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STORMWATER RUNOFF ON URBAN AREAS OF STEEP SLOPE

Municipal Environmental Research Laboratory
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STORMWATER RUNOFF ON URBAN AREAS OF STEEP SLOPE

by

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FOREWORD

The US Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

For effective control of water pollution due to storm runoff, a prerequisite is a reliable method to predict the quantity of the runoff. The time distribution of storm runoff depends on the rainfall and physical characteristics of the drainage basin. One important factor is the slope of the basin. Many cities have areas with steep slopes. This publication reports the effect of steep slope on storm runoff.

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ABSTRACT

A research is conducted to investigate the applicability of commonly used urban storm runoff prediction models to drainage basins with steep slopes. The hydraulics of runoff on steep slope areas is first reviewed and its difference from that for mild slope areas is discussed. Next the difficulties in applying commonly used methods to steep slope basins are presented. It appears that most engineers are not aware of the problems associated with runoff from steep slope areas and they do not realize that the numerical results given by the conventionally used methods, if obtainable, may not be reliable. A simple approximate method specifically for steep slope basins is proposed and an example is provided. The example utilizes the data from the Baker Street Drainage Basin in San Francisco.

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FIGURES (Continued)

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ABBREVIATIONS AND SYMBOLS

A = area;

B = water surface width;

C = runoff coefficient; also, coefficient or constant;

D = hydraulic depth = A/B;

d = pipe diameter;

$\mathbb{F}$ = Froude number of flow = $V/\sqrt{gD}$;

g = gravitational acceleration;

h = flow depth;

i = rainfall intensity;

K, K' = pressure distribution correction factors in Eq. 3;

L = length;

N = Kerby's coefficient in Eq. 7;

n = Manning's roughness factor;

Q = discharge;

Q_p = peak discharge;

q = lateral inflow per unit length of circumference σ;

R = hydraulic radius;

$\mathbb{R}$ = Reynolds number of flow = VR/ν;

S = slope;

S_f = friction slope;

S_o = bed slope = $\sin\theta$;

t = time;

t_c = time of concentration;

U_x = x-component of velocity of lateral flow;

V = flow velocity;

$\forall$ = volume;

x = longitudinal direction;

β = momentum flux correction factor;

θ = angle between ground surface or sewer invert and horizontal plane;

ν = kinematic viscosity;

σ = perimeter bounding flow area A; and

ϕ = central angle of water surface in sewer (Fig. 21).

ACKNOWLEDGMENTS

In an investigation on the methods for determination of volumes and flow rates of urban storm water runoff, which led to the publication of the report by Chow and Yen (1976) quoted in Reference, eight selected representative methods were evaluated by using recorded data of the Oakdale Avenue Drainage Basin in Chicago. In checking whether the recorded data of the Baker Street Drainage Basin in San Francisco is suitable to test these methods, it was realized that runoff from steep slope basins is a unique problem which requires further research and deserves a separate study. The recently developed sophisticated urban runoff prediction methods which produce satisfactory results for mild-sloped urban areas have great difficulties when applied to areas with very steep slopes such as the Baker Street Basin. Nonetheless, considerable experiences and insight of the problem of runoff on steep slope basins have been gained through the investigation. The authors would like to thank Messrs. Richard Field and Chi Yuan Fan of the U.S. Environmental Protection Agency for their encouragement to report the experience gained in the Baker Street study and for their granting an extension of the project so that the preparation of this report become possible.

Many people have directly or indirectly contributed to the work reported in this publication. The staff of the Division of Sanitary Engineering of the Bureau of Engineering of the Department of Public Works, City and County of San Francisco, particularly Mr. Robert T. Cockburn, are extremely cooperative in providing data and information concerning the Baker Street Drainage Basin. Mr. W. O. Maddaus of Engineering-Science, Inc. was also very helpful in providing data and information. Their help is sincerely appreciated. The authors would also like to thank Mrs. Norma Barton and Miss Hazel Dillman for their typing of the manuscript.

SECTION I

INTRODUCTION

Reflecting the public concern of the urban environment, recently considerable attention and interest have been extended to stormwater control and management. Many methods or "models" of various degrees of sophistication and reliability have been proposed to simulate urban rainstorm water runoff (Chow and Yen, 1976; Brandstetter, 1976; Colyer and Pethick, 1976). A few of these methods have been applied to field conditions. Some have been evaluated using recorded rainfall and runoff data different from the data that were used in developing the methods. Most of them have not been tested beyond the conditions of the original data that were used to develop the methods. Particularly, none of these methods was considered specifically for the case of urban drainage areas of very steep slopes. Since many cities are located along steep banks of rivers and coastal bays and also in mountains, steep slope urban drainage basins are not uncommon. Typical examples are San Francisco, Seattle, and a number of cities along the Mississippi River.

The problem in simulating stormwater runoff from steep slope areas stems from the failure of any existing mathematical model to represent reliably very high velocity gravity flow, i.e., unstable transient supercritical flow with roll waves which hydrodynamically is quite different from subcritical flow. The existing storm runoff simulation methods simply ignore the existence of the high velocity flow. When such a flow indeed occurs, some of these methods produce numerical results without any indication of the physical occurrence of the roll waves and provide no assurance of the reliability of the results which are often misleading if not in error. A few other methods, mostly the more reliable and sophisticated ones, simply break down producing no results because of the combination of numerical and hydrodynamic instabilities.

In a previous investigation Chow and Yen (1976) evaluated eight prediction methods for urban rainstorm water runoff. The eight methods are the rational method, unit hydrograph method, Chicago hydrograph method, British Transport and Road Research Laboratory method, University of Cincinnati Urban Runoff method, Dorsch Hydrograph-Volume-Method, EPA Storm Water Management Model, and Illinois Urban Storm Runoff method. The comparison and evaluation of these methods were done by using four recorded hyetographs of the Oakdale Avenue Drainage Basin in Chicago to produce the predicted hydrographs by the individual methods and the results were compared with recorded hydrographs. The Oakdale Drainage Basin has rather flat slopes and the storm runoffs on the ground and in the sewers are all within the subcritical flow regime. However, when these methods were applied to the San Francisco Baker Street Drainage Basin which is a steep slope urban area, considerable difficulties were encountered.

The objectives of this report are (a) to alarm the engineers dealing with the quantity and quality aspects of urban storm runoff the existence of problems in evaluating the runoff from steep slope areas, (b) to point out the sources and conditions of the problems for selected representative methods, thus providing useful information for practicing engineering in their selection of the most appropriate simulation method or methods for their particular cases involving steep slopes; and (c) to suggest a possible alternative method to determine storm runoff from steep slope areas. This report may be considered as a supplement to that by Chow and Yen (1976). The reader is suggested to read Sections IX and X of the latter in order to gain a more balanced view of the problem.

In this report, the physical phenomena of runoff on steep slope areas are described in Section IV. The applicability of the eight selected prediction methods to steep slope areas is discussed in Section V. A steep slope basin, the San Francisco Baker Street Drainage Basin, is described in Section VI. A proposed approximate method together with an example is presented in Section VII. Conclusions and recommendations are given in Sections II and III, respectively. This investigation was accomplished within a rather limited time and budget and hence limited scope. The study is definitely not exhaustive and perhaps it is more appropriately to be viewed as a peer of the state-of-the-art. Considerable research is still needed to advance the technology of this aspect of drainage problems.

SECTION II

SUMMARY AND CONCLUSIONS

The major purpose of this report is to alert the engineers of the possible difficulties that they may encounter when dealing with runoff on steep slope areas. The hydraulics of gravity flow on areas with steep slopes is first reviewed in Section IV and the sources of difficulties are discussed. Physically and mathematically, the difficulties are the combined result of four sources; namely, the incapability of the St. Venant Equations to simulate the highly unsteady nonuniform flow, the numerical instability, the hydrodynamic instability, and the lack of information on flow resistance to unstable supercritical flow with roll waves.

Based on hydraulic considerations, the applicability of eight selected representative urban storm water runoff prediction methods to steep slope areas as well as the associated difficulties for each of them is discussed in Section V. The eight methods, roughly in ascending levels of hydraulic sophistication, are: the rational method, unit hydrograph method, Chicago hydrograph method, University of Cincinnati Urban Runoff method, British Transport and Road Research Laboratory method (also ILLUDAS), Dorsch Hydrograph Volume method, Storm Water Management Model (both the EPA and WRE versions), and Illinois Urban Storm Runoff method. Ironically, it is the less sophisticated methods having low level of hydraulic considerations that can produce numerical results whereas the hydraulically higher level methods have difficulty in providing results. The difficulties normally occur when roll waves of the flow occur. However, it should be cautioned that the results of the hydraulically low level methods which take no consideration on roll waves may be inaccurate or even misleading. Information on the applicability to steep slope basins of other runoff prediction methods that are not specifically discussed in this report can be deduced by identifying the level of hydraulic sophistication of the method of interest to that of one of the eight methods evaluated.

Clearly, there is a lack of research and information on runoff from drainage basins with steep slopes. Meanwhile, in view of the flow travel time, detail and accuracy of available data and other hydraulic and hydrologic considerations, an approximate method that can be used for prediction of runoff from steep slope areas is formulated and proposed in Section VII. An example of the proposed method is also presented by using the data from the San Francisco Baker Street Drainage Basin which is described in Section VI.

SECTION III

RECOMMENDATIONS

Based on the experience and results obtained in this investigation, the following recommendations concerning the determination of stormwater runoff from urban areas with steep slopes are made:

1. Engineers dealing with urban runoff problems should be aware of the differences between runoff from steep slope drainage basins and those from mild slope basins. Particularly, they should understand that the sources of difficulties that would affect the accuracy of the results are different for the two different cases.

2. The methods with low level of hydraulic consideration, when applied to drainage basins with steep slopes, will produce numerical results. However, the adequacy and reliability of these results have not been established. Therefore, they may be considered only as rough approximations. These methods include the rational method (which gives only the peak discharge), Chicago hydrograph method, University of Cincinnati Urban Runoff model, and British Transport and Road Research Laboratory method. The choice of these methods is more or less a matter of convenient and personal preference since they are all rough approximations.

3. The unit hydrograph method is directly applicable to steep slope basins with adequate engineering accuracy provided the unit hydrographs can be reliably established through measured data or synthetic techniques. Therefore, this method is recommended to be used whenever feasible. Nevertheless, the engineer should always keep in mind the limitations such as linearity and time invariance of the basin that are associated with the unit hydrograph theory.

4. Ironically, it is the methods with higher levels of hydraulic consideration, namely the Dorsch Hydrograph-Volume-Method, Storm Water Management Models, and Illinois Urban Storm Runoff method, that would produce unreliable results or no results at all when applied to steep

slope basins and when roll waves occur in the unstable supercritical flow. Much more research has yet to be conducted before these advanced level methods can be modified to produce reliable results for practical uses. At present practicing engineers are recommended not to waste their efforts trying to use these methods to produce useful results for steep slope basins. It should be noted that this recommendation is in reverse of the recommendation made for mild slope basins by Chow and Yen (1976).

5. There is a lack of past investigation and information on runoff from drainage basins with steep slopes. As a result, there is also a lack of understanding of the physical and mathematical aspects of such runoffs. Much effort and research is needed in the future to advance this aspect of urban technology. Particularly, reliable, detailed and coordinated measurements of rainfall and runoff from steep slope basins are urgently needed. These measurements will be used to evaluate the degree of approximation of the hydraulically low level methods which can produce numerical results. They will also provide invaluable information for the improvement of the hydraulically high level methods.

6. The approximate method proposed in Section VII was developed for practical applications specifically to areas with steep slopes. It is not meant to be theoretically sophisticated and its improvement is possible and desirable. The method has not been verified by field data because reliable and adequate measurements of runoff from steep slope areas are not available at present to verify this or any other methods.

7. Many drainage basins have both steep and mild slope areas. Therefore, it is desirable to have a model that can be used in both types of areas and yet sufficiently reliable. One possibility is to integrate the proposed method into the existing simulation models after it is adequately tested. Further research is needed to develop such combined models as well as to develop more reliable models.

SECTION IV

HYDRAULICS OF RUNOFF ON SURFACE WITH STEEP SLOPES

IV-1. MATHEMATICAL REPRESENTATION OF SURFACE RUNOFF

Storm runoff on urban land surface and in partially filled sewers can be described mathematically by a pair of partial differential equations of hyperbolic type commonly known as the St. Venant equations

$$\frac{\partial h}{\partial t} + D\frac{\partial V}{\partial x} + V\frac{\partial h}{\partial x} = \frac{1}{B}\int_{\sigma} q \; d\sigma \tag{1}$$

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial}{\partial x}(h \cos\theta) = g(S_o - S_f) + \frac{1}{A}\int_{\sigma} (U_x - V)q \; d\sigma \tag{2}$$

in which x is the direction of the flow measured along the ground surface or sewer invert (Fig. 1); t is time; A is the flow cross sectional area; B is the width of the water surface; D = A/B is the hydraulic depth; V is the cross sectional average flow velocity; h is the depth of the flow above the ground or invert; θ is the angle between the channel bed and the horizontal; $S_o = \sin\theta$ is the bed slope, S_f is the friction slope; σ is the perimeter bounding A; q is the lateral discharge per unit length of σ having a velocity component U_x along the x-direction when joining or leaving the flow, being positive for inflow (e.g., rainfall) and negative for outflow (e.g., infiltration); and g is the gravitational acceleration. The first equation is the equation of continuity and the second the momentum equation.

The St. Venant equations have often been referred to as complete dynamic wave equations in the sense that they include all the influential terms representing the gradually varied unsteady free surface flow. Unfortunately, many engineers misinterpret the word "complete" as "exact" and subsequently attempt to apply the St. Venant equations to conditions that they are not valid. Large Froude number supercritical open-channel flow on drainage surfaces and sewers with steep slopes is one of such invalid cases.

Rigorously speaking, the St. Venant equations are cross sectional averaged one-dimensional equations and they are not exact (Yen, 1973b,

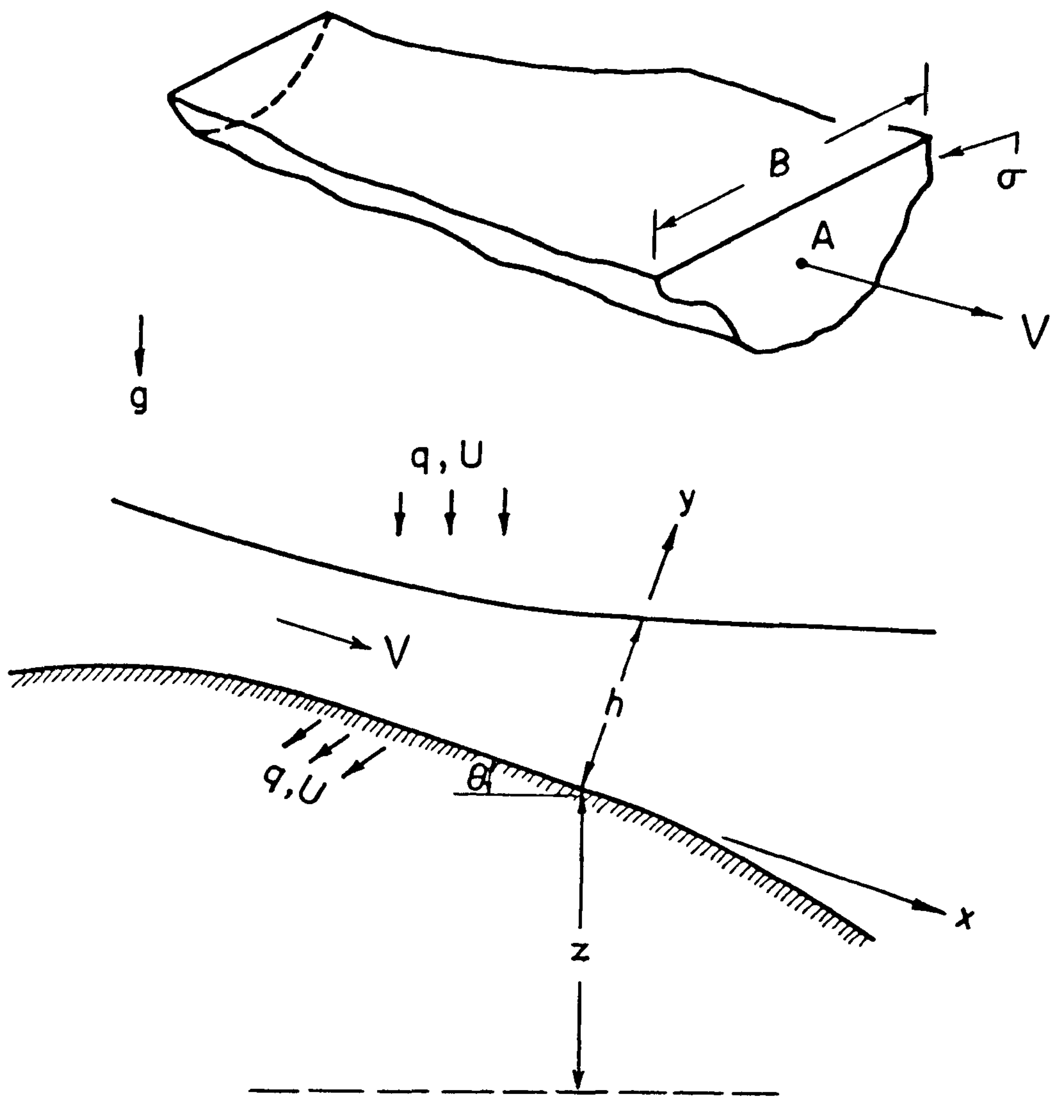


Fig. 1. Definition sketch of open-channel flow

Strelkoff, 1969). In the form of Eqs. 1 and 2 they are applicable to flow of an incompressible homogeneous fluid of constant density and viscosity in an essentially straight, non-deposit and non-erodible, prismatic channel (Yen, 1973b, 1975), and the following additional assumptions are involved in the derivation of Eq. 2:

(a) The pressure distribution over the flow cross sectional area A is hydrostatic.

(b) The x-component of the point velocity is uniformly distributed over A.

(c) The effect of spatial change of internal stresses on A is relatively negligible.

Assumption (c) usually does not cause much error. Assumptions (a) and (b) are valid for gradually varied flows including low Froude number supercritical flow and subcritical flow. However, they are invalid for supercritical flow with the Froude number greater than about two for which roll waves occur. This is the type of flow that occurs for runoff on steep slope areas. In a careful and detailed study on numerical solution of Eqs. 1 and 2 for transient supercritical flows, Zovne (1970) found that the solution becomes unstable when roll waves occur.

In order to eliminate the constraint of Assumption (b), a momentum flux correction factor β should be introduced. For uniform distribution of x-component of velocity on A, $\beta = 1$. To eliminate Assumption (a), two pressure distribution correction factors K and K' are introduced (Yen, 1973b, 1975). Both K and K' are equal to unity for hydrostatic pressure distribution. Accordingly, the more accurate momentum equation is

$$\frac{\partial V}{\partial t} + (2\beta-1)\, V\frac{\partial V}{\partial x} + V^2\,\frac{\partial \beta}{\partial x} + [(\beta-1)V^2 + (K-K')gh\,\cos\theta]\frac{1}{A}\,\frac{\partial A}{\partial x}$$

$$+\, g\,\frac{\partial}{\partial x}\,(Kh\,\cos\theta) = g(S_o - S_f) + \frac{1}{A}\int_\sigma (U_x - V)q\; d\sigma \qquad (3)$$

With the introduction of the correction factors β, K, and K' and allowing them to vary with both space and time, theoretically Eq. 3 together with Eq. 1 can be applied to solve problems of unsteady open-channel flows, including rapidly varied and supercritical flows with high Froude number, provided the friction coefficient S_f can be correctly represented. However, in practice, two major difficulties arise. First, the values of the three correction factors β, K, and K' are known only for special cases (Yen, 1973b, 1975). Their spatial and temporal variations for high Froude number flows with roll waves are at present unknown. Second, currently the knowledge on the friction slope, S_f, is limited to steady uniform flow and a few special cases of unsteady flows. No information exists on S_f for supercritical flow with roll waves. In fact, many engineers do not realize the difference between the friction slope and energy slope, using them indiscriminately and incorrectly in solving problems (Yen, 1973b).

Mathematically, the St. Venant equations, which are a pair of quasi-linear first order partial differential equations of hyperbolic type, cannot be solved analytically, and they are difficult enough to be solved numerically. Only in recent years have satisfactory numerical solution techniques been developed using modern computers (Amein and Fang, 1969; Baltzer and Lai, 1968; Chow and Ben-Zvi, 1973; Dronkers, 1964; Liggett and Woolhiser, 1967; Price, 1974; Sevuk and Yen, 1973; Yevjevich and Barnes, 1970). A method to apply the St. Venant equations to storm sewer networks with appropriate solution techniques has been proposed only recently for subcritical flow and supercritical flow without roll waves (Sevuk et al., 1973). So far there has been no attempt to solve Eqs. 1 and 3 for open channel flow problems. In view of the difficulties in numerical solution techniques and the variations of the correction factors and friction slope, the chance of obtaining an acceptable solution for Eqs. 1 and 3 in the immediate future is rather unlikely. Therefore, it is desirable to identify the various simplified forms of Eqs. 1 and 3 and to evaluate their applicability in solving problems involving runoff on areas of steep slopes.

IV-2. PHYSICAL PHENOMENA OF RUNOFF ON STEEP SLOPES

Physically, stormwater runoff on urban areas of steep slopes is characterized by fast moving water. The water flow is driven by the gravitational force while resisted through the viscosity of the water by the solid boundary at the bottom and air on the surface. Because the air resistance is relatively small and usually negligible, the high velocity water is usually at or near the free surface. When the bottom slope is small, the flow velocity is relatively low because of the small gravitational driving force, and it is slower than the celerity of the wave generated by a disturbance. Such a flow for small slope is called subcritical flow which is characterized by the value of the Froude number, $\mathbb{F}$, less than unity, where

$$\mathbb{F} = \frac{V}{\sqrt{gD}} \tag{4}$$

As the bottom slope increases, the flow velocity also increases as a result of increasing driving force. When the bottom slope is sufficiently steep, the flow velocity exceeds the wave celerity (i.e., the Froude number $\mathbb{F} > 1$) and the flow is called supercritical, analogous to supersonic motion of a fast flying object. In supercritical flow disturbant waves are swept downstream by the fast flow and they cannot travel upstream. In other words, the backwater effect of a supercritical flow can only propagate downstream but not upstream. This fact of no backwater effect from downstream for supercritical provides great advantages in solving mathematically urban storm runoff by using Eqs. 1 and 2 or 1 and 3 because the solution can be sought sewer by sewer or reach by reach of overland in sequence toward downstream instead of simultaneous solution for a number of sewers or reaches as for the case of subcritical flow.

However, complication arises when the bottom slope becomes even steeper. With greater driving force associated with the steeper slope, the water velocity near the free surface flows faster while that at the bottom remains zero, creating an instability condition and roll waves occur which completely changes the flow characteristics and increases energy dissipation. Roll waves occur when the Froude number of the flow exceeds approximately 1.5 to 2, depending on the geometry of the channel, the bed roughness and the unsteadiness of the flow. Unstable supercritical flow with roll waves associated with steep slopes rarely occur in natural streams but it happens often in urban conditions.

Roll wave is a subject that only a few studies have been conducted and only a little information is available. An idealized sketch of roll waves passing down a steep slope is shown in Fig. 2. Iwasa (1954) proposed a stability criterion for quasi-steady, uniform flow in a smooth rectangular

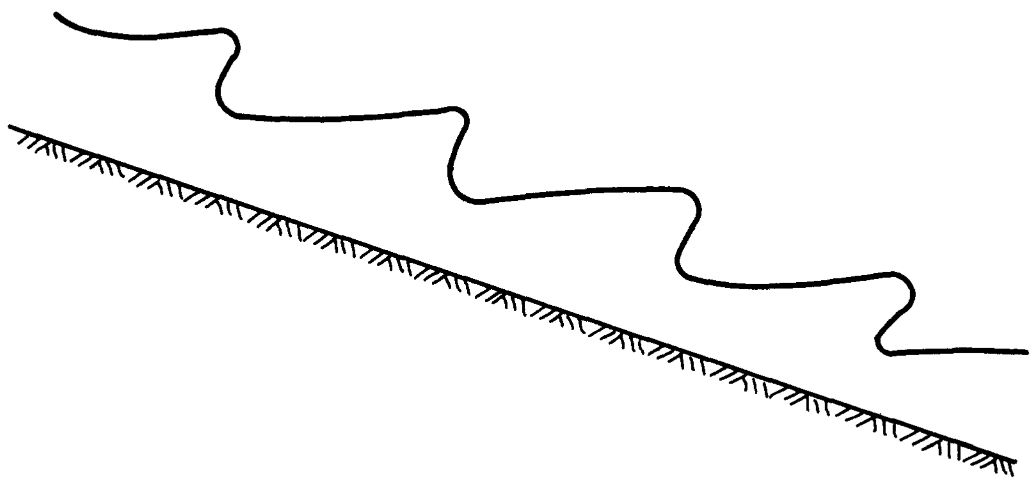


Fig. 2. Sketch of roll waves passing down a steep slope

channel which was experimentally checked by Koloseus and Davidian (1966). Mayer (1961) described the process of generation and propagation of roll waves. The occurrence of roll waves makes Eq. 2 invalid to represent the flow, whereas in Eq. 3, V, A, β, K and K' become highly variable while S_f is modified and unknown at the present.

