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Rational drainage design for the desert Southwest

Lueck, Curtis Calvin, Ph.D.
The University of Arizona, 1989

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RATIONAL DRAINAGE DESIGN FOR
THE DESERT SOUTHWEST

by
Curtis Calvin Lueck

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A Dissertation Submitted to the Faculty of the
DEPARTMENT OF CIVIL ENGINEERING
AND ENGINEERING MECHANICS

In Partial Fulfillment of the Requirements
For the Degree of

DOCTOR OF PHILOSOPHY
WITH A MAJOR IN CIVIL ENGINEERING

In the Graduate College

THE UNIVERSITY OF ARIZONA

1989
As members of the Final Examination Committee, we certify that we have read
the dissertation prepared by Curtis Calvin Lueck
entitled Rational Drainage Design for the Desert Southwest

and recommend that it be accepted as fulfilling the dissertation requirement
for the Degree of Doctor of Philosophy.

[Signatures and dates]

Final approval and acceptance of this dissertation is contingent upon the
candidate's submission of the final copy of the dissertation to the Graduate
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I hereby certify that I have read this dissertation prepared under my
direction and recommend that it be accepted as fulfilling the dissertation
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DEDICATION

to Maureen
ACKNOWLEDGEMENTS

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ABSTRACT

Drainage systems for the desert Southwest are currently designed without much consideration for the climatological or surficial conditions of the region. The "100-year" flood has become the design standard throughout the United States due to misunderstandings about requirements of the National Flood Insurance Program. The effect of larger floods is virtually ignored, seasonal variations of rainfall patterns and intensities are neglected, and hydrologic data collection is extremely limited in watersheds of the urbanizing Southwest. The laws of nature are obscured by the rules of man during the planning and design of desert drainageways.

Procedures for extrapolating runoff records and estimating the magnitude of the 100-year flood, including the LP III probability density function, the NOAA Atlas, and HEC-1, have been widely adopted in the arid regions as part of local drainage regulations. Plans are normally not approved unless the basis of design complies with the regulations. Assumptions inherent in the methods are questionable and data to verify the assumptions are limited.

Drainage design can be improved by using available field data and a simple method - based on the Rational Method - is developed. Benefit-cost analysis is a valuable tool for establishing project alternatives, project size, and cost/benefit allocation. An equitability index is defined for evaluating fairness, and it is combined with the benefit-cost ratio for refining and selecting project
design. Estimates of flood peaks can be improved by considering channel abstractions as "negative base flow"; by recognizing the presence of the n-value paradox; by extending flood records through paleohydrologic study; by monitoring rainfall, runoff, and the effectiveness of design strategies in urban catchments; and by using more suitable rainfall estimates. Drainage design can be made more rational by also considering sediment transport; by including non-structural design alternatives; and by evaluating a range of flood magnitudes, not just the 100-year flood. A conceptual drainage ordinance not based on the NFIP is presented.
CHAPTER 1
INTRODUCTION AND PROBLEM STATEMENT

To the most casual observer, the Southwestern states have obvious and startling differences from the remainder of the United States. The most obvious differences include vegetation and climate conditions which, in a large part, are reasons for the dramatic economic growth of this region since the late 1940's. These environmental attributes and the growth attributable to them produce a "healthy", recession-resistant economy attractive to developers and employers. Regional growth is a seldom-disputed fact of life in the southwest.

Subtleties of desert life provide regular challenges to planners, engineers, developers and politicians responsible for facilitating growth in some logical fashion. For the many migrants to the region, the ways of the more humid states seem inappropriate. This is certainly true in the disciplines and sub-disciplines of hydrology and hydraulics. Here, rivers are steeper, the soil is more porous, the plant cover is less dense, the air is drier, and the streams experience flow infrequently. To see an intense summer storm and the damage potential of a desert wash under flow is at first astounding.

To better accommodate imminent urbanization of the desert, the engineer should possess a profound understanding of its hydrologic and meteorological characteristics. However, the design professional may lack this understanding, or may be guided solely by agency regulations that tend to limit
the quality, creativeness, or effectiveness of thought. This dissertation is intended to help fill the gap in drainage design processes created when the designer ignores the desert physical system, or relies too much on guidelines of regulatory agencies. It attempts to strike a reasonable balance between the laws of nature and the laws of man in the design process. The emphasis of this paper is to review development of the field of civil engineering/hydrology, discuss current practices and trends, present more reasonable methods for use in the desert Southwest, and recommend ways to improve the field. This paper represents a synthesis of many existing concepts and several new ideas; it is not merely a detailed investigation into one highly confined sub-area of hydrology.

