be considered acceptable, the solution would of course be considerably less expensive. The merits of this latter approach relate directly to the value placed on the streams by the public.

Separate connections of foundation drains to the storm sewers in new areas is considered to be an expensive method of handling the extraneous flow problem. Further, there are some doubts that this method would eliminate completely the need to provide for extraneous flow allowance in the sewers and therefore this method is not recommended as a general solution at this time. It may be a justifiable expense for multiple-family unit housing where one connection serves two or more dwelling units.

In existing areas plagued with extraneous flow problems, the proposed approach follows along the same lines as for new districts. An effort should be made to reduce extraneous flow at the source. Where the interceptor capacity is limiting relative to the present wet weather flows, treatment or storage of excessive flows should also be provided while the attempt to reduce extraneous flow at the source is going on. The cost to handle the extraneous flow in existing districts is considered to be similar to that quoted for the new areas, but the task of improving surface drainage in existing areas will be much more difficult. This attempt to improve surface drainage in an existing area depends entirely on the cooperation of the homeowner and it is felt that an informed homeowner is an essential prerequisite to the success of such a program.

Additional Testing Requirements

It is evident that additional testing is required in certain aspects of extraneous flow. The monitoring of the foundation drains from individual houses is recommended. In addition, it would be informative if it could be determined what factors contribute to poor lot drainage. The effectiveness of area-wide measures to improve surface lot drainage and the resulting reduction of extraneous flow should be determined by the measurement of flows during rainstorms from these areas. A well organized public education program should be put into effect to acquaint everybody with the causes and effects of this extraneous flow problem.

General

As a final note, it should be recognized that these conclusions, particularly in regard to the methods of coping with extraneous flow, are based on rudimentary studies and are not intended to be specific solutions but guidelines as to where further examination is needed. This study has demonstrated that effective pollution control must start in the sewer system itself and that revisions in current design and construction practice are needed to cope with the problem of extraneous flow in sanitary sewers. This problem affects the ordinary citizen, the house builder and land developer and all public works authorities. A coordinated and team approach needs to be taken in solving this problem. Hopefully this study will be some help in determining the optimum solution.

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