Urban stormwater runoff is a spatial and temporal varying process and may cover all the six types of open channel flow classified on the basis of relative importance of inertial, viscous and gravity forces. The six types are: subcritical laminar flow, subcritical turbulent flow, supercritical laminar flow without roll waves, supercritical turbulent flow without roll waves, unstable supercritical laminar flow with roll waves, and unstable supercritical turbulent flow with roll waves. A diagram of the depth-velocity ranges for the different types of steady, uniform, wide open channel flow was given by Robertson and Rouse (1941) and is modified to include the roll wave cases as shown in Fig. 3. This diagram can be used only as an approximation for urban storm runoff because the runoff is an unsteady flow and often not with wide homogeneous channels. Nevertheless urban storm runoffs seldom have depth less than one-tenth of an inch (0.3 mm) because of surface unevenness. Therefore, from Fig. 3, it can

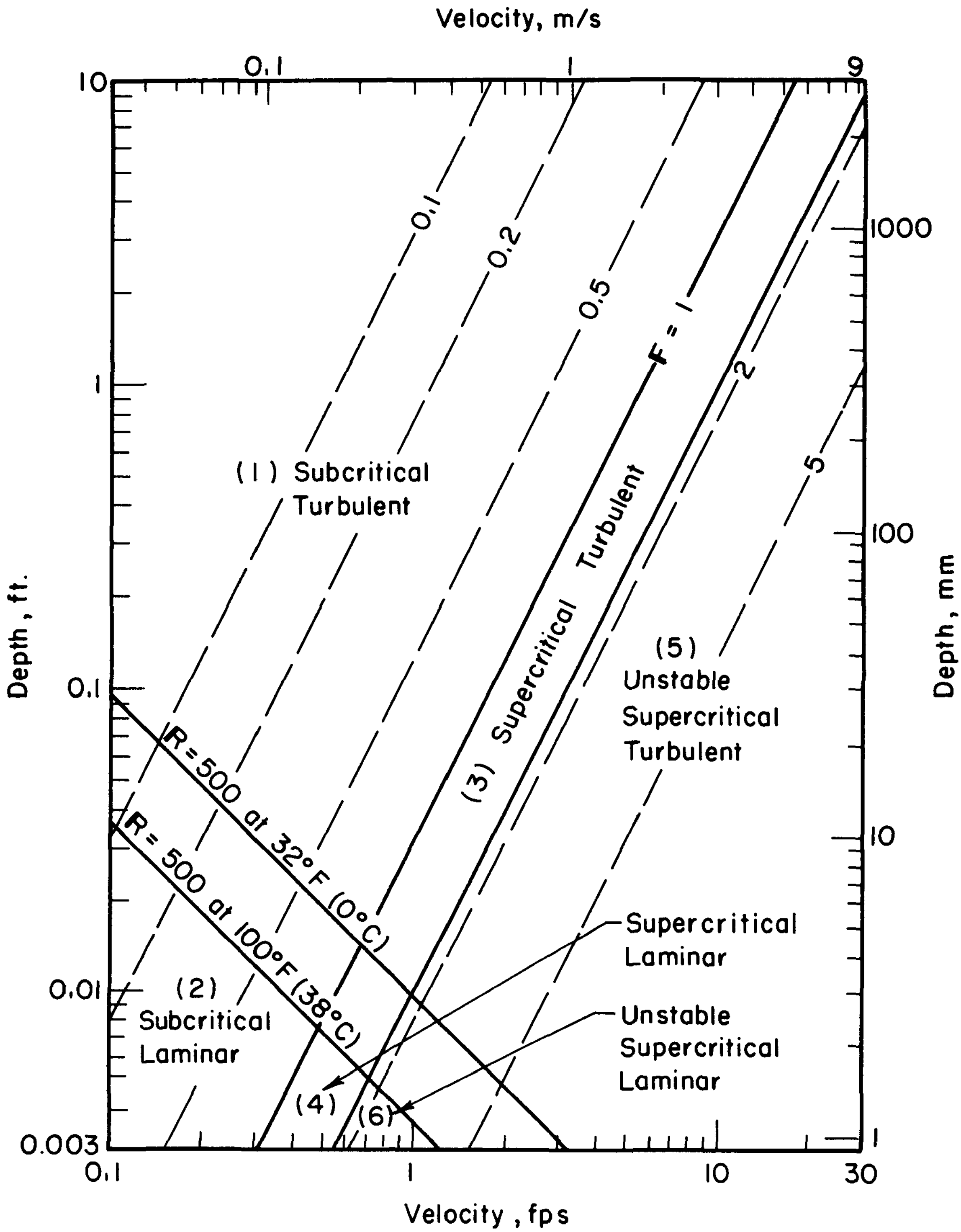


Fig. 3. Depth-velocity diagram for six types of open-channel flows

be postulated that urban storm runoff flow is rarely supercritical laminar (stable or unstable), seldom subcritical laminar, often stable subcritical or supercritical turbulent when the slope is not too steep, and is unstable supercritical turbulent flow with roll waves for steep slopes.

Inevitably any overland runoff due to rainstorm starts with subcritical flow of near zero velocity when the rainfall has satisfied the initial abstractions and overcome the initial retention due to surface depression and surface tension. For a steep slope as the flow velocity increases the flow will soon become supercritical. The transition of a flow from subcritical to supercritical is called a hydraulic drop. This transition from subcritical flow to supercritical flow occurs at different times for different places within the drainage area. In other words, the hydraulic drop is moving.

Likewise, as the rainfall ceases and the runoff is receding, the flow will change back from supercritical to subcritical. This transition is known as a hydraulic jump. Again, the occurrence time of the hydraulic jump is different for different locations and the hydraulic jump is moving. At the same instance there may exist more than one hydraulic jump or drop of different speeds along the channel of runoff. Two hydraulic jumps (or hydraulic drops) when they meet, do not compensate each other. They simply pass each other by and continue on their own propagation.

The existence of moving hydraulic jumps or drops for unsteady flows greatly complicate the solution method of Eqs. 1 and 2 or 3. Information on moving jumps or drops is rather limited. Zovne (1970) proposed an approximate technique to account for them in numerical solution of Eqs. 1 and 2.

It should also be mentioned that for a sewer for heavy rainstorms during the peak flow period the discharge may exceed the sewer capacity and the sewer may become surcharged. The transition from free surface flow to full-conduit flow of the sewer, and later from full-conduit flow back to free surface flow imposes both hydrodynamic and numerical instabilities. Although now the physical and mathematical concepts of these instabilities are clear and reasonably well understood, there exists no numerical solution technique that involves no restrictive assumptions.

In view of the previously discussed mathematical difficulties in obtaining solutions of Eqs. 1 and 3 for unstable transient supercritical flow with roll waves on steep slopes, together with the numerical difficulties in dealing with the transitions between the supercritical and subcritical flows and between the surcharged and free-surface flow sewers, one can only conclude that considerable research has yet to be conducted to develop a reliable solution method. At present from a practical viewpoint it will probably be more fruitful to identify or develop approximate methods (though unavoidably subject to limitations and criticisms) that can be used for solving runoff problems on steep slopes.

SECTION V

DIFFICULTIES IN APPLYING EXISTING RUNOFF PREDICTION METHODS TO STEEP SLOPES

Because of the difficulties and computation costs involved in solving the St. Venant equations, Eqs. 1 and 2, or the more complete form, Eqs. 1 and 3, for urban storm runoff problems, various simplifications of these equations have been used in most of the existing storm runoff prediction methods — many of them were without realizing the assumptions and limitations implicitly imposed. From a hydraulic viewpoint, these simplifications of the St. Venant equations can be classified as shown in Fig. 4 (Yen, 1973a). The physical significance of these simplifications has been discussed elsewhere (e.g., Lighthill and Whitham, 1955; Sevuk, 1973; Zovne, 1970) and is omitted here.

The existing storm runoff prediction methods can be classified in accordance with their respective levels of hydraulic simplifications (Fig. 4). As mentioned in Section I, INTRODUCTION, Chow and Yen (1976) evaluated eight runoff prediction methods which are listed in Table 1 except the rational method. These methods are listed respectively for the overland, gutter, and sewer flows in regard to the simplifications given in Fig. 4. Although many other methods have been proposed recently and appeared in the literature, at present these eight methods can still be regarded as representatives of the existing methods in terms of the various levels of hydraulic sophistications. In this section the applicability of these eight methods to urban areas of steep slopes is evaluated in view of the hydraulic theories. The computational procedures and details of each of these methods can be found in the references cited in Table 1 and in Chow and Yen (1976) or Colyer and Pethick (1976); therefore, they are not repeated here. However, a variation of Storm Water Management Model, the WRE SWMM, is included in the discussion of SWMM because of its recent publicity. A reader who is interested in methods other than those discussed in this section may identify by using Fig. 4 the hydraulic level of his chosen method to one of the eight methods discussed. He then can utilize the evaluations and recommendations in this report accordingly with appropriate modifications, if necessary.

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial}{\partial x}(h\cos\theta) - g(S_o - S_f) = \frac{1}{A}\int_{\sigma}(U_x - V)q\, d\sigma$$

Kinematic wave approximation

Diffusion wave approximation

Quasi-steady dynamic wave approximation

Dynamic wave model

Figure 4. Approximations of St. Venant equations hydraulic routing

All of the sophisticated methods to simulate storm runoff from urban drainage basins consist of two parts: the surface runoff model and the sewer system runoff model. The surface runoff routing techniques of these methods are based on various approximations to the St. Venant equations (Fig. 4) because solution of the complete St. Venant equations is not justifiable due to the computer time required and uncertainties on the resistance coefficient, flow boundaries of the surface runoff and data reliability. Contrarily, the information on sewer systems is usually well defined. Moreover, from a stormwater management viewpoint, errors in surface flow prediction are more tolerable than those in sewer flow prediction. Consequently, almost all of these methods have a more sophisticated scheme for sewer routing than for surface flow routing. For drainage basin with steep slopes, the occurrence of roll waves on street pavements and overland surfaces, though more often seen, is less serious than its occurrence in sewers. Also, for steep slope surfaces, usually the initial abstractions from rainfall are smaller and less important as compared to mild slope surfaces.

Table 1. HYDRAULIC CHARACTERISTICS OF ROUTING OF DISTRIBUTED URBAN STORM RUNOFF MODELS EVALUATE

Model	Surface Flow		Sewer Flow	Selected References
	Overland	Gutter		
Chicago hydrograph	(Linear kinematic wave), Izzard's method	Linear kinematic wave, storage routing with Manning's formula	Linear kinematic wave, storage routing with Manning's formula or time offset method	Tholin and Keifer, 1960
TRRL	— (Flow time-area method)	—	Reservoir routing lagged by time of travel in sewer	Watkins, 1962; 1963; Terstriep and Stall, 1969
UCUR	Manning's formula and empirical detention storage function	Continuing equation of steady spatially varied flow	No routing, lagged by time of travel in sewer	Papadakis and Preul, 1972; Univ. of Cincinnati, 1970
Dorsch	Linear kinematic wave, storage routing with uniform-depth continuity equation and Manning's formula		Simplified diffusion wave with partial backwater effects	Klym et al., 1972 Vogel and Klym, 1973
EPA SWMM	Linear kinematic wave, storage routing with uniform depth continuity equation and Manning's formula	Linear kinematic wave, storage routing with Manning's formula and continuity equation	Improved nonlinear kinematic wave, with partial backwater effects	Metcalf & Eddy, Inc., et al., 1971 Heaney et al., 1973 Huber et al., 1975
WRE SWMM	Same as EPA SWMM	Same as EPA SWMM	Nonlinear dynamic wave, explicit scheme	Kibler et al., 1975
Illinois	Nonlinear kinematic wave, with Darcy-Weisbach's formula	Nonlinear kinematic wave, with Manning's formula	Nonlinear dynamic wave, method of characteristics	Sevuk et al., 1973 Chow and Yen, 1976 Yen et al., 1976

V-1. THE RATIONAL METHOD

The rational method is a black box type lumped system method giving no consideration to either surface or sewer routing. Discussions on application of the rational method for urban storm drainage can be found in Yen et al. (1974), Colyer and Pethick (1976), McPherson (1969), Chow (1964), and Manuals and Reports on Engineering Practice No. 37 of ASCE and WPCF (1969). The rational method gives only the peak discharge, Q_p, but not the runoff hydrograph; i.e.,

$$Q_p = CiA \tag{4}$$

in which C is the runoff coefficient; i is the rainfall intensity, and A is the drainage area. The effects of the surface condition, antecedent moisture condition and slope are adsorbed in the runoff coefficient C. The value of i is assumed equal to the average rainfall intensity over a duration equal to the so-called time of concentration which, for urban area, is equal to the largest of the sum of overland flow time and sewer flow time for the different possible flow paths.

The problems of the rational method, when applied to steep slopes, come from the estimation of C and i, more seriously from the latter than the former. The values of C given in the standard references are empirical values from areas of relatively mild slopes with no or insignificant effects of roll waves. For steep slopes C should be chosen as the higher value of the range for the given condition.

The error due to i for steep slopes comes mainly from the overestimation of the duration of the rainfall, which is due in turn mostly to the overestimation of the overland flow time. Several methods have been proposed for estimation of the overland flow time in urban areas. Izzard's (1946) formula has the form

$$t_c = C_1(i^{1/3} + C_2 i^{-2/3})(L/S)^{1/3} \tag{5}$$

in which t_c is the time of overland flow; L is the length and S the slope of the overland flow path; and C_1 and C_2 are coefficients. This formula is useful only for small areas of less than 1 acre (0.4 ha) in size because of the limitation that the nondimensional value of iL/ν should not exceed approximately 1000 (or $iL<500$ with i in in./hr and L in ft) where L is the length of the flow path and ν is the kinematic viscosity of the water.

A formula to estimate the overland flow time that has been adopted by the U.S. Department of Transportation Federal Aviation Administration (1970) for airport drainage design is

$$t_c = C_3 (1.1-C)L^{1/2}/S^{1/3} \tag{6}$$

in which t_c is in min; C is the runoff coefficient; and $C_3 = 0.39$ when L is in ft and $C_3 = 0.69$ when L is in m.

Another formula proposed by Kerby that is often used for urban overland flow, with t_c in min, is

$$t_c = C_4 (NLS^{-0.5})^{0.467} \tag{7}$$

in which N is a "retardance coefficient" ranging from 0.02 for smooth impervious surface to 0.80 for conifer timberland or dense grass, and assumed dimensionless here. The constant C_4 is equal to 0.83 for L in ft and 1.44 for L in m. The formula is applicable to L less than 1200 ft (365 m).

Equations 5, 6, and 7 are all empirical formulas derived from data from surfaces with relatively mild slopes. They overestimate the flow time when applied to surface with steep slopes. Consequently, using any of these three equations to estimate the rainfall duration for the rational method will result in a lower intensity, and hence a lower Q_p.

By using the kinematic wave approximation (Fig. 4), it is possible to establish a formula to estimate approximately the flow time. Ragan and Duru (1972) proposed that for simple, homogeneous, constant slope overland surface, and assuming constant Manning's roughness factor n, t_c is

$$t_c = C_5 (n^2L^2/S)^{0.3} i^{-0.4} \tag{8}$$

in which t_c is in min, $C_5 = 0.93$ for L in ft and i in in./hr and $C_5 = 6.9$ for L in m and i in mm/hr. Equation 8 provides a more reliable and theoretically reasonable estimate of t_c than Eqs. 5, 6, or 7. Hence it is recommended to use in estimating the overland flow time for the rational method. However, one should realize that Eq. 8 is not without difficulties in applications, and it is accurate only within the limitation of the assumptions involved. For instance, a decision must be made in judging the average condition of the overland surface in order to estimate the representative slope S and Manning's roughness n, especially in the case of sheet flow for which n varies considerably. Also, since the rainfall intensity is a function of the rainfall duration, it may be necessary to find t_c through trial-and-error for successive values of i, which in fact is closer to the value of the rainfall excess averaged over the duration than the corresponding average value of the rainfall.

With the proper estimation of the runoff coefficient and time of concentration, and hence i, the rational method is equally applicable to steep slopes as to flat slopes. It is suggested that Eq. 8 be used for the estimation of the overland flow travel time until a better method becomes available for the estimation. The sewer flow travel time may be approximated by the steady uniform flow velocity of a half full pipe if one is not willing to use any one of the slightly more reliably but more complicated available techniques. It should be mentioned here that for urban storm drainage in estimating the time of concentration normally an accuracy of one or half a minute is sufficiently adequate. It is also suggested that for steep slopes the runoff coefficient should be chosen as the high value over the range for the given surface condition.

V-2. UNIT HYDROGRAPH METHOD

Any black box type lumped system method which takes no specific consideration on how the water flows inside the drainage area would not recognize explicitly whether the drainage area slope is steep or flat. Hence the runoff prediction procedure of such a lumped system method is equally applicable to steep as well as flat slopes provided the appropriate values of the lumped system parameters are used. The rational method discussed in the preceding subsection is an example. The unit hydrograph is another example.

The unit hydrograph of a drainage basin is esentially a reference scale which is obtained through deduction of the past records of rainfall and runoff. In application of this reference scale, the unit hydrograph is simply re-applied to similar conditions to produce the runoff hydrographs for the rainfalls being considered. In other words, the deduced unit hydrograph already accounts for the effect of the slope. Therefore, so long as the rainfall and runoff are within the range that the linearity assumption that is basic to the unit hydrograph theory (Yen et al. 1969, 1973) is valid, the unit hydrograph method is applicable to drainage basins of flat slopes as well as steep slopes, and to basins with simple as well as complicated drainage patterns. However, in many urban areas the physical characteristics of the drainage basin change with time, and there may exist no data to establish the unit hydrograph, subsequently making the unit hydrograph method impractical.

Several techniques have been proposed to synthesize unit hydrographs. One group is to transform the unit hydrographs from gaged to ungaged drainage basins. This approach to establish synthetic unit hydrographs have not been proven successful for urban areas. Contrarily, synthetic unit hydrographs obtained through theoretical or semi-theoretical techniques have proved feasible with limited degree of success. In fact, this concept of synthetic unit hydrographs is adopted in the proposed method for prediction of runoff from steep slope areas that will be described in Section VII.

V-3. CHICAGO HYDROGRAPH METHOD

Any storm runoff simulation model that hydraulically is of the level of kinematic wave or lower is incapable of reflecting the dynamic effect of the flow. Therefore, the model cannot faithfully predict the flow when the dynamic effect becomes important. Lighthill and Whitham (1955) suggested that for simplified cases of unidirectional wide-channel flow the dynamic effect becomes important when the Froude number approaches about 2. This, of course, does not take into consideration of the rapidly varied flow conditions such as hydraulic jumps. Actually, for storm runoff on heterogeneous urban land surface and in sewers, the dynamic effect becomes significant for flow having the Froude number as low as 0.8. However, in the kinematic approximation because the dynamic effect is not reflected, the calculation will produce numerical results which differ from the true solutions. Whether the numerical result can be considered as an acceptable approximation depends on the physical conditions of the flow and the solution accuracy required.

Chicago hydrograph method, by using modified Izzard's (1946) method for overland flow computations and storage routing with Manning's formula for gutter and sewer flow computations (the time-offset procedure is hydraulically much less desirable) is essentially a linear kinematic wave approximation incapable of accounting for the dynamic effect due to steep slopes. Moreover, the method was developed based on the conditions of flat land which is typical in Chicago. Presumably, the linear kinematic wave routing scheme used in the gutter and sewer flows may be used as an approximation for steep slope areas, provided the overland runoff routing is modified and the applicability is verified with field data. Therefore, before such verification is done, the Chicago hydrograph method in its present form is inappropriate to be used for steep slope areas despite the fact that it could produce numerical results.

V-4. TRANSPORT AND ROAD RESEARCH LABORATORY METHOD

The British TRRL method, commonly known as RRL method in the United States, is another hydrograph routing method that hydraulically is of a level lower than the linear kinematic wave approximation. The flow from impervious surface is estimated by a flow-time area method and the sewer flow by reservoir routing with the hydrographs lagged by the time of travel computed based on steady uniform full pipe flow velocity. Since the dynamic effect of the flow cannot be accounted for, there is no computational stability problem when applied to steep slopes and the method will produce numerical results of undetermined reliability. However, the method was developed based on British data of less intense rainfall and relatively mild ground slope with roofs draining directly into sewers. Therefore, it is inappropriate to apply the TRRL method to steep slope areas in the United States without modifications and without sufficient testing to establish its adequacy and reliability.

Based on their experience in applying the TRRL method to rainfall-runoff data from American watersheds, Terstriep and Stall (1974) proposed a model called ILLUDAS which is a modification of the TRRL method. The major differences between the ILLUDAS and TRRL are that the former considers also the runoff contribution from pervious areas by applying the same flow time-area method to both the pervious and impervious areas, and that it uses Manning's formula instead of Colebrook-White formula as in the TRRL method to compute the sewer flow.

Realizing the inadequacy of the TRRL method when applied to outside of the Great Britain and based on their experience in Kenya, the TRRL method developers also proposed a modified version to account for the runoff contribution from pervious areas. They used a linear reservoir model instead of time-area method to represent the runoff from pervious areas. It appears that this linear reservoir model would work well if the abstractions follow an exponential decay function of the type of Horton's infiltration formula.

Neither the ILLUDAS nor the modified version of TRRL method has changed the hydraulic level of the original TRRL method. Therefore, they share the same conclusions as the TRRL method for their applicability to areas of steep slopes.

V-5. UNIVERSITY OF CINCINNATI URBAN RUNOFF MODEL

The University of Cincinnati Urban Runoff (UCUR) model is included here, despite its lack of popularity, merely because it was evaluated previously (Chow and Yen, 1976). Hydraulically it differs from other methods in that it has a higher level and more sophisticated routing scheme for the overland surface flow than for both gutter and sewer flows. The overland flow routing is a simplified kinematic wave approximation whereas the gutter flow is routed simply by using the continuity equation and the sewer flow by lagging the hydrograph without modification.

Being hydraulically a relatively low level method, the UCUR model will give numerical results when applied to steep slope drainage basins. However, the reliability of the results is in doubt. As mentioned in Subsection V-3, for a flow on a steep slope the dynamic effect becomes important and the kinematic wave approximation that is used for overland flow routing may not be adequate. But this inadequacy is relatively insignificant when compared to the gutter flow routing of the UCUR model which uses only the continuity equation of steady spatially varied flow. No geometric or hydraulic characteristics of the gutter are required except the gutter length. For two gutters having different gutter slopes but identical length and overland flow conditions, the UCUR model will give identical inlet hydrographs although in fact the gutter with steeper slope will give a hydrograph with higher and earlier peak discharge.

Hydraulically, the UCUR, TRRL, and Chicago methods are of similar level which will produce numerical results when applied to drainage basins with steep slopes. However, one has to be extremely careful in accepting these results even as an approximation because of their doubtful reliability inherent with the simplicity of the methods. Particularly, the UCUR model is inferior to the other two methods and yet it is much more complicated in application.

V-6. DORSCH HYDROGRAPH-VOLUME-METHOD

The Hydrograph-Volume-Method (HVM) proposed by the German firm, Dorsch Consult, is a method that has been widely misunderstood and misinterpreted in the United States. Similar to the TRRL method, the surface flow is treated without considering separately the overland flow and gutter flow. The surface runoff is routed by using a linear kinematic wave approximation. As discussed previously, the kinematic wave approximation will impose no computational difficulty when applied to steep slope surface but its reliability is questionable.

The sewer flow is routed by using a simplified diffusion wave approximation. The simplification is made through an assumption on the ratio between friction slope and sewer slope (Vogel and Klym, 1973). This assumption substitutes the specification of one of the two boundary conditions required in solving the differential equations. For subcritical flow this eliminates the need of the downstream boundary condition, enabling the solution to proceed towards downstream sewer by sewer in a cascading manner, and consequently it eliminates the need to solve many simultaneous algebraic equations to obtain the solution. It also eliminates the means to realistically account for the downstream backwater effect for subcritical flow. However, for supercritical flow, the two boundary conditions are normally specified at the upstream end of the sewer. Therefore, this assumption on friction slope is merely an unnecessary condition that further jeopardizes the reliability of the solution.

Moreover, for unstable supercritical flow with roll waves, diffusion wave approximation offers no mechanism to realistically account for the dynamic effect of the flow. Therefore, a true nonlinear diffusion wave model will not adequately represent the flow and yet poses considerable computational difficulties. However, in the Dorsch HVM, the assumption on the ratio of friction and channel slopes may have reduced or even eliminated the computational difficulties, and thus provide numerical results. Nevertheless, the significance and accuracy of the computed results, if obtainable, has yet to be evaluated and is not investigated in this study because of the limitation in budget and manpower of this project and the proprietary nature of the method.

V-7. STORM WATER MANAGEMENT MODEL

An evaluation and comparison of the EPA SWMM for mild slope drainage areas has been reported previously (Chow and Yen, 1976). Recently another version of the model, WRE SWMM, has become nonproprietary and available to the public under the support of U.S. EPA. Therefore its applicability to areas of steep slope is also discussed here.

As shown in Table 1, the EPA SWMM uses linear kinematic wave routing for overland and gutter flows and an improved nonlinear kinematic wave routing for sewer flow. Therefore, the model could provide numerical results if the numerical solution scheme and the computational time interval are properly chosen. In the EPA SWMM a four-point implicit finite difference scheme is used in solving the nonlinear kinematic wave approximation to sewer flow. For supercritical flow no iteration is performed (Metcalf & Eddy et al., 1971, Vol. 1, p. 127, Vol. 3, pp. 127 and 128). When the downstream backwater effect is insignificant, the pipe flow may also be approximated by using the linear kinematic wave model of gutter flow (Metcalf & Eddy et al., 1971, Vol. 1, p. 76 and Vol. 3, p. 52). When the sewer is connected at its downstream end to a large storage element from which the backwater effect is important, the water surface is assumed to extend horizontally upstream until it intercepts the sewer invert (Metcalf & Eddy et al., 1971, Vol. 1, pp. 129-132 and Vol. 2, pp. 160-162). The sewer flow above this interception point is calculated by using the kinematic wave approximation. For supercritical sewer flow this backwater assumption precludes the consideration of the formation of hydraulic jump which may actually occur in such flow condition.