**Drainage Infrastructure and Its Purpose**

Drainage systems carry surface waters generated by excess rainfall safely to some downstream location. For natural conditions in an uninhabited region, safety is not a concern. The natural system of arroyos, channels, and rivers is therefore sufficient. However, when human occupancy occurs near these natural watercourses, modifications are sometimes necessary to protect life and property from this sometimes unwise intrusion. The inverse proposition would be for human occupancy to not intrude.

Typical drainage modifications include curbs, gutters, and storm drains associated with roadways as well as the arrangement, alignment, and profiles of
the roadways. They also include the construction of open channels and river improvement projects such as levees and revetments. In the semi-arid southwest, these local drainage systems are usually open channels rather than the buried conduit systems of the eastern states. One reason for this difference is the lack of freezing weather; another is the many days of dry weather. Drainage systems can also include detention and retention basins designed to decrease or retard storm water flows. Non-structural management techniques such as zoning controls and land use restrictions are also integral parts of the stormwater management system.

Designers are further compelled to meet federal, state, and local requirements of the National Flood Insurance Program (NFIP). In many instances, these requirements supersede all others and are understood to preclude consideration of alternative designs. These requirements have been misunderstood and misapplied to projects not affected by the insurance program. Although this dilemma is not peculiar to the southwest, its effects are exacerbated by anomalies of desert environments.

Climatic and hydrologic characteristics have lead to some departures from design standards frequently used in other regions. Examples include the common use of dip sections rather than hydraulic structures along roadway/watercourse intersections. Dry river beds are used to mine aggregate used in new construction or to infiltrate sewage effluent into the aquifer. In other regions, these practices may be unfeasible or impossible.
Typical Facilities

Despite the predominant reliance on ground water and importation of flow from exotic perennial rivers for domestic water supplies, storm water runoff has traditionally been regarded as a problem rather than a resource. This attitude is changing as state and local laws begin to mandate water conservation practices, including the recharge or supplemental use of excess rainfall in urban areas. Detention and retention facilities in arid regions have become more prevalent recently, and serve purposes other than exclusively flood control. Typical detention and retention facilities today may include parks, landscaping, and other open space land uses that are damaged only slightly by stormwater inflow or impoundment.

Other differences between arid and non-arid regions in resolving the inconvenience of runoff are apparent. Desert cities appear less finished because curb and gutter systems are not always an integral part of roadway design. Roadways sometimes serve as drainageways through the use of an inverted cross section. Dip sections rather than hydraulic structures accommodate cross flow of drainage. Porous pavement can be used in warm desert areas with less risk of freeze/thaw failure than in colder regions.

In light of the hydrologic and climatic conditions prevalent throughout the southwest, the designer must adopt a different set of considerations in tackling a drainage problem. Designers must recognize the following:
• desert floods are infrequent but sometimes severe
• streams are often steeper than in other regions
• runoff flows more quickly, and is more sediment laden, and
• bank erosion is extensive during major floods

Only when an engineer or hydrologist is aware of these differences can a design adequately protect life and property from the desert flood.

**The Problem**

Meteorological, hydrologic, and climatic conditions of the arid southwestern states are distinctly different from regions east of the 105th meridian (Hansen, 1977). These variations need to be considered in the design of drainage systems, streets, and subdivisions, as well as in the management of land use within floodplains. Reality indicates that these variations are often not fully recognized, understood, and addressed by design professionals or federal, state, and local regulatory agencies.

Many communities throughout the United States are subject to flooding and have subscribed to the federal flood insurance program. This program offers the communities subsidies for the actuarial based rates for insurance for damages from flooding of private property if certain federally-mandated procedures are followed by the community. The Federal Emergency Management Agency (FEMA) typically applies its rules similarly in the southwest
and other states. There is a lack of differentiation of the special characteristics of the arid states that makes these policies inappropriate.