Moreover, as discussed in Subsection V-3, the kinematic wave approximation does not account for the dynamic effect of the flow and is inaccurate for supercritical flow with roll waves. Of course, whether an approximation is acceptable depends on the required accuracy. However, it is beyond the scope of the present study to investigate the accuracy of the kinematic wave model to approximate flow in steep slope areas. Since, unlike the subcritical flow and stable supercritical flow cases, theoretical solutions are not yet available for unstable supercritical flow with roll waves, such an accuracy verification can only be accomplished by using laboratory or well controlled field data from small, well defined drainage areas, which unfortunately are also unavailable. The EPA SWMM developers applied the model to the San Francisco Baker Street Drainage Basin (which will be described in the following section) to simulate the runoff from recorded rainstorms (Metcalf & Eddy et al., 1971, Vol. 2, pp. 27-28). The agreement between the computed and measured results is far from satisfactory. It cannot be identified whether the discrepancies are due mainly to the data deficiency or the model inadequacy.

The WRE version of SWMM has identical overland and gutter flow routing schemes as the EPA SWMM. However, the sewer routing is a dynamic wave scheme solving the St. Venant equations using an explicit finite difference scheme. Explicit scheme is easy to formulate, easy to understand, and easy to program. Thus, it is a logical first trial for anyone who is not familiar with numerical solution techniques for the St. Venant equations. Unfortunately, the explicit scheme is also notoriously known for its poor computational stability and efficiency (e.g., see Gunaratnam and Perkins, 1970; Price, 1974; Sevuk and Yen, 1973; Yevjevich and Barnes, 1970). From a practical viewpoint it can be used only for sharp flood waves of short durations which would restrict the applicability of the explicit scheme to a small drainage basin of only a few sewer pipes under a short duration heavy rainfall. Moreover, as discussed in Section IV, the St. Venant equations cannot adequately represent the unstable supercritical flow and stability problems will develop in the solution process. Therefore, WRE SWMM is not applicable to steep slope areas without further restrictive assumptions as in the EPA SWMM.

V-8. ILLINOIS URBAN STORM RUNOFF MODEL

Hydraulically the Illinois Urban Storm Runoff (IUSR) model is probably the most sophisticated of all the existing urban storm runoff prediction models. As summarized in Table 1, the overland runoff is routed by using a nonlinear kinematic wave approximation (Eq. 1 and Fig. 4) with the friction slope S_f computed by using Darcy-Weisbach's formula (Chow and Yen, 1976). This is because for overland sheet flow Manning's roughness factor n is no longer a constant. Gutter flows are routed also by using a nonlinear kinematic approximation but the friction slope is estimated by using Manning's formula with constant n which is simpler than Darcy-Weisbach's and yet provides adequate accuracy under the normal conditions of gutter flows. Similar to the linear kinematic wave approximation that is used in SWMM in routing surface flow, the nonlinear kinematic wave approximations for surface runoff in the IUSR model when applied to steep slopes will produce numerical results but the accuracy has not been verified.

The sewer flows of the IUSR model are routed by using a nonlinear complete dynamic wave model with a first-order method of characteristics numerical scheme. Both the upstream and downstream backwater effects from the sewer junctions are accounted for. The more complicated routing scheme for sewer flows than for surface flows is justified by the fact that in terms of their effects on the accuracy of the sewer outflow hydrographs, approximation of surface flows is more tolerable than that for sewer flows, and that sewer flows have much better definite boundary conditions. Moreover, for combined sewer systems, dry weather flows are confined in the sewers while no surface flow computation is needed. The consideration of backwater effects at junctions and manholes makes a model such as the IUSR method hydrodynamically superior to other sewer routing techniques. The sewer routing technique of the IUSR method is efficient and stable for subcritical and stable supercritical flows. However, when applied to unstable supercritical flow with roll waves, the computation becomes unstable and usually no solution can be obtained. The instability comes both from numerical and hydrodynamic causes. The hydrodynamic instability is particularly serious when the roll waves grow and tend to overtop and break. As mentioned previously the St. Venant equations cannot adequately represent the unstable supercritical flow with roll waves. Conversely, investigation on the possible solution techniques for Eq. 3 together with Eq. 1, which can represent mathematically the supercritical flow with roll waves, has never been conducted.

In this research project the IUSR model was applied to the Baker Street Drainage Basin which is a steep slope basin as will be described in the Section VI. Several futile attempts were made trying to overcome the stability problems to produce some meaningful results. The effort was soon given up because it was a very difficult task far beyond the time and financial limitations of the project. Thus, in its present form the IUSR model is inapplicable to steep slope areas where roll waves would occur.

SECTION VI

EXAMPLE URBAN AREA OF STEEP SLOPE -- BAKER STREET DRAINAGE BASIN IN SAN FRANCISCO

The Baker Street Drainage Basin in San Francisco is selected as an example to illustrate problems involved in storm runoff simulation for urban areas having steep slopes. The basin is located at the north shore east of the Golden Gate Bridge and the Presidio Military Reservation in the City of San Francisco (Fig. 5). Because of the time and financial limitations of this research project, neither an instrumental nor a photographic survey was undertaken to collect the drainage basin physical data. Most of the information presented in this section was obtained from the city maps and drawings provided by the Engineering Bureau of the Department of Public Works of the City and County of San Francisco. A detailed visual survey was also conducted to supplement and to confirm the data.

VI-1. GENERAL BASIN CHARACTERISTICS

The Baker Street Drainage Basin has a combined sewer system, draining a mostly residential area with steep slopes. The outlet of the basin is the diversion structure located at Baker Street and Marina Boulevard at which point dry weather flow (including the sanitary sewage from the Presidio) and runoff from light rainfall is intercepted and transported through the Marina Pumping Station to the North Point Sewage Treatment Plant. The interceptor was designed to carry the dry weather flow plus the runoff from a rainfall of 0.02 in./hr (0.5 mm/hr) with a rational formula runoff coefficient $C = 0.65$. Outflow from the basin exceeding the capacity of the interceptor is discharged directly north into the San Francisco Bay through the outfall structure at about 250 ft offshore at the Outer Marina Beach.

The size of the Baker Street Drainage Basin above the diversion structure as shown by the heavy solid line in Fig. 6 is 180 acres (0.73 km^2).

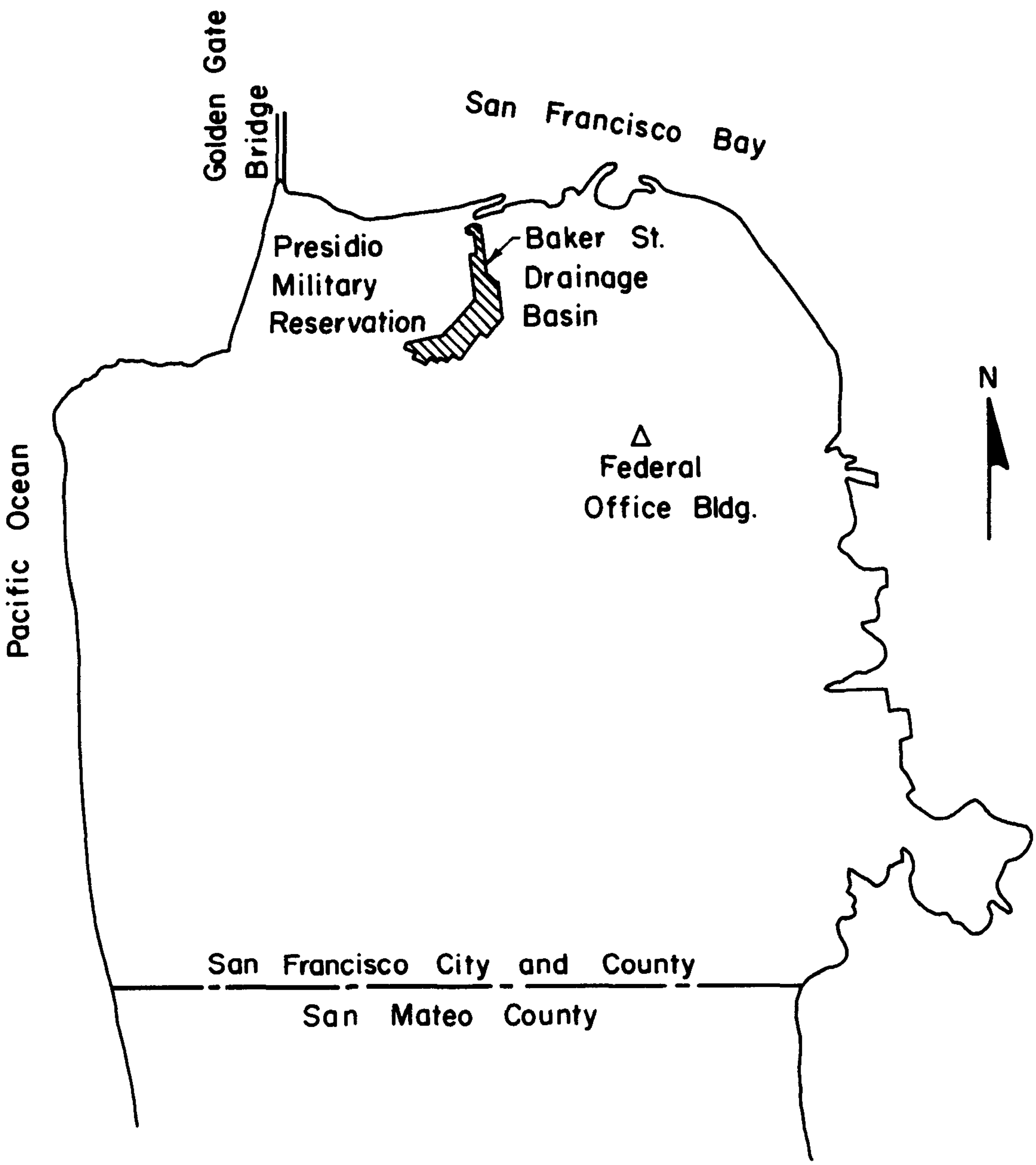


Fig. 5. Location map of Baker Street Drainage Basin

The population within the drainage basin is of the order of eight thousand, excluding that from the Presidio Military Reservation. The basin is actually divided into two parts: an upper basin and a lower basin. The upper (southern) basin is bounded approximately at its east by Lyon Street, north by the earth embankment of the Presidio, west by Cherry and Maple Streets, and south by Clay Street. The size of the upper basin is 68 acres (0.27 km^2), mostly medium to high income dwellings with a significant amount of trees and shrubs. The streets are well maintained with one or more litter boxes in almost every block. The gutters and inlets are reasonably clean. The lower (northern) basin is bounded at its west from the Presidio by Lyon Street, south by Pacific Street, east by Devisadero and Broderick, and north by Jefferson, Marina, Lyon, Baker and Bay as shown by the solid line in Fig. 6. The size of the lower basin above the basin outlet (interceptor) is 112 acres (0.45 km^2), consisting of medium to above medium housing, with some community commercial activities, most of which are on or immediately below Lombard Street. The residential houses in the lower basin have small lawns with little or no shrubs and trees. The streets surrounding the commercial area are generally more littered than the residential area.

The runoff from the upper basin is drained into an egg-shaped sewer off Pacific between Locust and Laurel Streets. The egg-shaped sewer which is buried diagonally in the Presidio, connects the upper basin to the lower basin at Lyon and Union Streets.

The most noticeable physical feature of the basin is its steep slope, particularly in the southern (upper) portion of the lower basin, where in a five-block distance, the elevation rises along Lyon Street from 90 ft at Filbert to 370 ft at Pacific, and along Baker from 60 ft at Filbert to 340 ft at Pacific. In fact the portion of Baker Street between Broadway and Vallejo is so steep (a grade of 35%) that the street has long been closed and grass and weeds are full grown on the surface. The percentage of the total basin area for different ranges of land slope is listed in Table 2 based on the information given in a report by Engineering-Science, Inc. (1971). The topography of the basin is shown in Fig. 6. The numbers at the corners of the blocks are the elevations in feet at the intersections. The highest ground

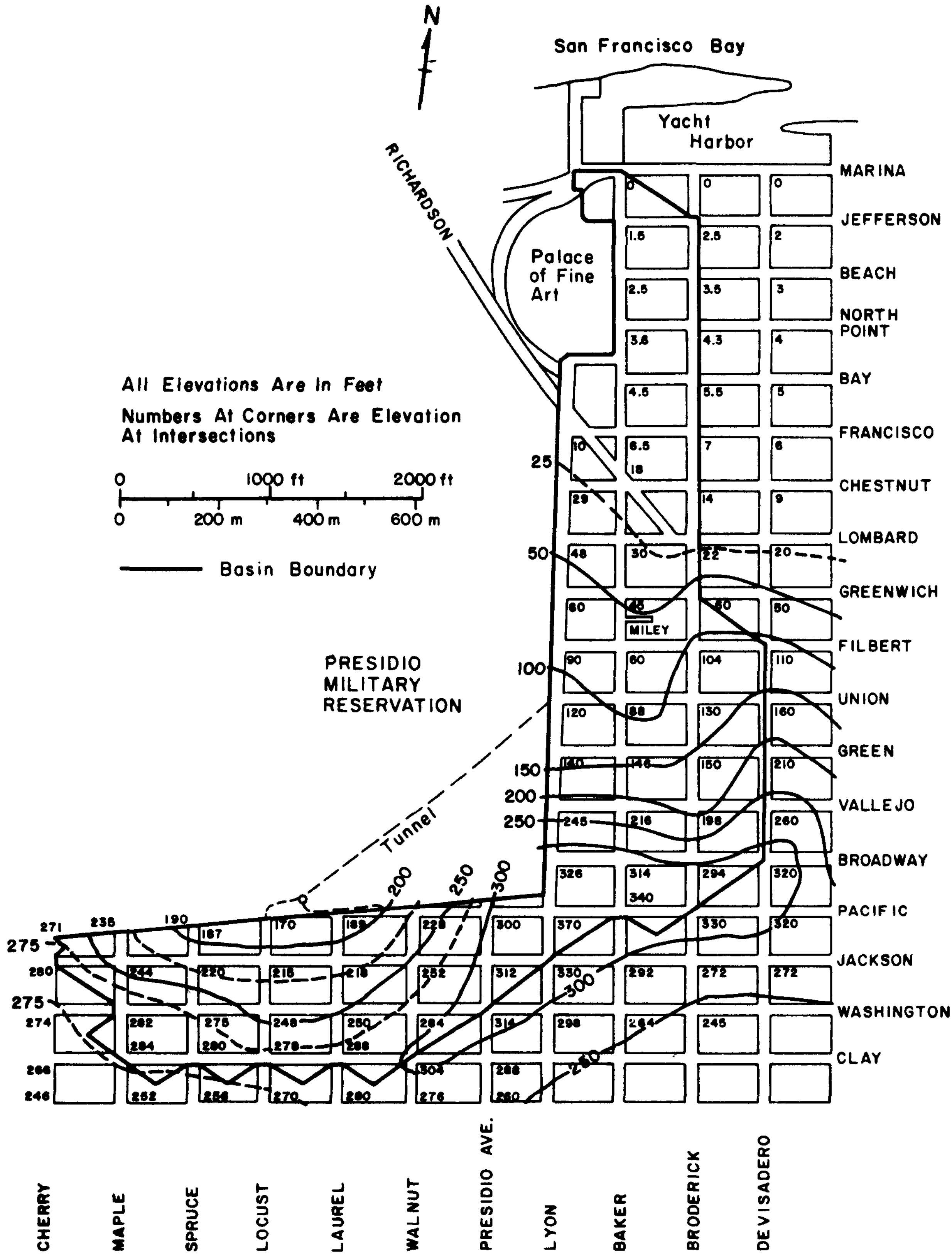


Fig. 6. Topograph of Baker Street Drainage Basin

TABLE 2. LAND SLOPE DISTRIBUTION OF BAKER STREET DRAINAGE BASIN (Engineering-Science, Inc., 1971)

Land slope, %	<2	2-5	5-10	10-15	15-20	>20
Percent of Basin Area	18.1	9.9	33.7	24.7	6.5	7.1

in the basin is 370 ft above the mean sea level at the intersection of Lyon and Pacific which is also the dividing point between the upper and lower basins. Typical photographic views of the basin are shown in Fig. 7.

VI-2. SURFACE DRAINAGE PATTERN

The surface drainage pattern of the land surface of the Baker Street Basin is shown in Fig. 8. In general the rainwater that falls on the land surface is collected through gutters into inlet catch basins which in turn are connected to sewers. There are four types of land surface; namely, roofs, lawns and yards, paved sidewalks, and street pavements. Some of the roof and yard runoffs are drained directly into the combined sewers without flowing through the gutters. The relative percentages of size of the four different types of land surface vary from block to block. Detailed data on the distribution of these four types of surfaces for each block of the Baker Street Basin is unavailable. It has been estimated (Engineering-Science, Inc., 1971) that the land use of the entire drainage basin consists of 80% residential, 8% commercial, the remaining 12% of land being vacant or belonging to governmental agencies, and there is no industry. About 60% of the basin area is impervious, including nearly 9 miles (14 km) of streets.

The standard nominal width of the roadway pavement of most of the streets is 68.8 ft (21 m). The exceptions are Broadway (82.5 ft or 25 m), Marina, Richardson, and most of Lombard, which have a width of about 100 ft (30 m). All the streets are paved with asphalt or concrete. Most streets have gutters and sidewalks on both sides. The sidewalks are paved, usually 9 ft (2.7 m) to 12 ft (3.7 m) wide. As shown in Fig. 7c, many houses in the

(a) Typical view of upper basin -- Jackson Street between Maple and Spruce

(b) Baker Street looking north from Vallejo Street

(c) Typical view of lower basin

Fig. 7. Photographic views of Baker Street Drainage Basin

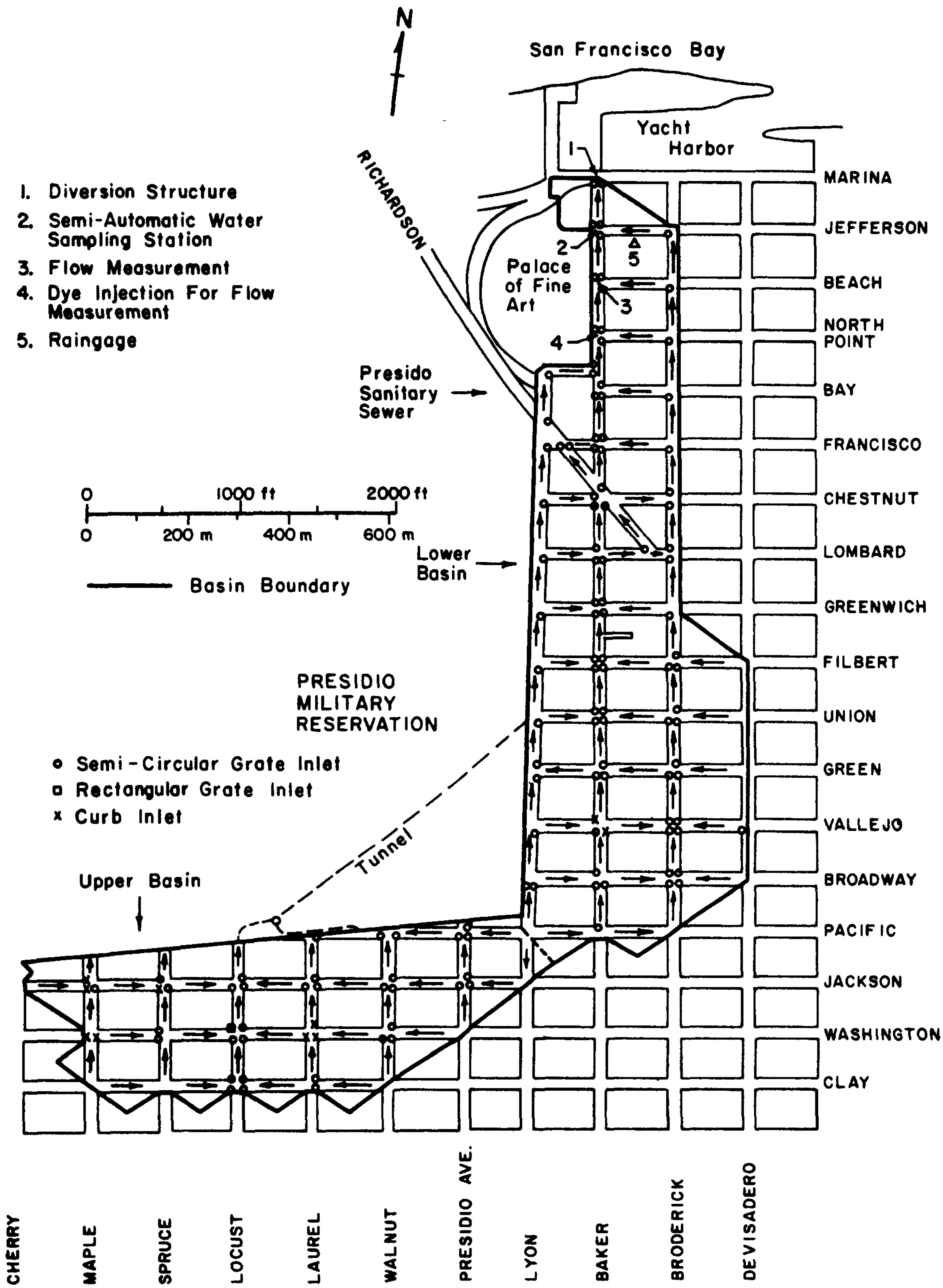


Fig. 8. Baker Street surface drainage pattern

lower basin are built adjacent to the sidewalk without separation by a lawn or yard. In the upper (southern) part of the lower basin and in the upper basin, many houses were built 6 ft (2 m) to 10 ft (3 m) away from the sidewalk, separated by a lawn or paved yard. In general the sizes of the lawns and yards in the Baker Street Basin are much smaller than those in standard American suburban houses. The standard block size excluding the roadway pavement is 412.5 ft (126 m) by 255.4 ft (78 m) in the upper basin and 412.5 ft (126 m) by 275.0 ft (84 m) in the lower basin, respectively. However, blocks adjacent to Presidio Avenue in the upper basin and to Lyon, Broadway, Lombard, and Richardson in the Lower basin differ from the standard sizes.

All the roadway pavements of the streets are crowned, except a few streets which have steep lateral slope so they incline only to one side. According to the standards of the City of San Francisco the crown is 1.0, 0.8 or 0.6 percent of the roadway width between curbs, when the street grade is respectively 0 to 0.03, greater than 0.03 to 0.06, and greater than 0.06.

All the street gutters are formed by the sidewalk curb connected to the roadway pavement. Most of the gutters are approximately triangular in cross sectional shape and with the gutter bottom set to fit the crown or superelevation of the adjacent roadway. The concrete curbs are built with a slope of 1:4 with vertical, and the curb height is 6 in. (15 cm) or 8 in. (20 cm) as shown in Fig. 9. Most of the gutters and curbs in the drainage basin are as those shown in Fig. 10. A typical view of the curb and gutter is illustrated in Fig. 11. (Note that the piece of white paper is 11" x 8-1/2" in size). However, many curbs and gutters are interrupted by the driveway of the houses which is a rather common phenomenon in urban residential areas.

The length and slope of the gutters are listed in Table 3. In the table the streets that have a gutter at only one side or unequal gutter lengths at both sides of the street are specified. Otherwise it is understood that each street has one gutter at each side with identical length and slope. The values of gutter slope are taken from a map supplied by the Bureau of Engineering of the City and County of San Francisco. They

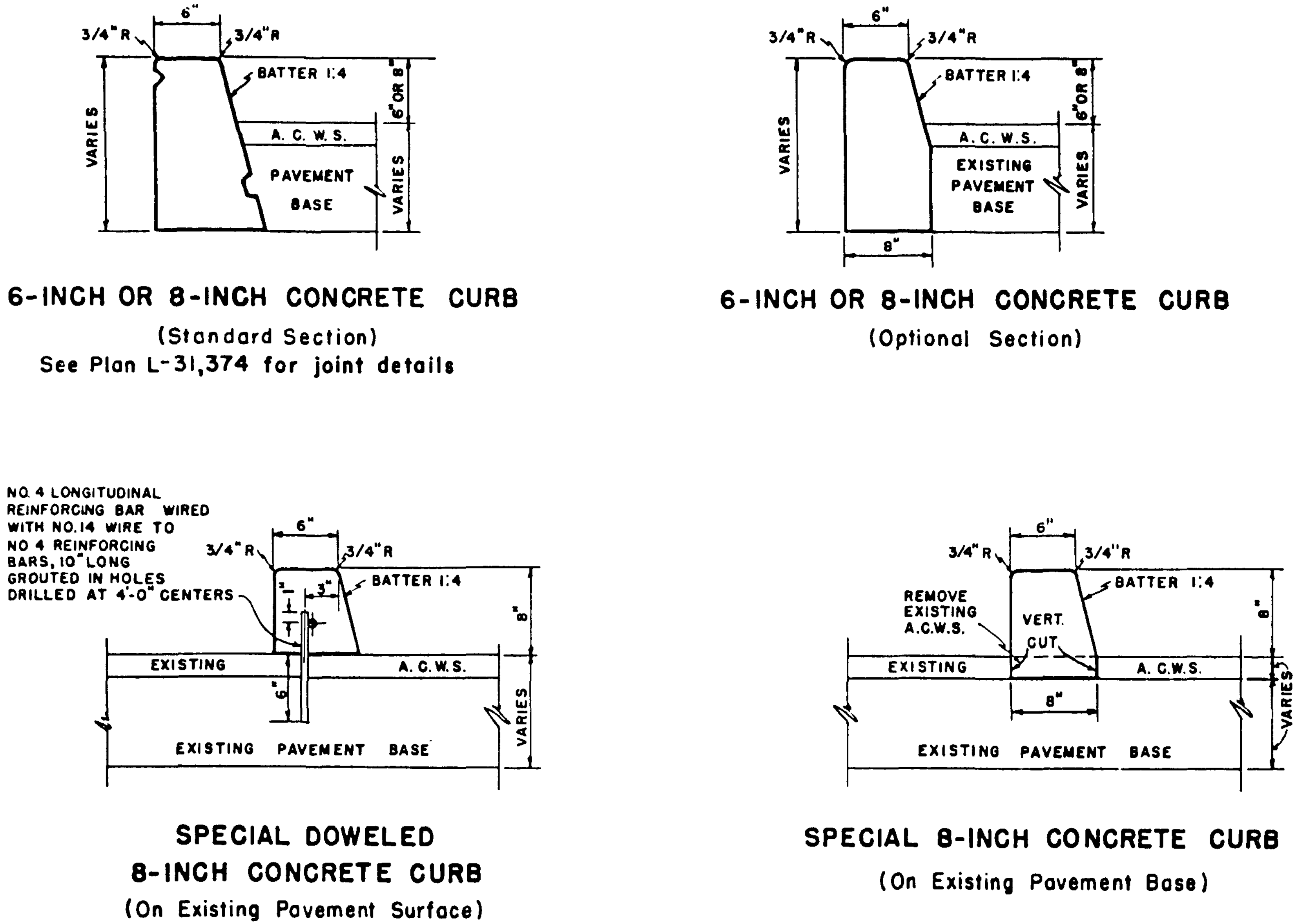


Fig. 9. Details of typical sidewalk curb

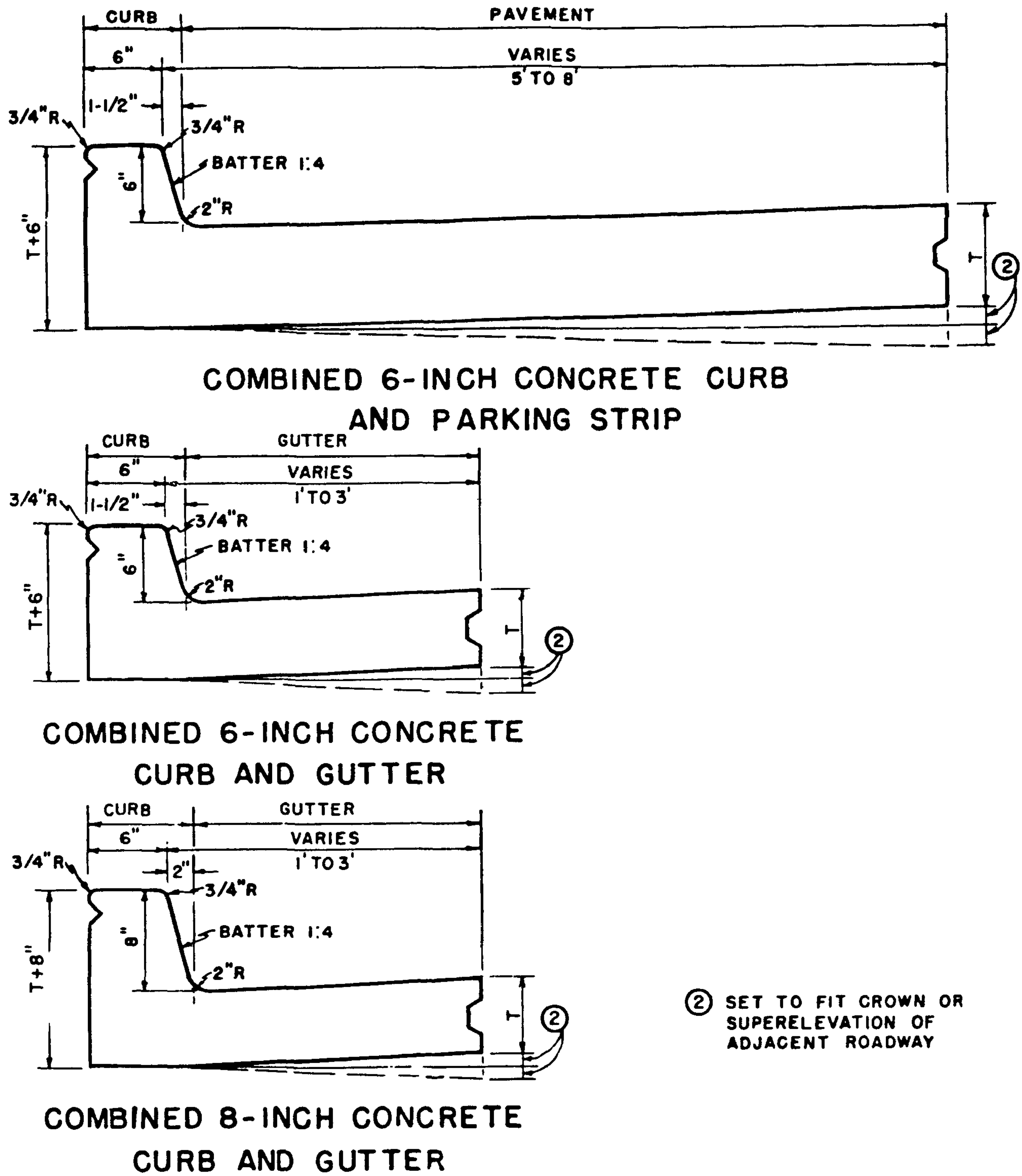


Fig. 10. *Cross sections of typical curb and gutter*

Table 3. BAKER STREET DRAINAGE BASIN SEWER AND GUTTER DATA

Location			Gutter				Sewer						
On street	From street	To street	Length ft	Length m	Longi-tudinal slope	Type of inlet at down-stream	From node	To node	Nominal size	Length ft	Length m	Slope	Shape
Upper Basin													
Maple	Clay	Washington	255	78	0.0078	C-C	284a	281	8"	290	88	0.0062	C
Maple	Washington	Jackson	255	78	0.148	C-S	281	244	12"	324	99	0.117	C
Maple	Jackson	Pacific	140	43	0.073(E) 0.063(W)		243	233	8"	140	43	0.050	C
Spruce	Clay	Washington	255	78	0.019	S-	280a	275	12"	324	99	0.016	C
Spruce	Washington	Jackson	255	78	0.215	C-S	275	221	12"	324	99	0.170	C
Spruce	Jackson	Pacific	190	58	0.168		221	187	15"	230	70	0.145	C
Locust	Clay	Washington	255	78	0.117	S-S	278	248	12"	324	99	0.093	C
Locust	Washington	Jackson	255	78	0.129	S-S	248	215b	15"	324	99	0.102	C
Locust	Jackson	Pacific	210	64	0.196		215b	170	15"	280	85	0.159	C
Locust	Pacific	Tunnel	-	-		-	170	502	2'6"x3'9"	255	78	0.0159	E
Pacific	Maple	Spruce	-	-		-	233	187	8"	480	146	0.096	C
Pacific	Spruce	Locust	-	-			187	170	18"	480	146	0.035	C
Pacific	Laurel	Locust	-	-			186	170	8"	360	110	0.033	C
Clay	Maple	Spruce	412	126	0.0097		284b	280a	8"	440	134	0.0083	C
Clay	Spruce	Locust	412	126	0.0048	S-S	280b	278	8"	440	134	0.0042	C
Clay	Laurel	Locust	412	126	0.024	S-S	287	278	8"	440	134	0.021	C
Clay	Walnut	Laurel	412	126	0.038	S-S	303	288	8"	440	134	0.033	C
Washington	Maple	Spruce	412	126	0.017	S-S	282	275	8"	440	134	0.015	C
Washington	Spruce	Locust	412	126	0.065	R-S	274	248	8"	440	134	0.056	C
Washington	Laurel	Locust	412	126	0.0048	S-S	249	248	8"	440	134	0.0042	C
Washington	Walnut	Laurel	412	126	0.082	C-C	283	250	8"	440	134	0.071	C
Washington	Presidio Av	Walnut	424	129	0.070	S-S	314b	284c	8"	453	138	0.061	C
Jackson	Cherry	Maple	412	126	0.087	C-C	280c	244	8"	440	134	0.075	C
Jackson	Maple	Spruce	412	126	0.058	S-C	244	221	12"	480	146	0.050	C
Jackson	Spruce	Locust	412	126	0.012	-S	220	215b	8"	440	134	0.0104	C
Jackson	Laurel	Locust	412	126	0.0073	S-S	218	215b	8"	440	134	0.0062	C

Type of inlets: C = curb; S = semi-circular grate; R = rectangular grate
First letter refers to northern or western inlet at the intersection, second for south or east inlet.

Sewer shape: C = circular; E = egg shaped

Gutter lengths are for both sides unless marked; (E) = east side gutter, (N) = north side gutter, etc.

Table 3. BAKER STREET DRAINAGE BASIN SEWER AND GUTTER DATA (Cont'd)

Location			Gutter				Sewer						
On street	From street	To street	Length ft	Length m	Longi-tudinal slope	Type of inlet at down-stream	From node	To node	Nominal size	Length ft	Length m	Slope	Shape
Upper Basin													
Jackson	Walnut	Laurel	412	126	0.082	S-S	251	219	8"	440	134	0.071	C
Jackson	Presidio Av	Walnut	424	129	0.141	S-S	311	252	8"	453	138	0.122	C
Jackson	Lyon	Presidio Av	338	103	0.053	S-S	330b	312	12"	405	124	0.044	C
Laurel	Clay	Washington	255	78	0.148	C-C	288	250	12"	324	99	0.117	C
Laurel	Washington	Jackson	255	78	0.125	S-S	250	219	12"	324	99	0.099	C
Laurel	Jackson	Pacific	255	78	0.116	-S	219	188	15"	320	98	0.090	C
Walnut	Clay	Washington	255	78	0.078	S-S	304	284c	8"	290	88	0.062	C
Walnut	Washington	Jackson	255	78	0.125	S-S	284c	252	12"	324	99	0.099	C
Walnut	Jackson	Pacific	255	78	0.098(E) 0.110(W)	S-S	252	228	15"	324	99	0.074	C
Presidio Av	Washington	Jackson	255	78	0.0078	S-S	315	312	8"	290	88	0.0062	C
Presidio Av	Jackson	Pacific	255	78	0.047	S-S	312	300	12"	324	99	0.037	C
Lyon	Pacific	Jackson	255	78	0.156		370d	330b	8"	290	88	0.124	C
Pacific	Lyon	Presidio Av	338	103	0.206	S-S	370c	300	8"	373	114	0.172	C
Pacific	Presidio Av	Walnut	424*	129	0.170	-S	300	228	12"	492	150	0.147	C
Pacific	Walnut	Laurel	412*	126	0.082	-S	228	188	18"	485	148		C
Pacific	Laurel	-	-				188	501	18"	235	71		C
Pacific	-	Tunnel	-				501	502	18"	60	18		C
Tunnel	-	Lyon	-				502	120	2'6"x3'9"	2200	670	0.0157	E
Lower Basin													
Union	Lyon	Baker	355	108	0.090	S-S	120	88	2'x3'	370	113	0.060	E
Baker	Union	Filbert	275	84	0.100	S-S	88	60a	2'x3'	340	104	0.0945	E
Baker	Filbert	Miley	120**	37	0.054		60a	52	2'x3'	170	52	0.0393	E
Baker	Miley	Greenwich	120**	37	0.054	S-S	52	45	2'x3'	170	52	0.0393	E
Baker	Greenwich	Lombard	275	84	0.054	S-S	45	31	2'x3'	340	104	0.0495	E
Baker	Lombard	Chestnut	275	84	0.020	S-S	31	19	2'6"x3'9"	340	104	0.0305	E
Baker	Chestnut	Richardson	220(E)	67	0.035	S	19	17	2'6"x3'9"	30	9	0.0305	E
Baker	Richardson	Francisco	140(W)	43	0.035	S	17	7a	2'6"x3'9"	310	95	0.0305	E

*Gutter only at south side of street

**Length for east side gutter; west side gutter continuous from Filbert to Greenwich, 275 ft long

Table 3. BAKER STREET DRAINAGE BASIN SEWER AND GUTTER DATA (Cont'd)

Location			Gutter				Sewer						
On street	From street	To street	Length ft	Length m	Longitudinal slope	Type of inlet at downstream	From node	To node	Nominal size	Length ft	Length m	Slope	Shape
Lower Basin													
Baker	Francisco	Bay	275(E)	84	0.007	S	7a	5a	60"	340	104	0.0265	C
Baker	Bay	-	410(W)	125	0.006	S	5a	4f	60"	140	43	0.0265	C
Baker	-	Northpoint	275(E) 180(W)	84 55	0.003	S S	4f	4a	60"	200	61	0.0265	C
Baker	Northpoint	Beach	275	84	0.004	S	4a	3a	60"	340	104	0.0265	C
Baker	Beach	Jefferson	275	84	0.003	S	3a	2	60"	340	104	0.0265	C
Baker	Jefferson	Marina	275	84	0.005	S-S	2	0	66"	340	104	0.0265	C
Baker	Pacific	Broadway	255	78	0.098	S-S	340	314a	12"	331	94	0.079	C
Baker	Broadway	Vallejo	275	84	0.356	S-C	314a	216	12"	351	107	0.280	C
Baker	Vallejo	Green	275	84	0.254	S-S	216	146	15"	343	104	0.205	C
Baker	Green	Union	275	84	0.210	S-S	146	88	15"	343	104	0.170	C
Broderick	Pacific	Broadway	255	78	0.135	S-S	330a	294	12"	331	94	0.109	C
Broderick	Broadway	Vallejo	275	84	0.349	S-S	294	198	12"	351	107	0.270	C
Broderick	Vallejo	Green	275	84	0.174	S-S	198	151	15"	343	104	0.140	C
Broderick	Green	Union	275	84	0.072	S-S	151	130	15"	343	104	0.058	C
Broderick	Union	Filbert	275	84	0.094	S-S	130	104	18"	343	104	0.075	C
Broderick	Filbert	Greenwich	275	84	0.160	S-	104	60b	18"	343	104	0.128	C
Broderick	Greenwich	Lombard	260	79	0.126	S-	60b	22	18"	343	104	0.110	C
Broderick	Lombard	Chestnut	275	84	0.023	S-	22	14	2'x3'	340	104	0.023	E
Broderick	Chestnut	Francisco	275	84	0.025	S-	14	7c	2'x3'	340	104	0.020	E
Broderick	Francisco	Bay	275	84	0.005	S-	7b	6b	8"	343	104	0.004	C
Broderick	Bay	Northpoint	275	84	0.003	S-	6a	4e	8"	343	104	0.003	C
Broderick	Northpoint	Beach	275	84	0.003	S-	4d	4c	8"	343	104	0.003	C
Broderick	Beach	Jefferson	275	84	0.003	S-	4b	3b	8"	343	104	0.003	C
Lyon	Pacific	Broadway	255(E)	78	0.145	S-S	370b	326	8"	306	93	0.134	C
Lyon	Broadway	Vallejo	275(E)	84	0.260	-S	325	245	8"	310	94	0.236	C
Lyon	Vallejo	Green	275(E)	84	0.370	-S	190	140	8"	170	52	0.300	C
Lyon	Green	Union	275(E)	84	0.062	-S	140	120	12"	343	104	0.062	C
Lyon	Union	Filbert	275(E)	84	0.109	-S	119	90	8"	310	94	0.090	C
Lyon	Filbert	Greenwich	275(E)	84	0.100	-S	89	60c	8"	310	94	0.087	C
Lyon	Greenwich	Lombard	275(E)	84	0.043	-S	58	48	8"	310	94	0.035	C
Lyon	Lombard	Chestnut	275(E)	84	0.055	-S	47	29	8"	310	94	0.055	C
Lyon	Chestnut	Francisco	275(E)	84	0.072	-S	20	9	8"	310	94	0.055	C
Lyon	Francisco	Bay	290(E)	88	0.010	-S	7d	5b	8"	440	134	0.020	C

Table 3. BAKER STREET DRAINAGE BASIN SEWER AND GUTTER DATA (Cont'd)

Location			Gutter				Sewer						
On street	From street	To street	Length ft	Length m	Longi-tudinal slope	Type of inlet at down-stream	From node	To node	Nominal size	Length ft	Length m	Slope	Shape
Lower Basin													
Pacific	Lyon	Baker	412	126	0.072		370a	340	8"	440	134	0.070	C
Pacific	Baker	Broderick	412	126	0.024		339	330a	8"	440	134	0.021	C
Broadway	Lyon	Baker	391	119	0.030	S-S	326	314a	12"	460	140	0.018	C
Broadway	Baker	Broderick	412	126	0.048	S-S	313	294	8"	440	134	0.047	C
Broadway	Devisadero	Broderick	412	126	0.063	S-S	320	294	8"	440	134	0.060	C
Vallejo	Lyon	Baker	379	115	0.068(N) 0.076(S)	C S	245	216	12"	450	137	0.062	C
Vallejo	Baker	Broderick	412	126	0.043	S-S	215a	198	8"	440	134	0.040	C
Vallejo	Devisadero	Broderick	412	126	0.150	S-S	260	198	8"	440	134	0.140	C
Green	Baker	Lyon	367	112	0.024(N) 0.016(S)	S S	145	140	8"	402	123	0.017	C
Green	Broderick	Baker	412	126	0.009	S-S	150	146	8"	440	134	0.009	C
Green	Devisadero	Broderick	412	126	0.145	S-S	210	151	8"	440	134	0.136	C
Union	Broderick	Baker	412	126	0.101	S-S	129	88	8"	440	134	0.096	C
Union	Devisadero	Broderick	412	126	0.069	S-S	160	130	8"	440	134	0.068	C
Filbert	Lyon	Baker	342	104	0.084	S-S	90	60a	12"	410	125	0.071	C
Filbert	Broderick	Baker	412	126	0.106	S-S	103	60a	8"	440	134	0.100	C
Filbert	Devisadero	Broderick	412	126	0.014	S-S	110	104	8"	440	134	0.013	C
Miley	-	Baker	160	49	0.060		61	52	8"	180	55	0.060	C
Greenwich	Broderick	Baker	412	126	0.036	S-S	59	45	8"	440	134	0.033	C
Greenwich	Lyon	Baker	329	100	0.041	S-S	60c	45	12"	400	122	0.035	C
Lombard	Lyon	Baker	317	96	0.057	S-S	48	31	12"	385	118	0.046	C
Lombard	Baker	Broderick	412	126	0.017	S-S	30	22	8"	440	134	0.017	C
Chestnut	Lyon	Baker	304	93	0.027	S-S	29	19	12"	375	114	0.027	C
Chestnut	Baker	Broderick	360(N) 300(S)	110 91	0.010	S	16	14	8"	350	106	0.010	C
Francisco	Lyon	Richardson	60(S)	18	0.010	S	9	8b	12"	70	21	0.010	C
Francisco	Richardson	Baker	160(N) 110(S)	49 34	0.010	S	8b	7a	12"	290	88	0.010	C
Francisco	Broderick	Baker	412	126	0.002	S-S	7c	7a	2'6"x3'9"	480	146	0.0010	E

Table 3. BAKER STREET DRAINAGE BASIN SEWER AND GUTTER DATA (Concluded)

Location			Gutter				Sewer						
On street	From street	To street	Length		Longi-tudinal slope	Type of inlet at down-stream	From node	To node	Nominal size	Length		Slope	Shape
			ft	m						ft	m		
Lower Basin													
Richardson	Lombard	Baker	330(S)	100	0.012	S	25	19	12"	420	128	0.012	C
Richardson	Lombard	Baker	320(N)	97	0.012	S	24	17	12"	540	165	0.012	C
Richardson	Lyon	Francisco	160(N)	49	0.020	S	8a	8b	12"	100	30	0.020	C
Richardson	Baker	Francisco	160(N) 350(S)	49 106	0.020	S S	18	8b	12"	300	91	0.020	C
Bay	Broderick	Baker	412	126	0.003	S-S	6b	5a	12"	480	146	0.002	C
Bay	Lyon	Baker	280(S)	85	0.002	S-S	5b	4f	12"	350	106	0.002	C
Northpoint	Broderick	Baker	412	126	0.003	S-S	4e	4a	12"	480	146	0.002	C
Beach	Broderick	Baker	412	126	0.003	S-S	4c	3a	12"	480	146	0.002	C
Jefferson	Broderick	Baker	412	126	0.003	S-S	3b	2	12"	480	146	0.002	C
Marina	Lyon	Baker	412	126	0.001	-S	1	0	12"	290	88	0.001	C
Presidio Sanitary Sewer (Bay)		Baker	-				-	5a					

Gutter lengths are for both sides unless marked; (E) = east side gutter, (N) = north side gutter, etc.
Type of inlets: C = curb; S = semi-circular grate; R = rectangular grate
First letter refers to northern or western inlet at the intersection, second for south or east inlet.
Sewer shape: C = circular; E = egg shaped

Fig. 11. Typical view of sidewalk curb -- Broderick Street

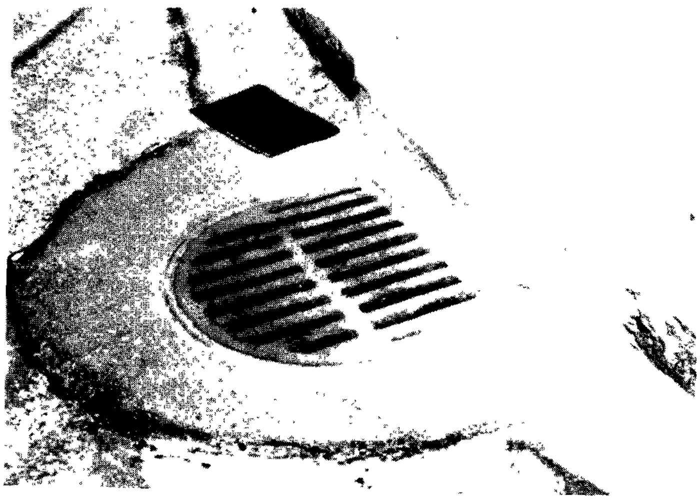


Fig. 12. Semi-circular grate inlet

are approximately equal to the difference between the intersection elevations given in Fig. 6 divided by the corresponding gutter length.

Runoffs in gutters are collected at the downstream end through gutter inlets into catch basins. Three types of inlets are used in the Baker Street Basin: semi-circular grate inlet, rectangular grate inlet, and curb inlet. Most of the inlets are semi-circular grate which is made of steel, consisting of a semi-circle of 2 ft (0.6 m) diameter and a 2 ft (0.6 m) by 1 ft (0.3 m) rectangle as shown in Fig. 12. There is only one rectangular grate inlet in the Baker Basin. The curb inlets are usually 2 ft (0.6 m) long with little or no depression. The type of inlet at the downstream end of each gutter is also given in Table 3. The first letter before the dash identifies the inlet for the north or west gutter of the street, the second letter (after the dash) identifies the inlet for the south or east gutter of the street. The distribution of the inlets are shown in Fig. 8. Also shown by arrows in the figure is the direction of street gutter flow according to the topographic condition.

Obviously, because of the shallow gutters and relatively small inlet openings, on a street with steep longitudinal slope a significant part of the fast moving storm water may flow on the roadway pavement, not getting into the gutters, and bypassing the inlets. It is not difficult to conceive that under heavy rainfall streets such as the southern portion of Broderick, Baker, and Lyon become a channel for the storm runoff. Fortunately the short duration rainfall intensity at San Francisco is much smaller than those at the eastern half of the nation. Hence, the chance of occurrence of having the major part of storm water runoff on surface instead of in the sewers is rather small.

VI-3. SEWER SYSTEM

The layout of the sewer system of the Baker Street Drainage Basin is shown in Fig. 13. The detailed information on the length, size, and slope of the sewers is given in Table 3. The sewer layout and sizes is obtained from a map provided by the Engineering Bureau of the City and County of San Francisco and it differs considerably in both the layout and sizes given in an earlier blueprint map which was also supplied by the Engineering Bureau.