These policies are implemented by state and local agencies to comply with the federal program requirements. Some local agencies recognize that policy-related problems exist and have tried to correct them. Most local agencies, however, do not have the expertise to recognize the problems. Similarly, many engineers and hydrologists in both public and private practice have either accepted "standard" solutions and procedures or have not recognized this problem.

Simply stated, the flood insurance program requires, at a minimum, protection of residential structures from the 100-year flood. This relatively imprecise goal has evolved into a standard for most drainage projects, and all to often the only solution considered is structural design for the 100-year flood. Non-structural solutions at different recurrence intervals are often not considered, and economic analysis of alternative designs is seldom developed. The rules have evolved to a point where engineering judgment based on an analytic approach is applied only on large scale federal projects.

Subsequent Sections

The history, development, and application of federal, state, and local flood control programs are discussed, beginning long before the Flood Control
Act of 1936. This discussion demonstrates that federal requirements have been misunderstood and misapplied. Rules have become so inflexible at the local level that project design has been limited and, in some cases are ineffective or inappropriate. The guidelines of professional associations are discussed along with their failure to differentiate between hydrologically different regions. Both professional organizations and universities provide only limited training about water law and professional liability in drainage system design. Collectively, these discussions represent the laws of man regarding applied hydrology.

The physical phenomena affecting drainage design, as well as the differences between arid and non-arid regions is addressed. The development and application of several hydrologic methods are discussed and compared. Concepts of statistics, probability, and regionalization are presented. The need to incorporate "unbelievable" data (outliers) rather than disregard these data is discussed. Paleohydrology is identified as a valuable resource to planners and engineers. Finally, the concept of the 100-year flood is defined and compared with the concept of probable maximum precipitation. Collectively, these discussions represent the laws of nature regarding applied hydrology.

The concept of benefit-cost analysis as an alternative analytic tool is discussed. More importantly, the issue of who pays for and who benefits from typical drainage system improvements is presented. The balance between cost and benefit to various entities is defined as equitability, and an index is
developed to help quantify this balance. The various costs of drainage system improvements are briefly discussed with reference to cost allocation methods.

Strategies for improving the plight of the hydrologist and engineer in the design of drainage systems in the arid region are presented. An improved, more rational methodology for estimating discharge from small desert catchments using available data is presented. Further research needs are suggested and recommendations for engineering education and legislative change are identified. Interim and ultimate solutions and recommendations for the design professional and regulatory agency are presented.

The reference matter reproduces two landmark papers in the field of engineering and applied hydrology by Mulvany (1852) and Kuichling (1889). Both are frequently cited as the origin of the commonly used "rational method". This dissertation emphasizes the need to obtain and use local rainfall and runoff data, as Mulvany and Kuichling did over one hundred years ago.
CHAPTER 2

REGULATIONS AND AGENCY INVOLVEMENT

The design of drainage systems is affected by laws, rules, and regulations that are implemented by numerous agencies at all levels of government. These range from the general state requirement that a registered professional engineer oversee project design to project-specific detailed analysis mandated by the Federal Emergency Management Agency through the National Flood Insurance Program. The project engineer is supposed to be familiar with the myriad of laws and ordinances as a condition of registration (Arizona State Board of Technical Registration, 1983), but it is unlikely that any engineer is familiar with all these laws, much less their legislative intent and effect on the design process.

Technical drainage issues are compounded by these regulations and their administration. This chapter deals with the laws, rules, and regulations that have a direct and noteworthy impact on the design of drainage systems and the agencies that implement them. It also discusses why many of the rules may have technical validity in the non-arid regions, but should not be blindly transposed to the desert states. The concluding sections discuss some of the professional organizations which provide drainage design guidelines to practitioners, and legal issues in handling storm water and runoff.
Many agencies of the federal government are directly or indirectly involved in drainage system design. Their involvement ranges from meteorological research to residential construction practices. Table 2-1 provides a summary of the more important agencies and their research or regulatory function. Evolution of the various federal flood control programs, including FEMA and the FIA, have been discussed by Arey (1971), White (1975) and others. The programs' history has been traced to the Swamp Lands Act of 1849. Table 2-2 identifies major federal actions in flood control regulation since the Swamp Lands Act. The table also indicates whether the action defines a problem or reacts to (solves) a problem or natural disaster. Most of the actions are attempts to solve crises and are necessarily dependent upon the legislatures' perception of the problem at hand. According to Arey, "decision-making in flood control over the years can be seen as a spiraling pattern of changing perceptions of the problem leading to the adoption of a new set of adjustments which then lend to new perceptions of the problem, and so on."