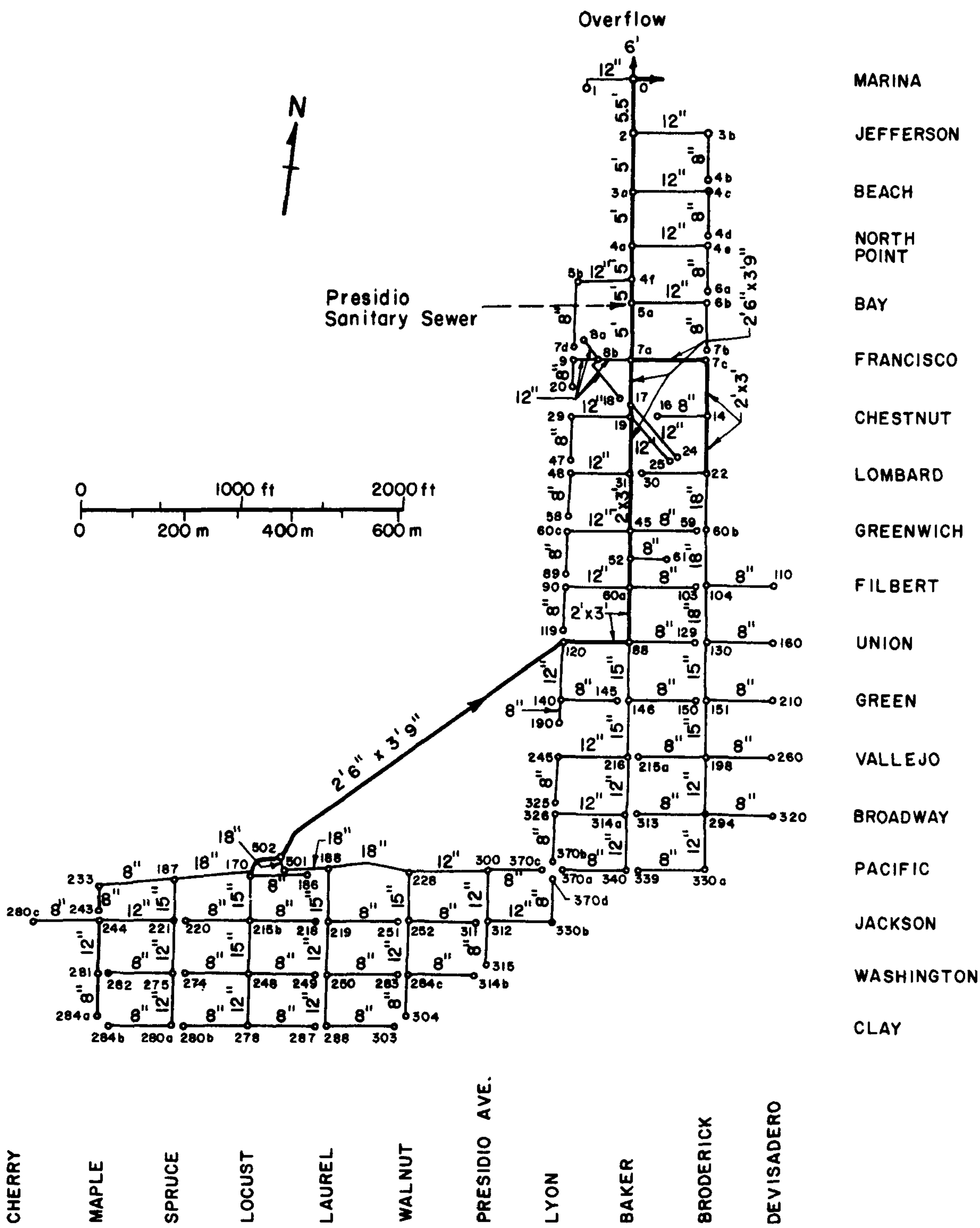


Fig. 13. Baker Street Drainage Basin sewer system

The node numbers used in Fig. 13 and Table 3 to identify the beginning and end of the sewers are approximately the ground elevations at the respective nodes, except Nodes 501 and 502. Different nodes having the same elevation are differentiated by a letter (a,b,c, etc.) following the number. Nearly all of the sewers are buried under the roadway pavement of the corresponding streets.

As shown in Fig. 13, the sewer system can be divided into three parts: the upper basin system, the main-line, and the lower basin system. The main-line starts at Node 170 at the junction of Locust and Pacific, draining the flow from the upper basin northward in a tunnel at the southeast corner of the Presidio Military Reservation to the lower basin at Node 120 at the junction of Lyon and Union, from where it runs eastward under Union for one block and then turns north under Baker Street, heading north to the diversion structure at the junction of Baker and Marina. The Presidio Military Reservation has a separated storm sewer system and supposedly no sewer connection is made to the main-line in the tunnel or anywhere else except at Node 5a at the junction of Baker and Bay where the Presidio sanitary sewer is connected to the Baker Street main-line.

The combined-flow sewers of the main-line are all egg-shape concrete pipes except the last 1720 ft (520 m, 5 blocks) which are circular concrete pipes. A typical cross section of an egg-shape sewer is shown in Fig. 14. The sizes of the egg-shape sewers are 2'-6" x 3'-9" from Node 170 (Locust and Pacific) to Node 120 (Lyon and Union), followed by 2' x 3' size to Node 31 (Baker and Lombard) and then 2'-6" x 3'-9" to Node 7a (Baker and Francisco). For the last five blocks from Node 7a to the diversion structure at Baker and Marina the diameters of the circular concrete pipes are 5 ft (1.5 m) for the first four blocks and 5.5 ft (1.6 m) for the last block. The slope of the main-line sewers obtained from the information supplied by the city of San Francisco is given in Table 3.

The upper basin sewer system can again be divided at Laurel Street into the east and west subsystems as shown in Fig. 13. All the sewers in the upper basin are circular vitrified clay pipes. The sewers in the eastern subsystem most likely have steeper slopes than those in the western

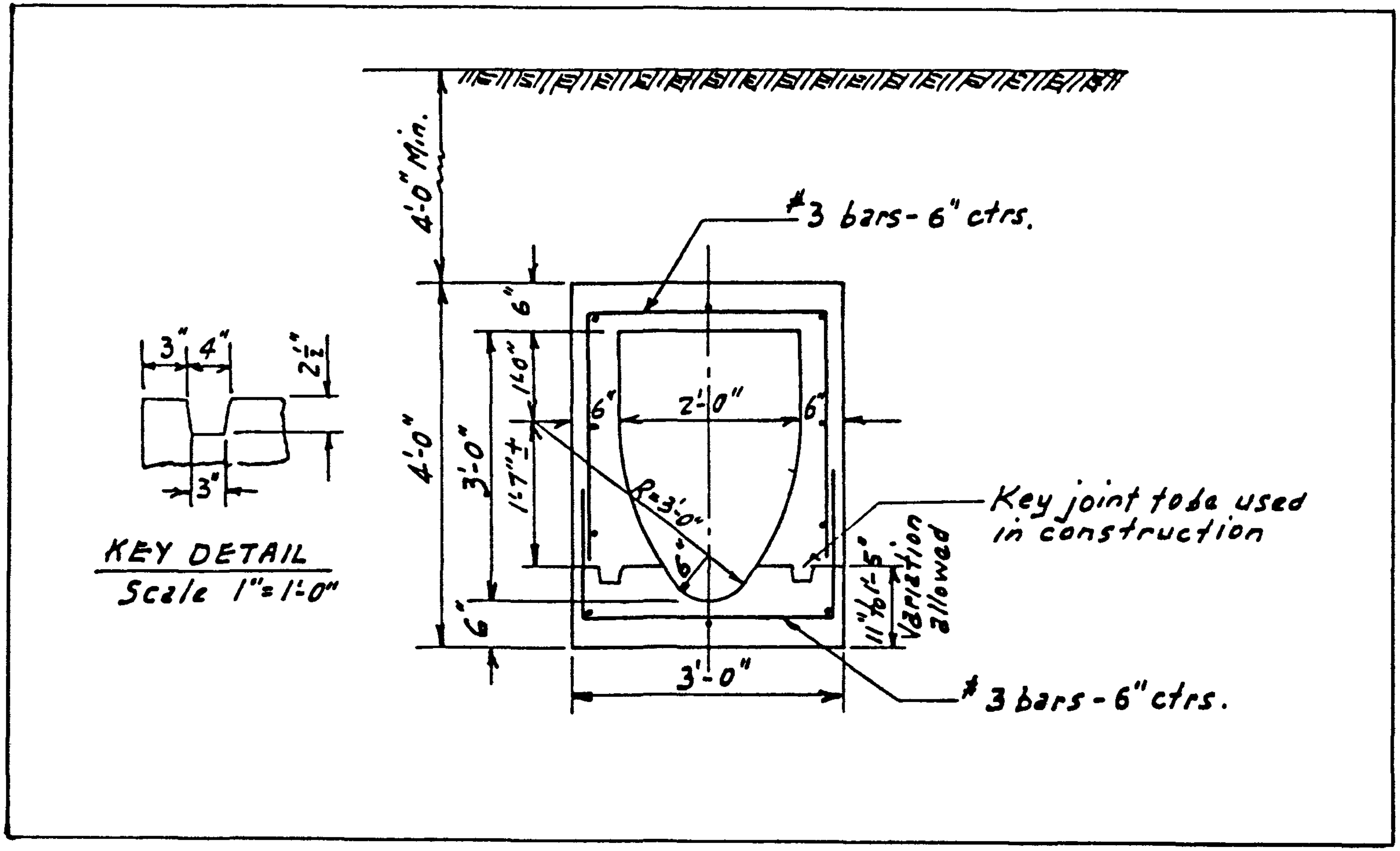


Fig. 14. Typical cross section of egg-shape sewer

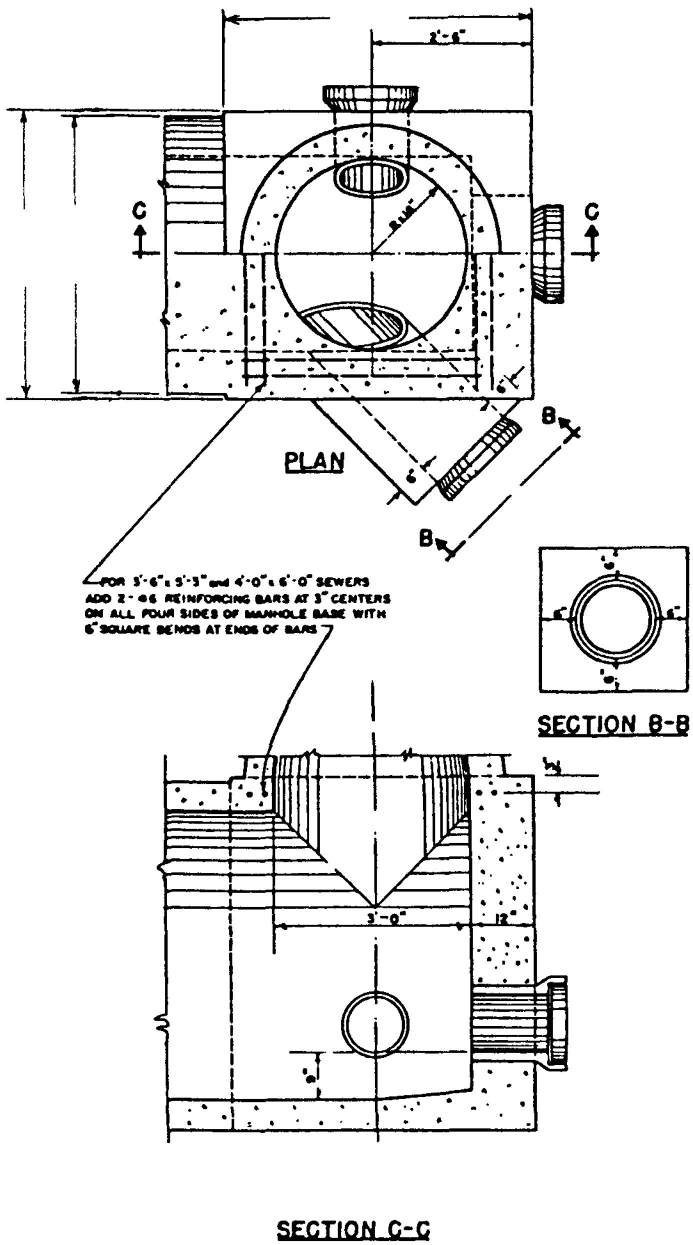


Fig. 15. Typical sewer junction

(a) Circular sewers

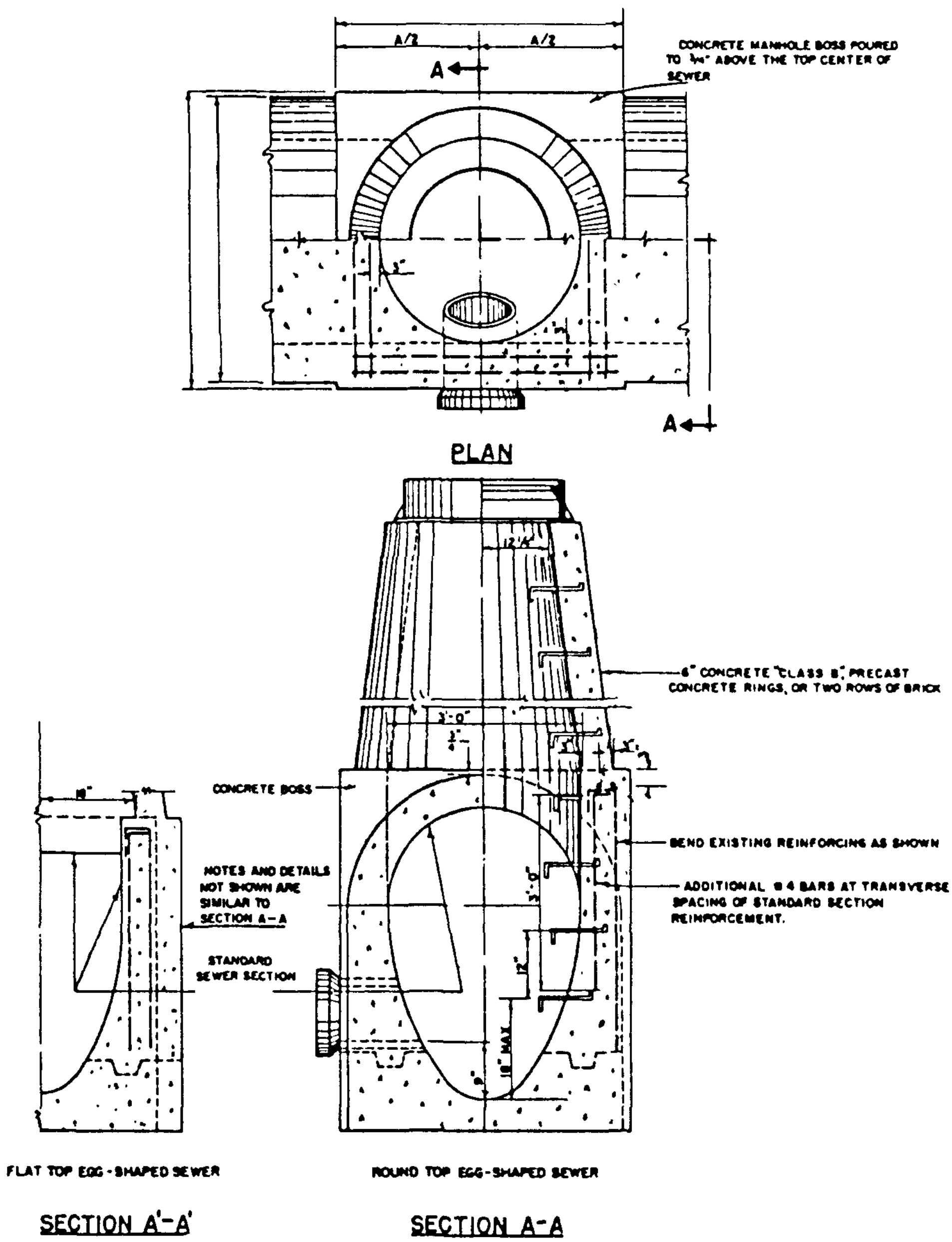


Fig. 15. Typical sewer junction

(b) Egg-shape sewers

subsystem. Since data on sewer slopes (except for the main-line) were not supplied in the information provided by the Engineering Bureau of the City and County of San Francisco, it is assumed that for the sewers in the upper basin as well as those in the lower basin the sewer slopes are equal to the corresponding average ground slopes, which is computed as difference of the elevations at the intersections (given in Fig. 6) divided by the distance between the intersections. The average ground slope is smaller than the corresponding street-gutter slope because of the relatively flat street surface at the intersections. The sewer slopes for both the upper and lower basins are listed in Table 3.

The lower basin consists of two major subsystems and 18 minor subsystems, each subsystem draining directly into the main-line. The two major subsystems are the Baker Street line which starts at the junction of Lyon and Pacific and joins the main-line at the junction of Baker and Union, and the Broderick line which starts at Node 58 (Baker and Pacific) and joins the main-line at Node 7a (Baker and Francisco). In general the sewers in the southern part of the lower basin have steeper slopes than those in the northern part, and those running from south to north have steeper slope than those running in the east-west direction. All the sewers in the lower basin are circular vitrified clay pipes, except the last three sections of the Broderick line, for which from Node 22 (Broderick and Lombard) to Node 7c (Broderick and Francisco) are 2' x 3' egg-shape concrete pipes, and from Node 7c to Node 7a (Baker and Francisco) are 2'-6" x 3'-9" egg-shape concrete pipes.

The sewers receive storm water from connecting upstream sewers and inlet catch basins. There is no unusually large catch basins, manholes, or sewer junctions. Their storage volume is small in comparison to the storm runoff volume when overflow at the diversion structure occurs. Typical junctions for circular and egg-shape sewers are illustrated in Fig. 15.

For the purpose of monitoring the quality of the sewer flow, a semi-automatic water sampling station was installed in the sewer system at the junction of Baker and Jefferson. The drainage area of the basin above this point is 175 ac (0.71 km^2). Sewer flow measurements were made by injecting dye at the junction of Baker and Northpoint and taking the measurements at Baker and Beach. The area of the basin upstream of Node 3a (Baker and Beach) is 171 ac (0.69 km^2).

VI-4. RAINFALL AND RUNOFF DATA

The U.S. National Weather Service has two official precipitation gauging stations near the Baker Street Drainage Basin: one at the Federal Office Building about 1.7 mi (2.5 km) to the southeast, and the other at the Richmond-Sunset Water Pollution Control Plant in the Golden Gate Park about 3 mi (5 km) to the southwest. Good records have been kept for these stations. However, it is well known that considerable spatial variability of the weather exists in the San Francisco Bay region because of the special topographic and climatic characteristics of the area. It has yet to be investigated how reliable the precipitation records of the above mentioned two stations are to represent the rainfall in the Baker Street area. During the 1969-70 winter season the Bureau of Engineering of the City and County of San Francisco installed a network of 13 recording rain gages, one of which is in the lower part of the Baker Street basin (Fig. 6). An analysis of the data from these rain gages and those from the National Weather Service stations confirmed that there exists significant spatial and temporal variations of rainfall within the city.

In 1969 the Bureau of Engineering of the city monitored the runoff from three rainstorms (April 4-5; October 15; and November 5) to investigate the quantity and quality of storm water runoff from the Baker Street Drainage Basin. The data were used in the verification of the Storm Water Management Model (SWMM) (Metcalf & Eddy, Inc. et al., 1971) and also in a project on treatment of combined sewer overflows by the dissolved air floatation process (Engineering-Science, Inc., 1971). The data as reported in the latter report are summarized in Table 4 and the corresponding hyetographs and hydrographs are shown in Figs. 16, 17, and 18 respectively for the three rainstorms. The discharge measurements were made by injecting dye at the junction of Northpoint and Baker Streets and taking concentration samples at the junction of Beach and Baker Streets. The hydrographs shown in Figs. 16 to 18 are the "due to

Table 4. RAINSTORM RUNOFF MONITORED FOR BAKER STREET DRAINAGE BASIN

Date of Rainstorm	Rainfall		Net runoff		Antecedent dry period	Initial[a] rainfall intensity		Initial[a] flow rate	
	in.	mm	$10^3 ft^3$	m^3	days	in./hr	mm/hr	cfs	m^3/s
4-5 April 1969	0.25	6.4	175	4950	1	0.055	1.40	15	0.42
15 October 1969	0.23	5.8	97	2750	<1	0.025	0.64	10	0.28
5 November 1969	0.64	16.3	298	8430	19	0.078	1.98	80	2.24

(a) Initial rainfall intensity and initial flow rate as observed at onset of overflow at diversion structure

Data from Engineering-Science, Inc. (1971)

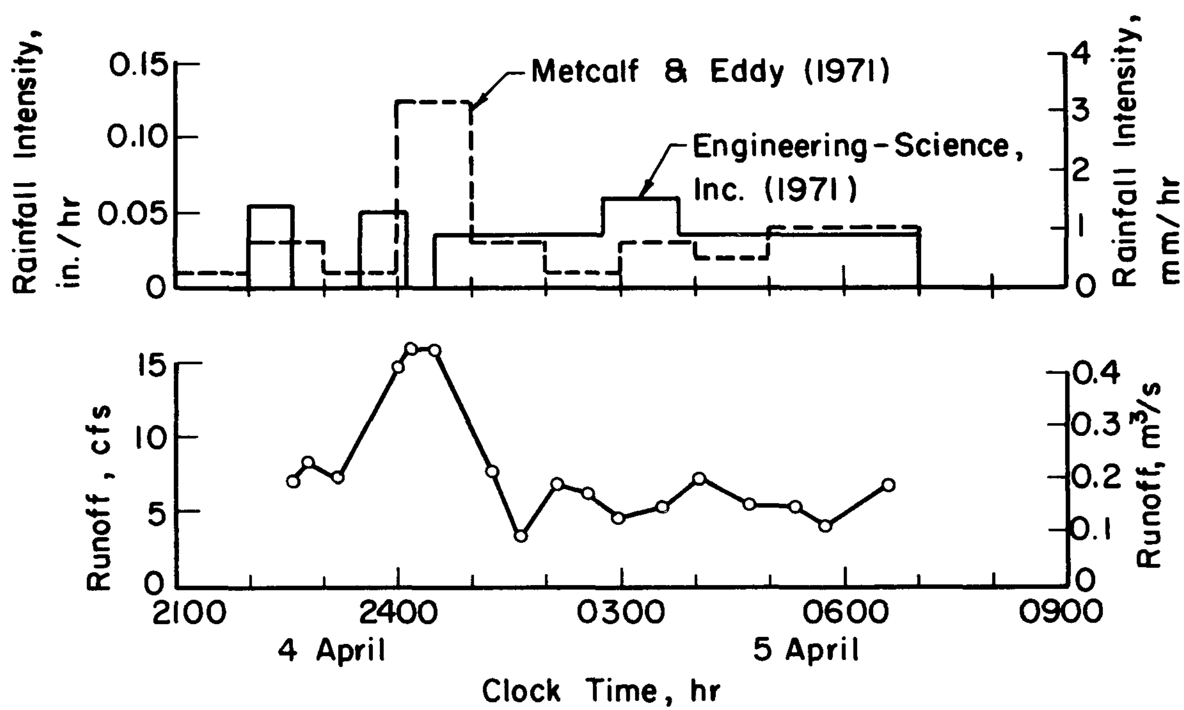


Fig. 16. Hyetograph and hydrograph for rainstorm of 4-5 April, 1969

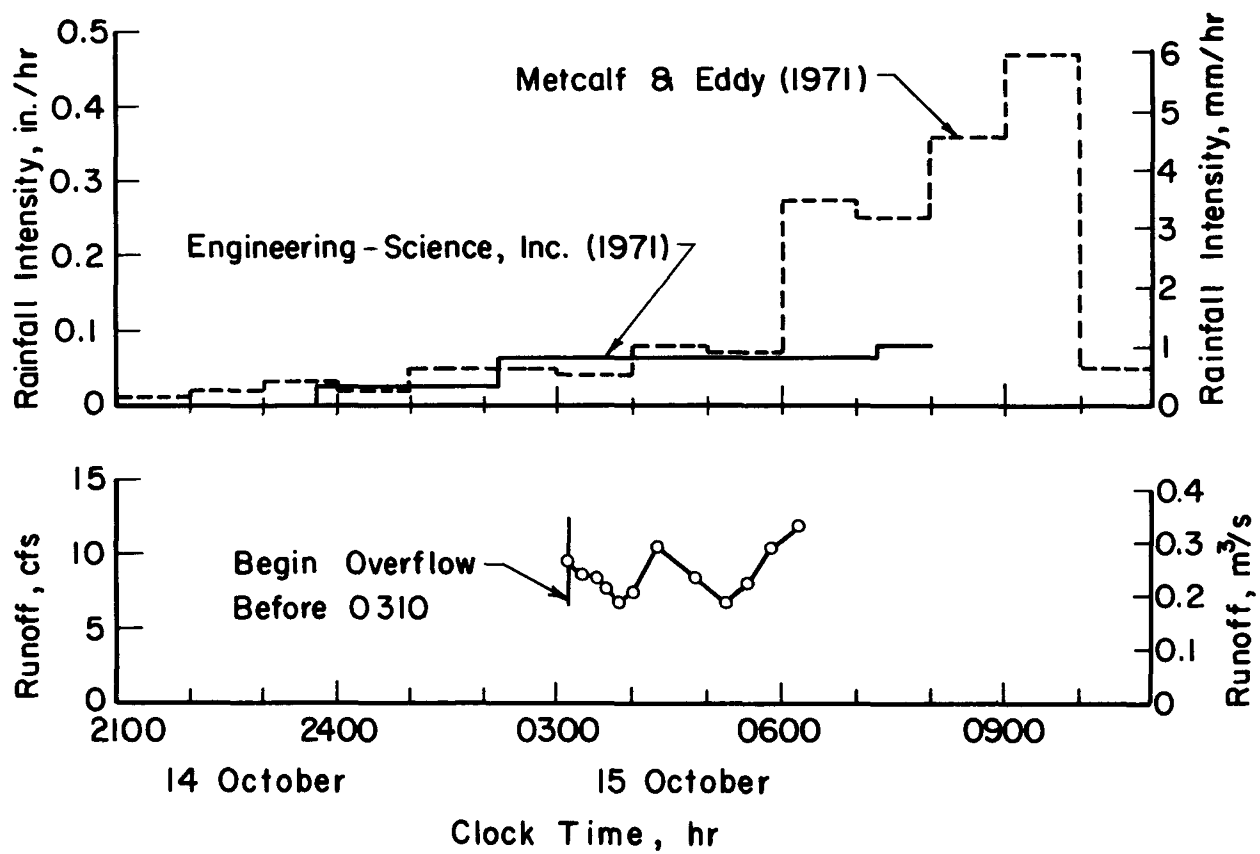


Fig. 17. Hyetograph and hydrograph for rainstorm of 15 October, 1969

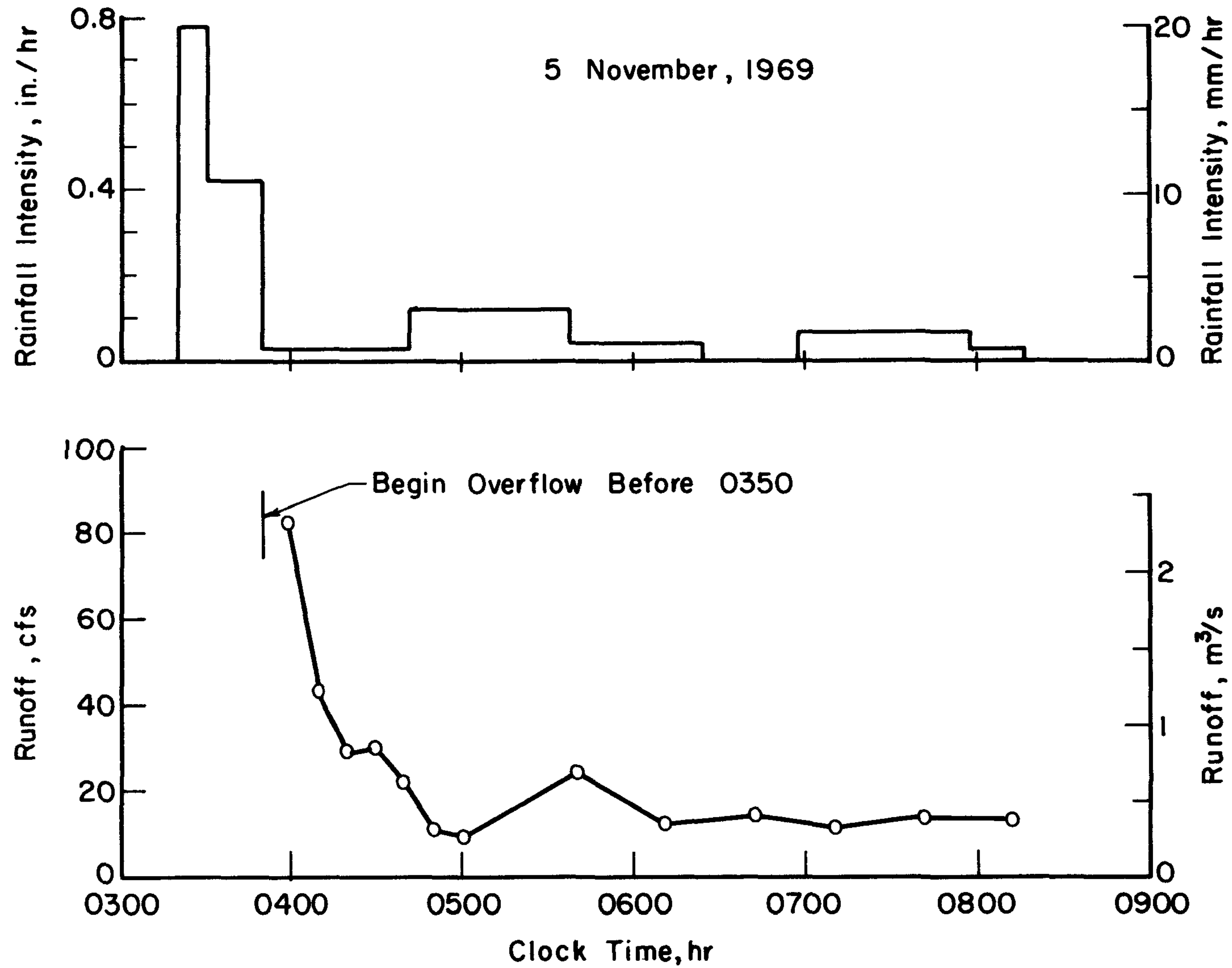


Fig. 18. Hyetograph and hydrograph for rainstorm of 5 November, 1969

storm" hydrographs which are the measured discharges deducted by the average dry weather flow of 2.64 cfs (0.074 m^3/s) which was determined by the dry-weather-flow measurements on 19-20 May, 1969. Runoff water quality samples were taken at the junction of Jefferson and Baker Streets. The pollutographs of COD, BOD, total and volatile suspended solids, grease, floatable materials, and conductivity for the runoff from these three rainstorms are shown in Appendix as Figs. A-1, A-2, A-3 and Table A-1. Detailed information on sampling techniques, dye injection flow measurement and data can be found in the report by Engineering-Science, Inc. (1971).

The rainfall intensity data for the rainstorms of April 4-5, 1969 and October 15, 1969 used in the verification of SWMM (Metcalf & Eddy, Inc. et al., 1971, Vol. 2, pp. 27 and 28) are also plotted in Figs. 16 and 17 since they differ from those presented by Engineering-Science, Inc. (1971). The total amount of rainfall in Fig. 16 is 0.31 in. (7.8 mm) under the hyetograph by Engineering-Science, Inc. and 0.35 in. (8.9 mm) by Metcalf & Eddy Inc. et al., and in Fig. 17 is 0.44 in. (13.4 mm) and 1.73 in. (52.5 mm), respectively. Presumably both sets of hyetographs were derived from the same rainfall depth record at the Federal Office Building gauging station at downtown San Francisco and should agree at least in terms of total rainfall volume. A check of these hyetographs against the official rainfall record at the Federal Office Building was not done in this investigation partly because it was felt that this record is not a true representation for the Baker Street Drainage Basin and partly due to the time and expenses in acquiring the record from the NOAA Environmental Data Service, National Climatic Center.

With the installation of a recording rain gage and a sewer flow level monitor within the drainage basin by the City Bureau of Engineering, a large amount of good quality data on rainfall and runoff becomes available. These data are available from the Division of Sanitary Engineering of the Bureau of Engineering, Department of Public Works, City and County of San Francisco. The rain gage is a tipping bucket automatic recording rain gage and the record is transmitted directly into a digital computer system in the central office of the Bureau of Engineering to record on magnetic tape and printout. The tipping buckets have a standard capacity of 0.01 in. (0.25 mm). During rainfall whenever the bucket tips a signal indicating the tipping time to the nearest second is transmitted to the computer and recorded. A typical printout is shown in Table 5. The rain gage for Baker Street is identified as gage No. 12. The value in the table under this column of No. 12 indicates, to the nearest second, the occurrence of 0.01 in. (0.25 mm) rainfall. For example, the first value, 43, in the column indicates a bucket of rainfall on the 295th day of the year (i.e., October 22), at the time 19 hr 18 min 43 sec. The hyetograph of the rainfall on October 22, 1973 is plotted in Fig. 19 using a 10 min interval. However, it is probably more reliable to obtain the hyetograph by first computing the cumulative rainfall as shown in Fig. 20. The slope of the cumulative curve represents the rainfall intensity and from which the hyetograph can be plotted.

The sewer runoff water level was monitored by an automatic level recording gage at Baker and Francisco Streets. A typical printout of the recorded data is shown in Table 6. The readings were taken at every 15 sec and the number for level indicates water level of flow in inches. For example, the number 3 circled in the table indicates that on Day 295 (October 22) at 18 hr 55 min 15 sec the flow water stage is 3 in. (76 mm) above the reference data. The stage can be converted into discharge if a proper rating curve is available.

Table 5. TYPICAL PRINT OUT OF SAN FRANCISCO RAINFALL DATA

SAN FRANCISCO RAINFALL DATA — 11/ 8/73 PAGE 2

CLOCK TIMES FOR EACH .01 INCH OF RAIN (EXCLUDING ANY TIPS WITH 'Q' TAGS)

DATA FROM DAY 295 MON 10/22/73

* NOTE -- VALUE LISTED IS SECOND *
* WITHIN MINUTE TIP OCCURRED. *

DPW GAUGE NUMBER

DAY	HR	MN	1	2	3	4	5	6	7	8	9	10	11	12	13	14	21	22	23	24	25	26	27	28	29	41	42	43	44	45	46	47
295	19	18		47										43			54			02	37										34	
	19	19				52		07		16	17		10			36		17	31						31							
	19	20	43					59	47			48			38							16	33							04	36	
	19	21								02	48		11			55	45	23	07	00	14											
	19	22		17															48			12	48							48		
	19	23												05	21					42	20	14			13						25	
	19	24						33		09	34	59												37								
	19	25	34	22									09			25	35	11	11				06		22						35	
	19	26				40						54	53	15	54							13								13		
	19	27							07												33		27									
	19	28	33	41				28		06			55	20	46		49			19												
	19	29							51		08	39				08		04	10												04	
	19	30				09		53		11												53	41									
	19	31	03	44						41	23	51	15	01	19			41	36						01					36		
	19	32	22						05			41	51			13	30			01	32										19	
		32				59								59						56												
	19	33						23	54	16	33	39	24		02							23	25									
	19	34	27	15		42		41		33			26	17		31		16	27											26		
		34							50					43																		
	19	35								07		05		02	09										04						11	
		35						36			37		35	21		33	32	33		38												
		35								46		45		39	41						43	47	49									
		35												49										57								
		35												59																		
	19	36												06		09																
		36									14	14		11	15																	
		36										26	23	18																		
		36		33										28				30												32		
		36										37		35																		
		36												40																		
		36											52	47		48																
		36							55					52	55																	
		36									58			57						58												
	19	37										06		02								04										
		37											10	07																		
		37												13					13													
		37										22	21	18																		
		37						23					28	23		24																
		37											37	32	35																	
		37								39		39	42	37																		
		37											48	43								45										
		37	53						55				55	49	55																55	
		37												56																		
	19	38											00	05																		
		38											07	12	09	15		08					09									
		38											20	18	22									20	20							
		38							28				25	25																		
		38											29		33															30		
		38											33	38		36																
		38							40		40		38		41																	
		38				43							42									46										
		38							49				48	54	51	56				50					49							
		38							57						58																	
	19	39							03			04	00																			

Table 6. TYPICAL PRINT OUT OF SEWER FLOW WATER LEVEL RECORD

SAN FRANCISCO SEWER DATA PAGE 4

SEWER MONITOR LEVELS, DAY 295 HR 6 TO DAY 295 HR 24 LISTED 12/13/73

SITE 200. DAY 295, HOUR 18. 10/22/73 MON 6.00 P.M.

MN	LEVEL	MN	LEVEL	MN	LEVEL	MN	LEVEL	MN	LEVEL	MN	LEVEL
00	2	10	1	20	1	30	1	40	1	50	2
	1		1		1		1		1		1
	1		1		1		1		1		1
	1		1		1		1		1		2
01	1	11	2	21	1	31	1	41	1	51	[illegible]
	1		1		1		1		1		2
	1		1		1		1		1		2
	1		1		1		1		1		3
02	1	12	1	22	1	32	1	42	1	52	2
	1		1		1		2		1		2
	1		2		1		1		1		2
	1		1		1		1		1		2
03	1	13	1	23	2	33	1	43	2	53	3
	1		1		1		1		1		3
	2		1		1		1		1		2
	1		1		1		1		1		2
04	1	14	1	24	2	34	1	44	1	54	3
	1		2		1		1		1		2
	2		1		1		1		1		3
	2		1		2		1		1		2
05	1	15	2	25	1	35	1	45	1	55	3
	1		1		1		2		2		3
	1		1		1		1		1		3
	2		1		1		2		1		2
06	2	16	2	26	1	36	1	46	1	56	2
	2		1		1		1		1		2
	1		1		1		1		1		3
	1		2		1		1		1		1
07	1	17	1	27	2	37	1	47	1	57	3
	1		2		1		1		1		2
	2		1		1		1		1		2
	1		1		1		1		1		2
08	2	18	1	28	1	38	1	48	1	58	3
	1		1		1		1		1		3
	1		1		1		1		1		2
	1		2		1		1		1		2
09	1	19	1	29	1	39	1	49	1	59	3
	1		2		1		1		2		3
	2		1		1		1		1		3
	1		1		2		1		2		3

SITE 200. DAY 295, HOUR 19. 10/22/73 MON 7.00 P.M.

MN	LEVEL	MN	LEVEL	MN	LEVEL	MN	LEVEL	MN	LEVEL	MN	LEVEL
00	2	10	2	20	0	30	0	40	0	50	99
	3		2		0		1		0		102
	3		2		0		0		1		97
	2		1		0		1		0		101
01	3	11	1	21	0	31	0	41	30	51	10[illegible]
	2		1		0		0		00		100
	3		1		0		0		00		97
	3		2		0		1		00		103
02	3	12	3	22	0	32	0	42	40	52	97
	2		2		1		2		5		97
	2		3		0		1		5		97
	2		3		0		1		90		97
03	2	13	3	23	1	33	0	43	10	53	92
	1		2		0		0		10		91
	2		1		1		0		00		90
	2		3		1		0		00		88
04	2	14	3	24	1	34	0	44	250	54	86
	2		2		1		0		460		86
	2		1		1		0		846		81
	1		0		1		0		94		80
05	2	15	1	25	1	35	0	45	95	55	76
	3		0		0		0		103		73
	1		0		1		0		102		71
	1		0		0		0		103		67
06	3	16	0	26	0	36	0	46	101	56	66
	3		0		0		0		98		65
	1		0		0		0		101		62
	1		0		0		0		103		61
07	1	17	0	27	0	37	0	47	101	57	59
	1		0		0		0		100		58
	2		0		0		0		101		55
	1		0		0		0		101		54
08	1	18	0	28	0	38	0	48	100	58	53
	2		0		0		0		99		51
	2		0		0		0		99		49
	0		0		0		0		100		47
09	1	19	0	29	1	39	0	49	103	59	49
	1		0		1		0		101		45
	1		0		0		0		99		47
	3		0		1		2		96		45

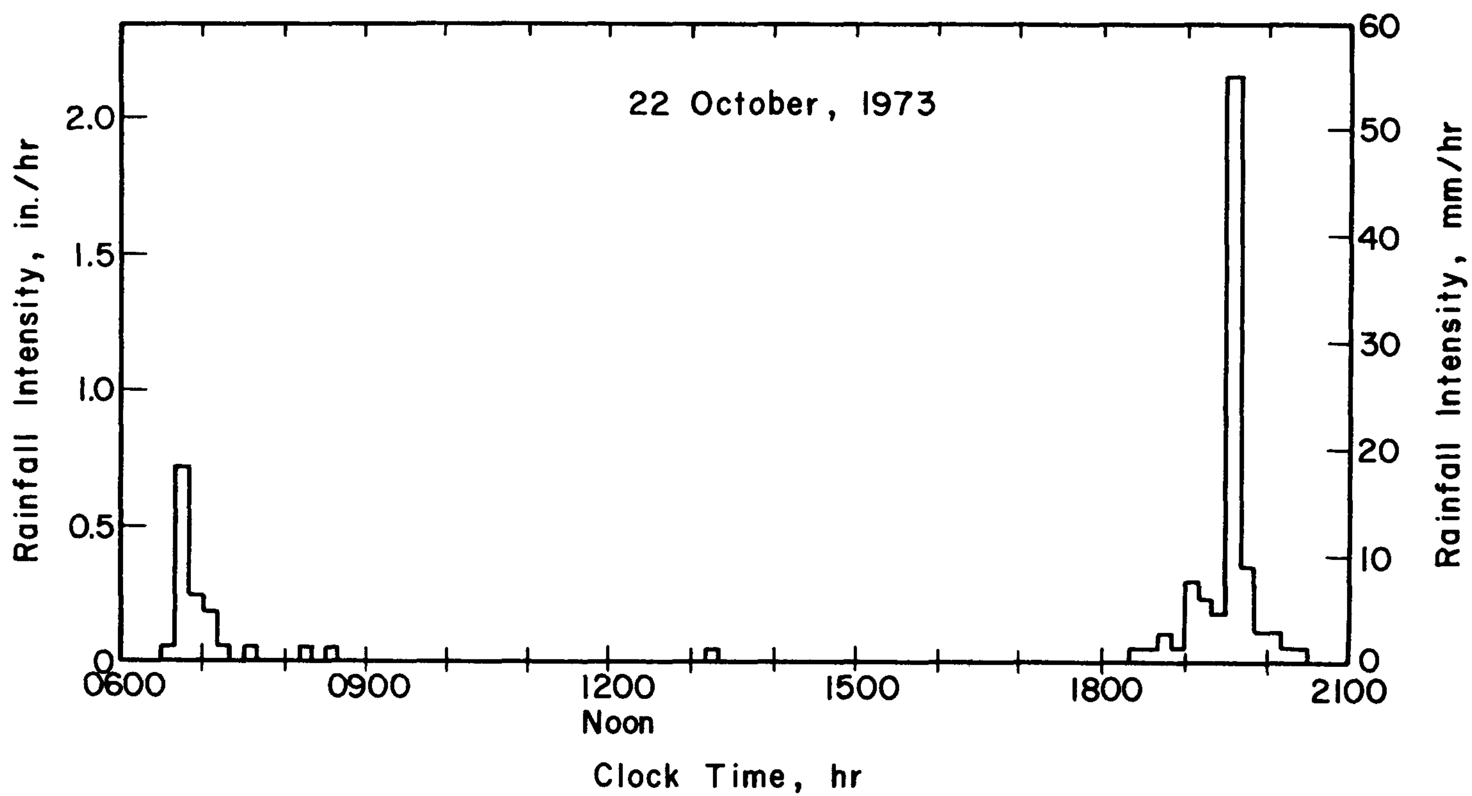


Fig. 19. Hyetograph for rainstorm of 22 October, 1973

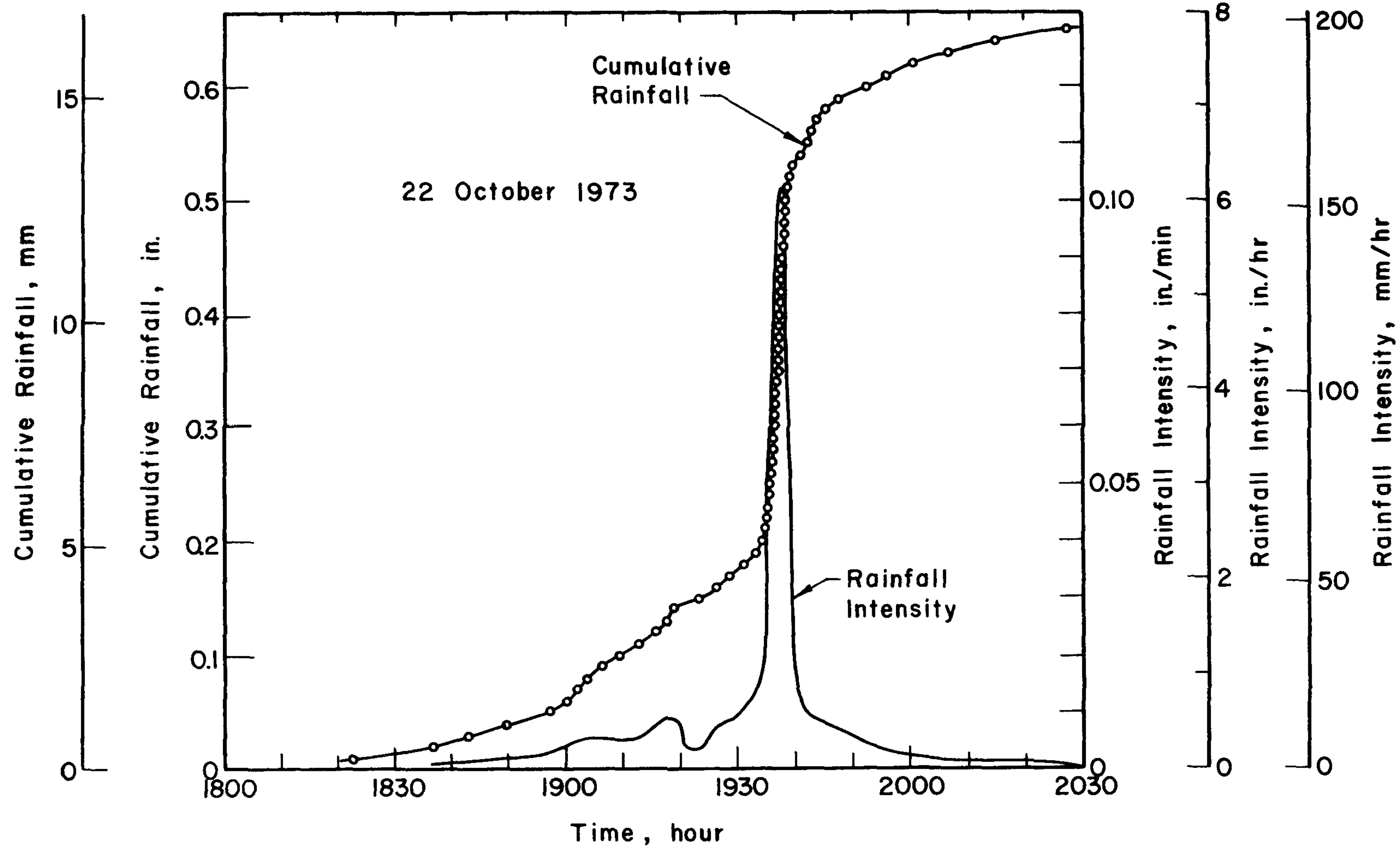


Fig. 20. Cumulative rainfall and intensity for rainstorm of 22 October, 1973

SECTION VII

PROPOSED SIMPLIFIED METHOD FOR RUNOFF FLOWRATE DETERMINATION FOR AREAS OF STEEP SLOPES

As discussed in Sections IV and V, ironically it is the supposedly more reliable, higher level of hydraulic sophistication runoff prediction models that have more difficulties in producing results when applied to areas having steep slopes. In fact, the model with the highest level of hydraulic sophistication, the Illinois Urban Storm Runoff Model, is simply unable to produce numerical results. Presumably the WRE SWMM will also be unable to produce any results if the computation is carried out by using the dynamic wave equations with an explicit numerical scheme. Moreover, even if a technique is developed that will be able to produce results, the computer cost to obtain such solutions would probably be prohibitively high and the accuracy cannot be guaranteed.

Conversely, from an engineering viewpoint, by considering the fact that the time of travel of the fast flowing runoff water on a steep slope surface, gutter, or sewer is relatively short, it is possible to develop simplified approximate methods that can be used for flow rate determination producing results that are within acceptable engineering accuracy. One such method which was developed in this project is described in this section.

The method is developed on the basis of the fact that the water travel time for the surface runoff and sewer flow in a block with steep slope is short relative to the computational time interval used in routing and hence accurate determination of the travel time becomes unnecessary. The simplified method consists of the development and utilization of the inlet unit hydrographs for surface runoff and a modified linear kinematic wave routing of the sewer flow.

VII-1. SURFACE RUNOFF AND INLET HYDROGRAPH DETERMINATION

The method described below for the determination of inlet hydrographs is applicable to steep slope areas as well as to mild slope areas.

(A) Determination of Inlet Unit Hydrographs

The first step is to classify the surface areas of the drainage basin into typical groups. For example, the Baker Street Drainage Basin can be classified into three groups; namely, the blocks of the Upper Basin and southern Lower Basin which are characterized by residential blocks with steep slopes; the blocks of the Lower Basin which are semi-residential blocks with some commercial and with medium slopes; and the blocks of the northern Lower Basin which are characterized by relatively densely built residential blocks with flat slopes.

For each group, for each block of identical size a typical unit hydrograph for each of the inlets of the block is established. Any method that is generally acceptable to produce unit hydrographs can be used to establish the inlet hydrographs. It can be determined as the average of a number of hydrographs measured at the inlet or identical inlets, exactly in the same manner as normally done to obtain unit hydrographs for larger natural watersheds. Or it can be determined by using any one of the synthetic methods. However, measured data of runoff at city block inlets are normally unavailable and unreliable. Also, past data may represent the runoff for a physical condition that is different from the present or future conditions. Under such circumstances the inlet unit hydrographs can only be derived based on synthetic methods.

One synthetic method that has been applied to the Baker Street Drainage Basin to establish inlet unit hydrographs is the Illinois Surface Runoff (ISR) model (Chow and Yen, 1976). For each block the surface is divided into rectangular strips. Within each strip the physical properties of the land surface as well as abstractions are assumed to be homogeneous. Each strip flows into a gutter which in turn flows into the inlet at its downstream end or turns the block corner into the next gutter. No more than 10 rectangular strips are allowed for each gutter. The flow directions of the strips and gutters are determined by the topographic conditions. Both the overland flow and gutter flow are simulated by the one-dimensional nonlinear kinematic wave approximation, e.g., Eq. 1 and from Fig. 4,

$$S_o - S_f + \frac{1}{gA} \int_\sigma (U_x - V) q \; d\sigma = 0 \tag{9}$$

with all the symbols defined before and listed in Notation. For overland flow the friction slope S_f is computed by using Darcy-Weisbach's formula with Weisbach's resistance coefficient estimated by a simplified Moody Diagram (Chow and Yen, 1976). For gutter flow S_f is computed by using Manning's formula. The reason to use the computationally more complicated Darcy-Weisbach formula for overland flow is because under sheet flow conditions Manning's n is no longer a constant. However, it should be reminded here that even by using Darcy-Weisbach formula the resistance coefficient used in routing is nothing but a representative average condition of the surface of the strip and it does not necessarily represent the actual surface roughness. The lateral flow q in Eqs. 1 and 9 is the rainfall rate or the infiltration rate. Infiltration is estimated by using Horton's formula. For gutter flow q may also represent the input from overland flow.

The nonlinear kinematic wave equations, Eqs. 1 and 9, are solved by using a four-point noncentral semi-implicit numerical scheme. Details of the ISR model and the computer program have been reported elsewhere (Chow and Yen, 1976). Inlet hydrographs for the block for several rainfalls of different durations are generated if necessary. For each rainfall duration the intensity is uniformly distributed over the duration and over the entire block. To avoid inaccuracy due to small rainfalls preferably the intensity used in the computation is about 1 to 3 in./hr (25-75 mm/hr). Finally, the computed hydrographs are converted into unit hydrographs for the respective inlets by the standard linear proportional technique of unit hydrograph theory. If possible, preferably several rainfalls of the same duration but different intensities would be used to generate several unit hydrographs of the same duration and the average of these unit hydrographs would be used as the inlet unit hydrograph.

In this development of the inlet unit hydrographs, in addition to the assumptions and reliability limitations of the kinematic wave approximation and the unit hydrograph theory, it is also assumed that the surface runoff takes place only around each individual block and no cross-block flow occurs. The flow is assumed completely intercepted by the inlets of the block.

Once the unit hydrographs are established for the inlets, they will be used repeatedly like a scale for different rainfalls having the same duration as the unit hydrograph and for any blocks that have similar physical characteristics as the one used to establish the unit hydrographs. For example, if in an urban area there are ten blocks of identical size having similar land usage and surface slopes, and each has three inlet catch basins, one at each of three of the four block corners, for a given rainfall duration it is necessary to establish only three unit hydrographs, one for each of the inlets. These three inlet unit hydrographs can then be applied to all the ten blocks and to all other rainfalls having the same duration to produce the corresponding inlet runoff hydrographs.

(B) Determination of Inlet Hydrograph by Using Unit Hydrographs

The inlet hydrographs are obtained by linear combination of unit hydrographs as specified in the standard procedure of unit hydrograph theory. The procedures include:

(a) Obtain the hyetograph that is to be considered — The hyetograph can be obtained from past record or from assumed future rainfall as the objective of the project dictates. The hyetograph is represented as block diagram rainfall depth over equal time intervals, mostly through digitization of rainfall data. For instance, the standard hourly rainfall data available from the U.S. National Oceanic and Atmospheric Administration Environmental Data Service are given in terms of depth over one hour interval. For inlet hydrograph determination block diagrams of rainfall depth over 10, 5, 2, or 1 min are more useful.

(b) Select the appropriate unit hydrographs - From the set of inlet unit hydrographs of different durations for different typical city blocks, select the ones with the appropriate duration and that represent the block being considered. For example, if the hyetographs are rainfall depth or intensity over equal time intervals of 10 min, then the 10-min unit hydrographs should be used.

(c) Obtain the component hydrographs — For each inlet multiply the inlet unit hydrograph by the depth of rainfall for each time interval of the hyetograph. For example, if the hyetograph consists of 0.8 in. (20 mm) of rainfall in the 10-min time interval followed immediately by 1.1 in. (28 mm) of rainfall in the next 10-min interval, the ordinate (discharge) of the 10-min unit hydrograph is multiplied by 0.8 for the first rainfall interval and by 1.1 for the second. These component hydrographs should be listed or plotted in accordance with their time sequence.

(d) Summation of component hydrographs to obtain inlet hydrograph — The component hydrographs obtained in (c), after listing or plotting the discharges (ordinates) at the appropriate time according to the time sequence of the rainfall, are summed up to produce the inlet hydrograph.

(e) Procedures (c) and (d) are repeated for each inlet and for all the blocks within the basin under consideration.

It should be emphasized that the most difficult part of this method, the establishment of the inlet hydrographs, need to be determined only once. After established they are used again and again. Once the inlet unit hydrographs are established, Step (A) on p. 65 is no longer needed. Contrarily, the inlet hydrographs need to be computed every time a rainstorm is considered. The ISR Model is too costly to be used for computing surface runoff in drainage basins of any significant sizes. Conversely, the proposed method of a combination of the ISR model together with the unit hydrograph theory applied to representative blocks considerably simplify the computations and yet preserve sufficiently the accuracy as required in practical applications.

VII-2. SEWER FLOW ROUTING

With all the inlet hydrographs determined as described in the preceding section, the input into the sewer system is known. The sewer system can be described by the node-link representation as discussed elsewhere

(Sevuk et al., 1973; Chow and Yen, 1976). The inflow hydrographs are then routed sewer by sewer towards downstream using a simplified kinematic wave approximate technique. In this technique the inflow hydrograph at the upstream end of a sewer is transferred into the outflow hydrograph at its downstream with distortion reflecting approximately the sewer flow travel time and sewer storage. The sewer flow velocity is approximated by using Manning's formula and from the sewer geometry shown in Fig. 21,

$$V = \frac{C_n}{n} \left[\frac{d}{4}\left(1 - \frac{\sin\phi}{\phi}\right)\right]^{2/3} S_o^{1/2} \tag{10}$$

in which the constant $C_n = 1$ for SI units and $C_n = 1.49$ for English units. The flow central angle ϕ is given by noting that from $Q = AV$, one obtains

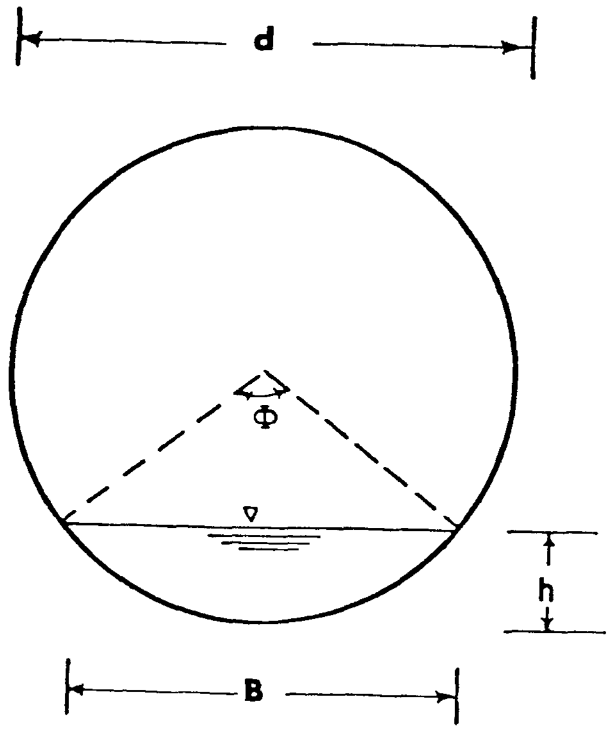
$$\phi\left(1 - \frac{\sin\phi}{\phi}\right)^{5/3} = 20.1 \frac{nQ}{C_n} d^{-8/3} S_o^{-1/2} \tag{11}$$

The travel time is computed by

$$t = L/V \tag{12}$$

in which L is the sewer length. The volume of sewer storage, $\Delta\forall$, during a time interval $\Delta t = t_2 - t_1$ is approximately equal to $(B_1+B_2)\Delta h/2$ where the surface width $B = d \sin(\phi/2)$ and the depth $h = (d/2)[1-\cos(\phi/2)]$.

In the hydrograph routing, an inflow discharge Q_{u2} which occurs at time t_2 is shifted to the downstream outflow hydrograph at the time $t_2 + (L/V)$ and with the magnitude (discharge) reduced by $\Delta\forall/\Delta t$. This procedure is repeated until the entire hydrograph is routed. From a theoretical viewpoint, this simple routing technique is a very rough approximation. However, from a practical viewpoint, its accuracy is quite acceptable despite its simplicity. Normally, in routing of rainstorm runoff in actual drainage basins, the computational time interval is of the order of one min, if not much longer. In a sewer having a steep slope, the flow velocity is higher than 5 fps (1.5 m/s) except at the very beginning and very end of the runoff, and probably in most of the time faster than 10 fps (3.0 m/s). The length of a sewer between junctions is normally less than 1000 ft (300 m), and usually between



$$\text{Flow area} \quad A = \frac{d^2}{8} (\Phi - \sin \Phi)$$

$$\text{Hydraulic radius} \quad R = \frac{d}{4} \left(1 - \frac{\sin \Phi}{\Phi}\right)$$

$$\text{Depth} \quad h = \frac{d}{2} \left(1 - \cos \frac{\Phi}{2}\right)$$

$$\text{Water surface width} \quad B = d \sin \frac{\Phi}{2}$$

Fig. 21. Circular Sewer Flow Cross Section

300 ft (90 m) to 500 ft (150 m) in American cities. Thus the flow travel time in a sewer with a steep slope is usually less than 2 min. Thus, in view of a computational time interval of 1 min or longer, an error in travel time on the order of 10 sec would not cause significant error in the results of routing. Moreover, when the driving force of the flow is predominant, as in the case of the gravitational force for flow on steep slopes, the flood peak attenuation is relatively insignificant in comparison to the flood discharge. Therefore, the proposed scheme provides a reasonable simple approximation. This is particularly significant since at present a theoretically sound solution of the equations representing the unstable supercritical flow with roll waves cannot be obtained. Nevertheless, it should be reminded here that the proposed technique has not been adequately tested because of the lack of available data, and modifications may be desirable if verification with field data, when available, indicates so. This simple sewer routing technique, of course, is applicable to mild slope as well. However, presumably it is much less accurate when applied to mild slope sewers and, hydraulically, it is of the same level as Chicago hydrograph, TRRL, and UCUR methods. But its reliability in comparison to these three methods when applied to mild slope cases has not been investigated.

VII-3. EXAMPLE: BAKER STREET UPPER BASIN

The proposed simple method for storm runoff flow rate determination for steep slope areas is applied to the upper basin of the Baker Street Drainage Basin. The physical characteristics and drainage pattern of the Upper Baker Street Basin have been described in detail in Section VI. The reason for choosing only the Upper Basin as an example is due partly to the project computational economics and partly to the relatively flat land of the northern part of the lower basin. At any rate, the existing runoff data for the entire Baker Street Drainage Basin is inadequate to verify the accuracy of the proposed method even if the runoff for the entire basin is computed. Moreover, it does not seem to be difficult to install a measurement device in the egg-shape sewer in the tunnel to record the runoff from the upper basin.

(A) Establishment of Inlet Unit Hydrographs

A typical block of the Upper Baker Street Basin is shown at the upper right corner of Fig. 22. The rectangular block has three inlets and four gutters of different slopes. In this example the block is arbitrarily subdivided into four isosceles triangles with their common apex at the center of the block (Fig. 23). Each triangle drains into the adjacent gutter which in turn drains into an inlet. The half street roadway width is 34.4 ft (30.4 m). The four gutters all have different slopes. The Illinois Surface Runoff (ISR) computer program of the IUSR model is applied to this typical block for a rainfall of 1 in./hr (2.5 mm/hr) intensity uniformly over a 10-min duration for three different sets of slopes given in Fig. 22. The computed hydrographs are then converted into unit hydrographs by multiplying the ordinate scale by 6 (=60 min/10 min). The computed unit hydrographs for the lowest inlet of the block, called Type C inlet in Fig. 23, are shown in Fig. 22 for the three sets of gutter slopes. An average Type C inlet unit hydrograph is then obtained and used as the standard 10-min unit hydrograph for this inlet.

In applying the ISR program to the typical block to produce the inlet unit hydrographs such as those shown in Fig. 22, no infiltration is considered, i.e., infiltration is assumed equal to zero. Thus, these unit hydrographs are actually direct runoff unit hydrographs. Therefore, later in applying these hydrographs the abstractions should be first taken off from the rainfall before the unit hydrograph application. Each of the isosceles triangular overland area of the block is approximated by a number of rectangular strips of equal width. The gutters are assumed 2 ft (0.61 m) in width with Manning's n = 0.013. The inlets are assumed to be rectangular grate inlets of 2 ft by 2 ft (0.61 m by 0.61 m) in size with an opening ratio of 0.60. It has been found that hydraulically this assumed rectangular grate inlet behaves similar to the semicircular grate inlet that is popular in the Upper Basin. The two intermediate inlets (Types A and B in Fig. 23) each receives water from a gutter and carryover to the downstream gutter is allowed if the inlet capacity is exceeded. The lowest inlet, Type C inlet, receives water from both gutters joining to it, and there is no carryover from this inlet.

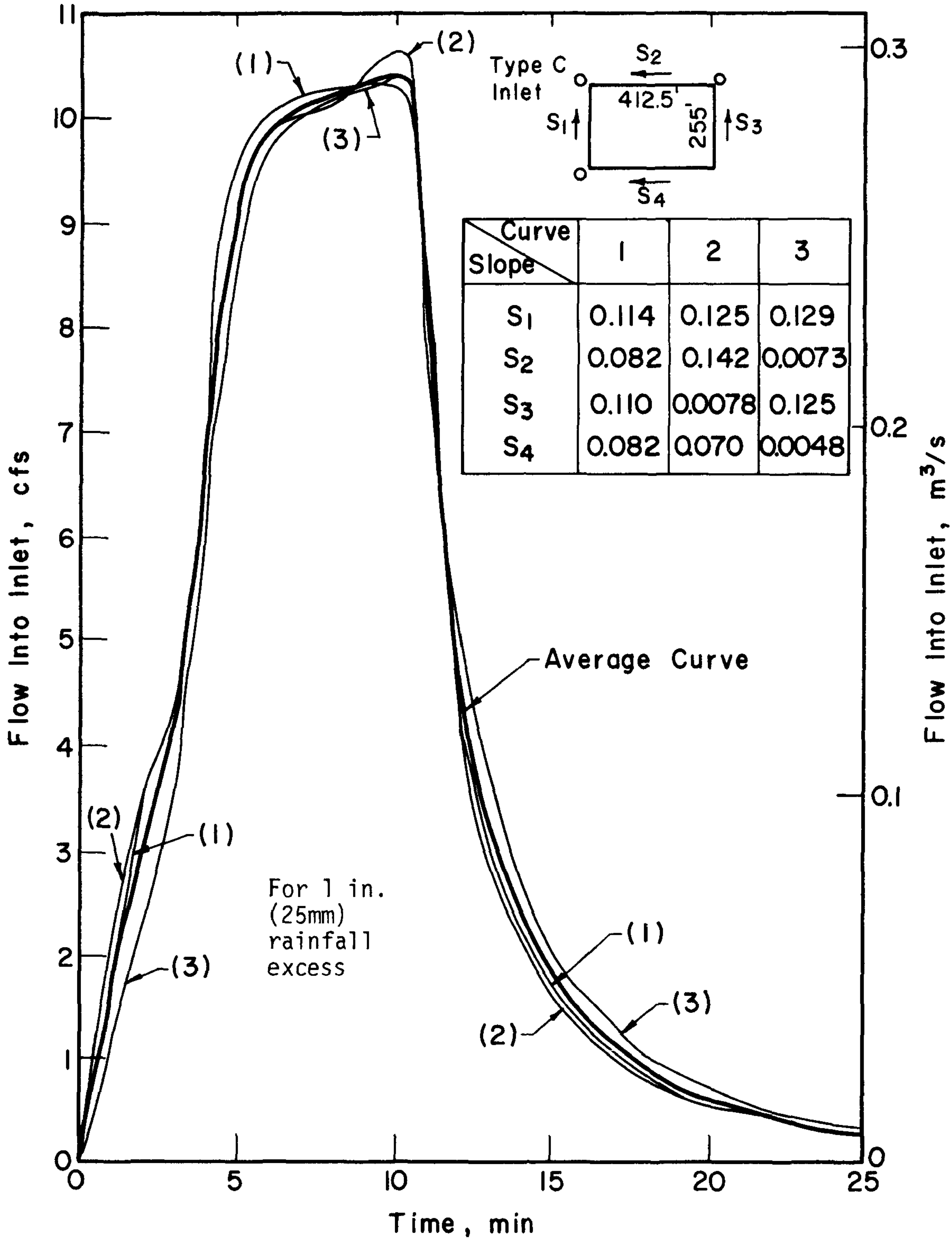


Fig. 22. Type C 10-min inlet unit hydrograph for Upper Basin

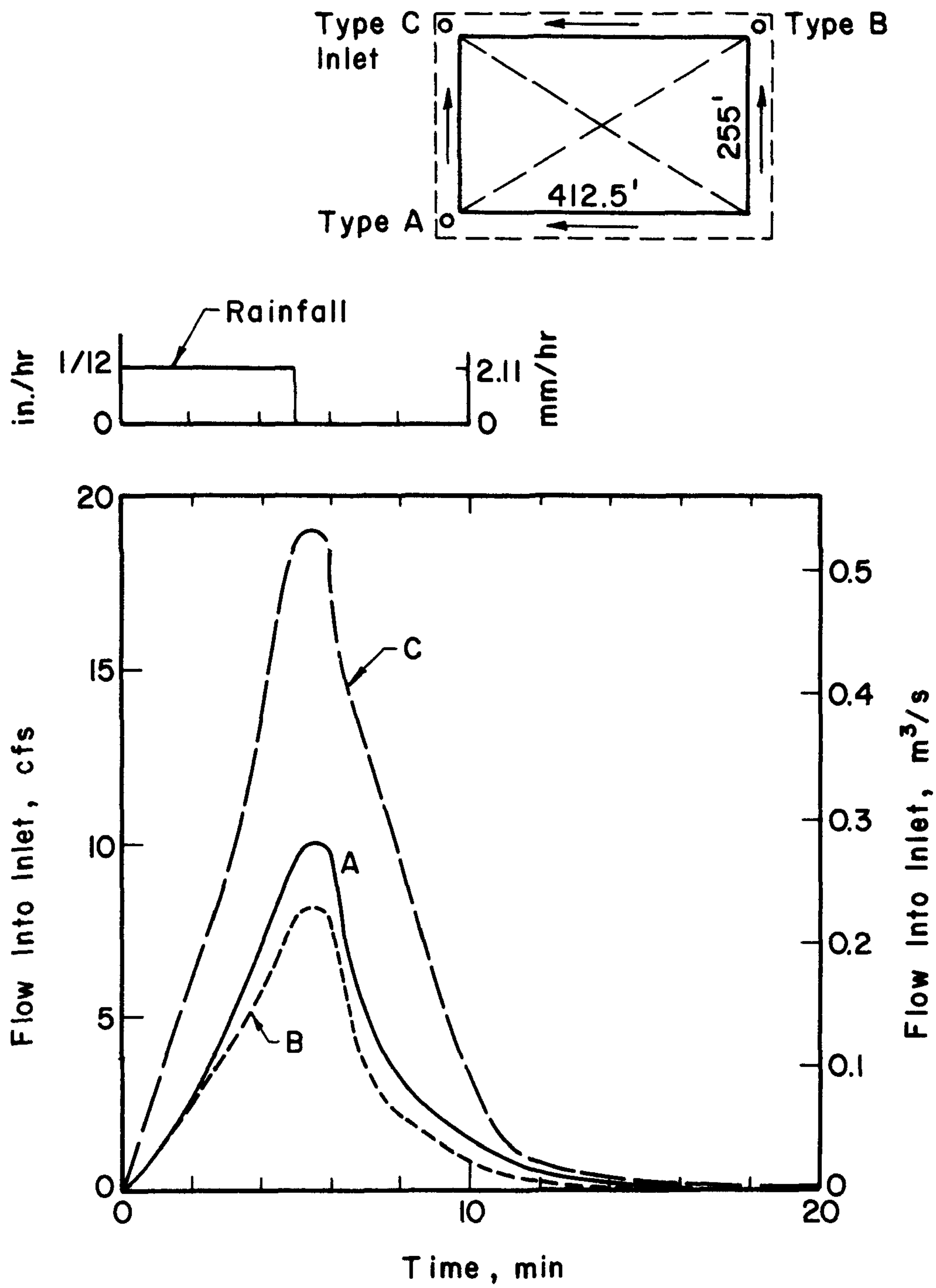


Fig. 23. Standard 5-min inlet unit hydrographs for Upper Basin

Similarly, inlet unit hydrographs for other durations can be established. The average 5-min unit hydrographs for inlets A, B, and C are shown in Fig. 23 and will be used for the rest of the example.

(B) Determination of Rainfall

The rainstorm used in this example is the October 22, 1973 rainfall between the clock hour 18:00 and 20:30 (Fig. 20). The reason for adoption of this rainstorm as an example is because of its good quality detail data in comparison to the rainstorms shown in Figs. 16-18. From the time variation of the rainfall intensity shown in Fig. 20, it is clear that a 10-min duration is too long to represent the rainfall. Contrarily, a 1-min or 2-min duration would require a large amount of computations. By observation it seems that a 5-min duration would be adequately faithful in representing the rainfall without resulting in a large amount of computation. In order to preserve the peak rainfall rate occurred at the time between 19:34 to 19:39, the durations are taking as successive 5 minutes start from 18:19.

(C) Application of Inlet Unit Hydrographs

The 5-min inlet unit hydrographs are then applied to each 5-min duration intervals in succession for each inlet in the block. For example, for the Type C inlet shown in Fig. 23, for the October 22, 1973 rainstorm there are 20 hydrographs between the time 19:39 and 20:19, each lagged to the other by 5 minutes in succession, and each produced by a rainfall of 5-min duration. The ordinates of each of these hydrographs are equal to the corresponding ordinates of the 5-min Type C inlet unit hydrographs shown in Fig. 23 multiplied by the depth of rainfall in the respective 5-min interval. These hydrograph ordinates with appropriate successive 5-min time shifting are then added arithmetically to produce the Type C inlet hydrograph for this particular rainstorm. This Type C inlet hydrograph can subsequently be applied to all the Type C inlets in the basin.

Inlet hydrographs for the October 22, 1973 rainstorm for Types A and B inlets can be similarly established. The identification of the types of inlet hydrographs for the Upper Baker Street Basin is shown in Table 7.

It should be noted here that such identification needs to be done only once, just like the inlet unit hydrographs, and once identified, the table can be used for other rainstorms.

(D) Determination of Sewer Inflow Hydrographs

The inlet hydrographs are transformed into sewer inflow hydrographs assuming there is no time lag. In reality this lag time is of the order of magnitude of seconds and hence can be neglected. If a sewer receives water from more than one inlet catch basin or also from upstream sewers, the hydrographs from these inlets and sewers are summed up arithematically at their respective appropriate time to produce the inflow hydrograph for the sewer under consideration.

(E) Hydrograph Routing in a Sewer

The inflow hydrograph of a sewer is transformed into the outflow hydrograph by using the method and equations described in Subsection VII-2. The procedure can be illustrated in Table 8 and Fig. 24 using the sewer on Laurel Street between Clay and Washington (Nodes 288 to 250) as an example. The inflow has been determined as described in (D) and is listed in the second column of Table 8. The sewer diameter is 12 in. (30 mm) and the sewer slope $S_o = 0.0117$ (Table 3). Thus, with Manning's n = 0.013, Eq. 11 can be reduced to

$$\phi\left(1 - \frac{\sin\phi}{\phi}\right)^{5/3} = 0.514Q \quad Q \text{ in cfs} \qquad (13)$$
$$= 18.2Q \quad Q \text{ in } m^3/s$$

The corresponding equation to compute the reference velocity V is

$$V = C_v\left(1 - \frac{\sin\phi}{\phi}\right)^{2/3} \qquad (14)$$

where

$$C_v = 15.5 \text{ for V in fps}$$

$$C_v = 4.7 \text{ for V in m/s}$$

The computed ϕ and V are given as the third and fourth columns in Table 8.

Table 7. INLET HYDROGRAPH IDENTIFICATION FOR UPPER BASIN

At Intersection of		Node	Type of Inlet Hydrograph A	B	C	Upstream Sewer Flow from Nodes
Maple	Clay	284a	-	-	-	
Maple	Washington	281	-	1*	-	284a
Maple	Jackson	244	2	2	-	281, 280c
Jackson	Cherry	280c	-	-	-	
Jackson	Spruce	221	1	-	1	244, 275
Clay	Maple	284b	-	-	-	
Spruce	Clay	280a	2	-	-	284b
Spruce	Washington	275	1	-	1	280a, 282
Washington	Maple	282	-	1*	-	
Maple	Jackson	243	-	-	-	
Pacific	Maple	233	-	1*	-	243
Pacific	Spruce	187	-	-	1*	233, 221
Locust	Pacific	170	-	-	2	187, 215b, 186
Pacific	(Laurel)	186	-	1*	-	
Clay	Spruce	280b	-	-	-	
Locust	Clay	278	2*+2	-	-	280b, 287
Clay	Laurel	287	-	-	-	
Washington	Spruce	274	-	-	-	
Washington	Laurel	249	-	1	-	
Locust	Washington	248	1*+1	-	2	249, 274, 278
Locust	Jackson	215b	1*	-	2	248, 220, 218
Jackson	Spruce	220	-	1	-	
Jackson	Laurel	218	-	1	-	
Pacific	Lyon	370c	-	-	-	
Lyon	Pacific	370a	-	-	-	
Jackson	Lyon	330b	-	2	-	370a
Presidio Av	Jackson	312	2*	1*	-	330b, 315
Presidio Av	Washington	315	-	-	-	
Pacific	Presidio Av	300	-	1*	1	312, 370c
Pacific	Walnut	228	-	1*	1*	300, 252
Jackson	Walnut	311	-	1*	-	
Washington	Walnut	314b	-	-	-	
Walnut	Clay	304	-	-	-	
Walnut	Washington	284c	2*	1*	-	314b, 304
Walnut	Jackson	252	1*	-	1*	284c, 311
Clay	Walnut	303	-	-	-	
Laurel	Clay	288	2	-	-	303
Laurel	Washington	250	1	-	1	288, 283
Washington	Walnut	283	-	1	-	
Jackson	Walnut	251	-	1	-	
Laurel	Jackson	219	1	-	1	251, 250
Pacific	Laurel	188	-	-	1	219, 228
Pacific		501	-	-	-	188
Pacific		502	-	-	-	501, 170

*Special inlet unit hydrographs because of block size or flat slope.

Table 8. HYDROGRAPH COMPUTATION FOR SEWER BETWEEN NODES 288 AND 250

Clock hr	Time min	Inflow at Node 288		ϕ	Velocity		Sewer flow time	Increment sewer storage, $\Delta\forall$		Discharge reduction due to storage $\Delta\forall/\Delta t$		Sewer Outflow discharge clock time			Interpolated outflow at Node 250	
		cfs	$10^{-3}m^3/s$	rad	fps	m/s	sec	ft^3	$10^{-3}m^3$	cfs	$10^{-6}m^3/s$	cfs	$10^{-3}m^3/s$	min	cfs	$10^{-3}m^3/s$
19	00	0.040	1.1	0.824	3.57	1.09	91	0.0084	0.24	0.00014	3.9	0.040	1.1	19:01.52	0.032	0.9
	01	0.126	3.5	1.084	5.06	1.54	64	0.0137	0.38	0.00023	6.4	0.126	3.5	02.07	0.032	0.9
	02	0.236	6.6	1.262	6.09	1.85	53	0.0136	0.38	0.00022	6.2	0.236	6.6	02.88	0.115	3.2
	03	0.346	9.7	1.388	6.86	2.09	47	0.0119	0.33	0.00020	5.6	0.346	9.7	03.79	0.252	7.1
	⋮	⋮														⋮

The nominal sewer flow travel time is computed as L/V where the length L = 324 ft (99 m). The change of sewer storage in time interval $\Delta t = t_2 - t_1$, in cu ft is

$$\Delta V = \frac{1}{4} (\sin \frac{\phi_1}{2} + \sin \frac{\phi_2}{2})(\cos \frac{\phi_1}{2} - \cos \frac{\phi_2}{2}) \qquad (15)$$

The time interval Δt used in the computation is $\Delta t = 1$ min. Therefore, the average discharge reduction due to the sewer storage is $\Delta V/\Delta t$ and the computed values are given in the seventh column of Table 8. It can be seen from the values in Table 8 that the discharge reduction due to sewer storage is relatively small and usually can be neglected, consequently reducing about 20% of the computations.

The sewer outflow can now be computed as $Q_{inflow} - \Delta V/\Delta t$ and is at the time on the outflow hydrograph equal to the clock time of the inflow plus the sewer flow travel time. This computed time of the outflow hydrograph usually is at odd numbers not coinciding with the time intervals used for the general computation. For instance, the inflow of 0.126 cfs (0.0035 m^3/s) at 19:01 entering through the upstream end of the sewer (Nodes 288 to 250) shown in Fig. 24a and Table 8 becomes the outflow of 0.126 cfs (0.0035 m^3/s) at 19:02.07 at the downstream end of the sewer, shown as a cross point in Fig. 24b. A linear interpolation is then used to obtain the outflow discharge at the time intervals of computation, e.g., 0.126 - [(0.126 - 0.040) (2.07 - 2.00)/(2.07 - 1.52)] = 0.115 cfs (0.0009 m^3/s) for the outflow at time 19:02 in Fig. 24b and Table 8.

(F) The above procedures are repeated sewer by sewer in sequence until all the sewer flows are routed and the basin outflow hydrograph is obtained. The computed runoff hydrograph for the October 22, 1973 rainstorm for the Upper Baker Street Drainage Basin is shown in Fig. 25.

The proposed method would be as tedious as the Chicago Hydrograph or TRRL methods if the computations are done by hand calculations. However, it is relatively easy to program for digital computers and requires very little computation time. In fact, the computation can be done with reasonable efficiency on a programmable pocket calculator providing the geometric and rainfall data are hand loaded requiring no storage locations and the surface and sewer flows are computed separately.

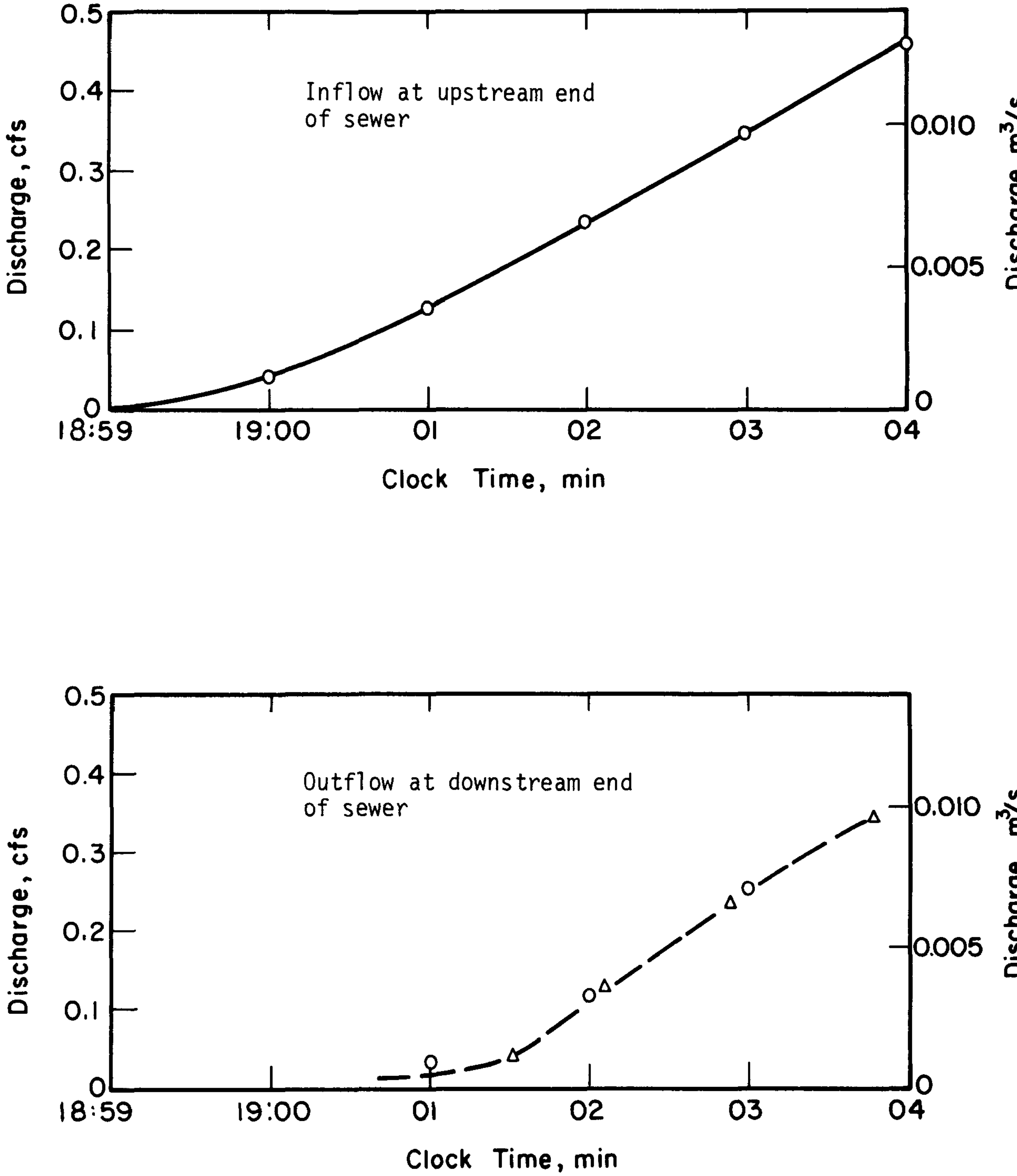


Fig. 24. Hydrograph routing in sewer

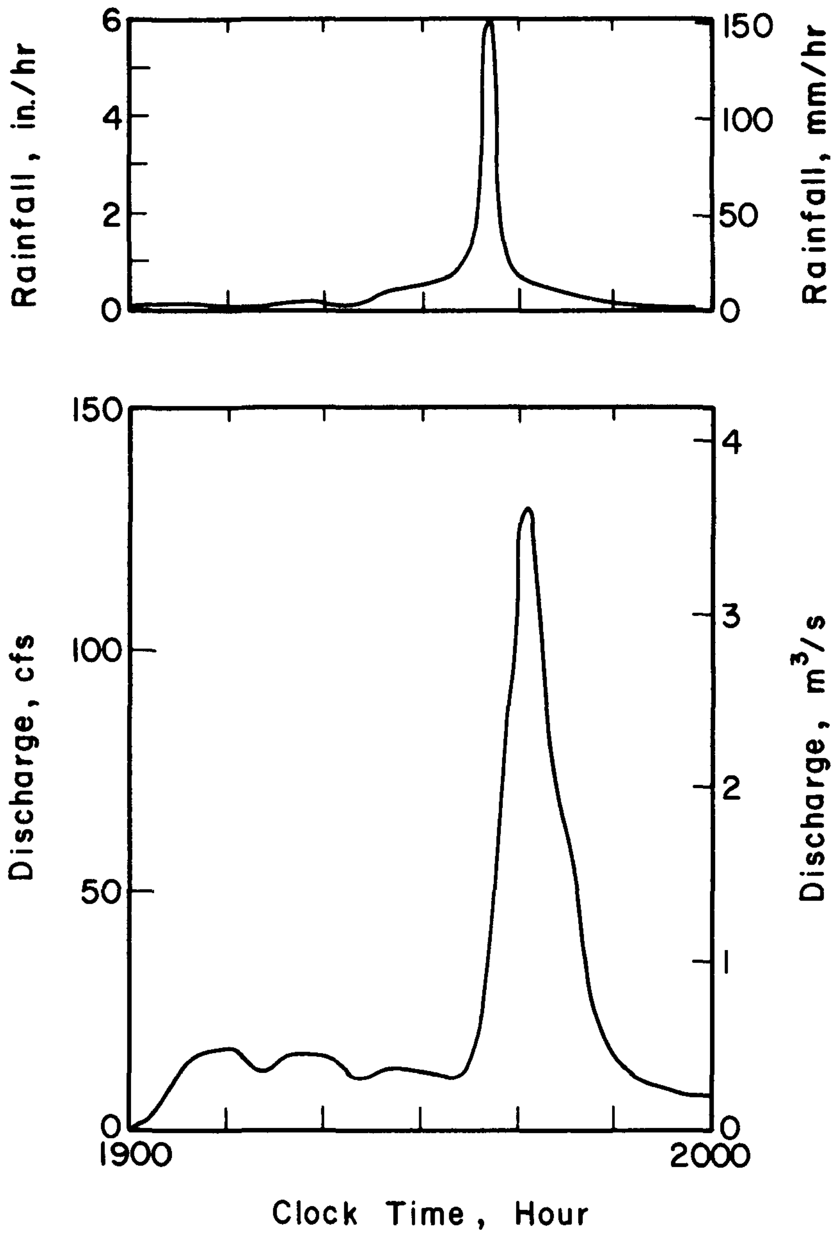


Fig. 25. Runoff from upper basin for rainstorm of 22 October, 1973

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APPENDIX

BAKER STREET DRAINAGE BASIN RUNOFF WATER QUALITY

The runoff water quality data of dry weather flow and the rainstorms of 4-5 April, 15 October, and 5 November, 1969 for the Baker Street Drainage Basin are taken from the report by Engineering-Science, Inc. (1971). The combined sewer flow cumulative pollutant discharges for each of the three rainstorms are summarized in Table A-1 and the corresponding pollutographs are shown in Figs. A-1 to A-3. The dry weather flow quality is summarized in Table A-2. More detailed data as well as description on the sampling technique can be found in the original report.

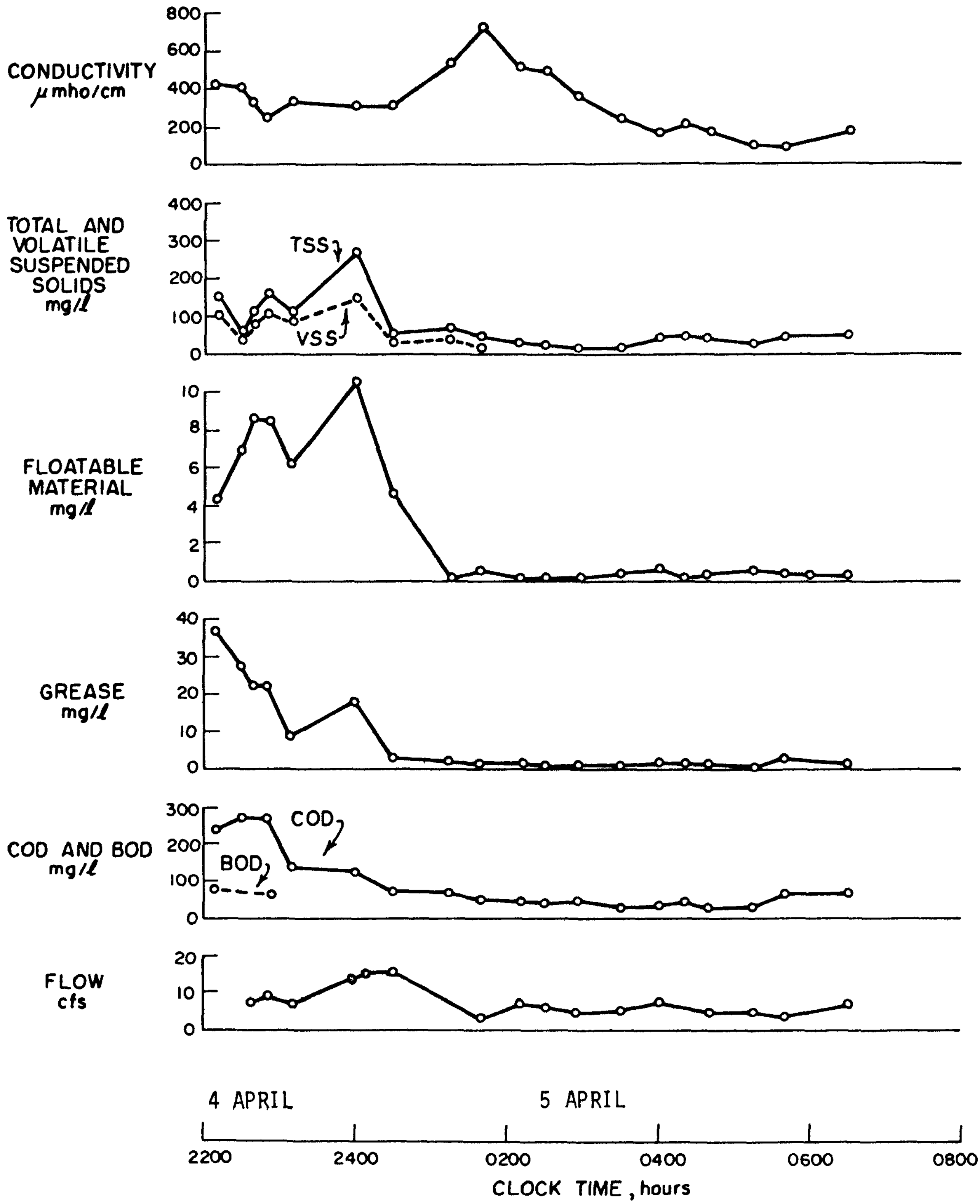

Fig. A-1. Pollutographs for runoff on 4-5 April, 1969

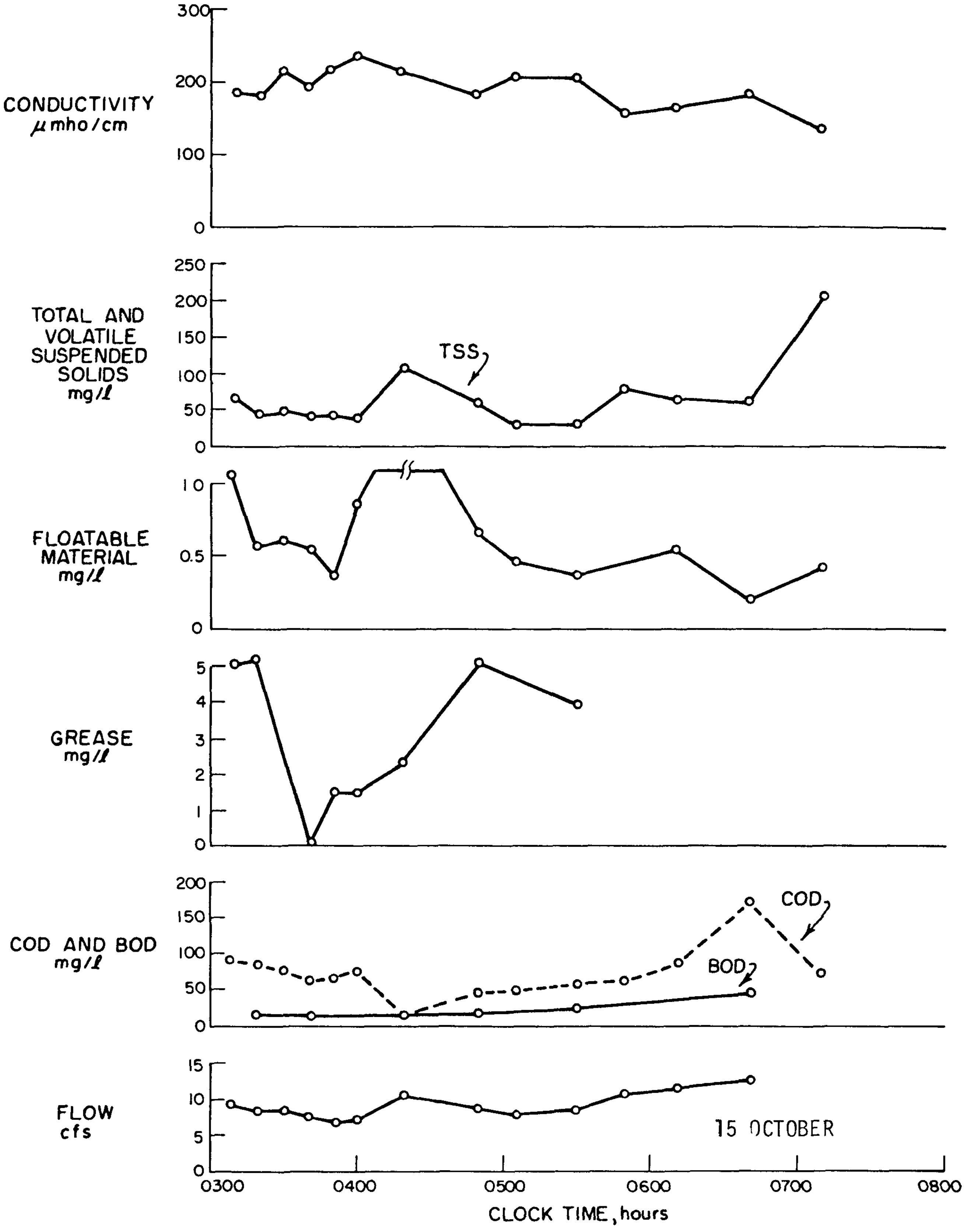

Fig. A-2. Pollutographs for runoff on 15 October, 1969

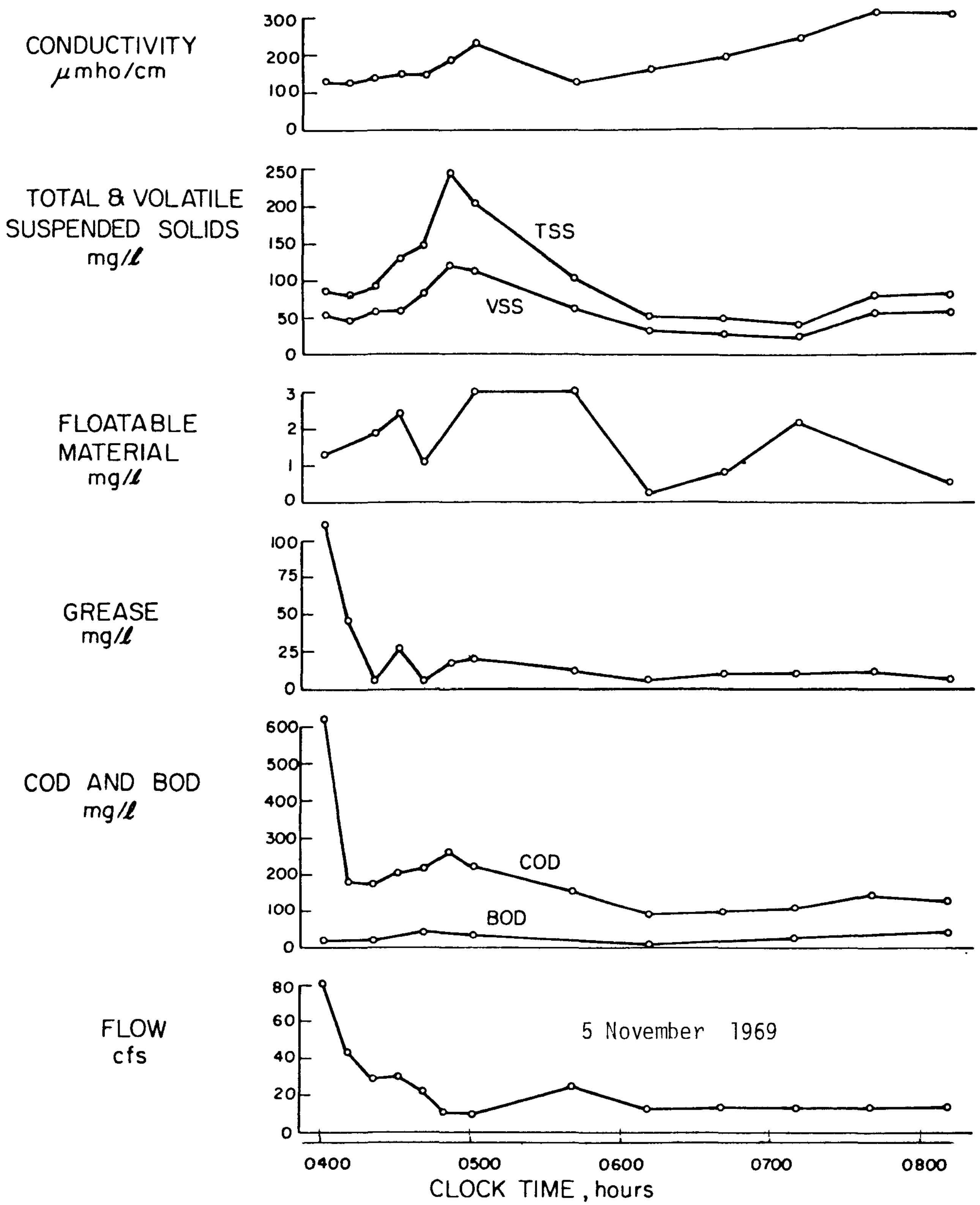

Fig. A-3. Pollutographs for runoff on 5 November, 1969

Table A-1. BAKER STREET DRAINAGE BASIN RUNOFF WATER QUALITY CUMULATIVE NET MASS EMISSION

Date of Rainstorm	COD		Total Suspended Solids		Total Nitrogen		Ortho-phosphate-P		Floatable Materials		Hexane Extractables	
	lb	kg	lb	kg	lb	kg	lb	kg	lb	kg	lb	kg
4-5 April 1969	749	340	1,030	467	217	98	24.9	11.3	39.7	18.0	-29.2	-13.2
15 October 1969	347	157	402	182	20.7	9.4	25.0	11.3	3.9	1.8	13.4	6.1
5 November 1969	3,110	1,410	1,500	680	142	64	9.9	4.5	30.1	13.7	382	173

Data from Engineering Science, Inc. (1971)

Table A-2. BAKER STREET DRAINAGE BASIN DRY WEATHER FLOW QUALITY[a]

Parameter	Quantity		Weighted Mean Concentration
	lbs per capita per day	g per capita per day	mg/ℓ
Flow	131[b]		2.64[c]
COD	0.321	146	294
TSS	0.142	64.4	130
VSS	0.122	55.3	112
Floatables	0.0020	0.9	1.8
HEM	0.062	28.1	56.8
Total Nitrogen	0.020	9.1	18.3
Ammonia Nitrogen	0.007	3.2	6.4
Orthophosphate-P	0.0064	2.9	5.9

[a] Sampled on 19-20 May, 1969; data from Engineering Science, Inc. (1971).

[b] Average dry weather flow = 131 gal/capita-day = 496 ℓ/capita-day.

[c] Average dry weather flow = 2.64 cfs = 0.074 m^3/s.

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

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15. SUPPLEMENTARY NOTES

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16. ABSTRACT

A research is conducted to investigate the applicability of commonly used urban storm runoff prediction models to drainage basins with steep slopes. The hydraulics of runoff on steep slope areas is first reviewed and its difference from that for mild slope areas is discussed. Next the difficulties in applying commonly used methods to steep slope basins are presented. It appears that most engineers are not aware of the problems associated with runoff from steep slope areas and they do not realize that the numerical results given by the conventionally used methods, if obtainable, may not be reliable. A simple approximate method specifically for steep slope basins is proposed and an example is provided. The example utilizes the data from the Baker Street Drainage Basin in San Francisco.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Drainage Rainfall Runoff Storm sewers Surface drainage Computer programs Mathematical models	Hydrographs. Steep Slope. Baker Street Basin. San Francisco.	13B

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