Despite the development of federal laws and the creation of agencies to implement them, the technology available for design engineers to solve drainage-related problems has not changed much since the beginning of federal involvement. The most important changes have been use of energy rather than animal power in construction and the availability of federal project funding.
<table>
<thead>
<tr>
<th>Federal Agencies</th>
<th>Functions and Roles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Federal Emergency Management Agency</td>
<td>Emergency Management</td>
</tr>
<tr>
<td>Small Business Administration</td>
<td>Small Business Administration</td>
</tr>
<tr>
<td>Tennessee Valley Authority</td>
<td>Tennessee Valley Authority</td>
</tr>
<tr>
<td>Federal Housing Administration</td>
<td>Federal Housing Administration</td>
</tr>
<tr>
<td>Department of Housing and Urban Development</td>
<td>Department of Housing and Urban Development</td>
</tr>
<tr>
<td>Federal Trade Commission</td>
<td>Federal Trade Commission</td>
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<tr>
<td>Department of Commerce</td>
<td>Department of Commerce</td>
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<tr>
<td>Department of Agriculture</td>
<td>Department of Agriculture</td>
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<tr>
<td>Federal Reserve System</td>
<td>Federal Reserve System</td>
</tr>
<tr>
<td>Department of Health and Human Services</td>
<td>Department of Health and Human Services</td>
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<tr>
<td>Department of the Interior</td>
<td>Department of the Interior</td>
</tr>
<tr>
<td>National Oceanic and Atmospheric Administration</td>
<td>National Oceanic and Atmospheric Administration</td>
</tr>
<tr>
<td>Food and Drug Administration</td>
<td>Food and Drug Administration</td>
</tr>
<tr>
<td>Environmental Protection Agency</td>
<td>Environmental Protection Agency</td>
</tr>
<tr>
<td>Occupational Safety and Health Administration</td>
<td>Occupational Safety and Health Administration</td>
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<tr>
<td>Department of Labor</td>
<td>Department of Labor</td>
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<td>Department of Transportation</td>
<td>Department of Transportation</td>
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<td>Federal Aviation Administration</td>
<td>Federal Aviation Administration</td>
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<td>Department of Commerce</td>
<td>Department of Commerce</td>
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<td>Department of Justice</td>
<td>Department of Justice</td>
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<td>Department of Agriculture</td>
<td>Department of Agriculture</td>
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<tr>
<td>Department of Health and Human Services</td>
<td>Department of Health and Human Services</td>
</tr>
</tbody>
</table>

**Table 2.1**

**Federal Agencies and Their Flood Control Function**

*Source: FEMA 1996*
### TABLE 2-2

**FEDERAL ACTIONS IN FLOOD CONTROL**

Major Events in the History of Flood Control in the United States 1849-1977

<table>
<thead>
<tr>
<th>Year</th>
<th>Event</th>
<th>Defines (D)</th>
<th>Solves (S)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1849</td>
<td>Swamp Lands Act provides incentive for local levee building-Lower Mississippi</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>1861</td>
<td>Humphrey-Abbott Report by Corps of Engrs. recommends federal levee building</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>1874</td>
<td>First direct federal relief provided for flood victims</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>1874</td>
<td>Mississippi River Study Commission established</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>1879</td>
<td>Mississippi River Commission established to improve navigation (flood control implied)</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>1890</td>
<td>Flood forecasting shifted to U.S. Weather Bureau</td>
<td>D/S</td>
<td></td>
</tr>
<tr>
<td>1908</td>
<td>Governor's Conference on Conservation &amp; Inland Waterways Comm. recommend multipurpose development</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>1909</td>
<td>Corps of Engrs. directed to assess hydro-power potential of all projects</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>1917</td>
<td>Flood Control Act allocates first federal funds exclusively for flood control</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>1920</td>
<td>Flood Control Act requires local cooperation in flood control surveys</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>1925</td>
<td>&quot;308&quot; Reports authorized. To be comprehensive and on basin wide basis</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>1933</td>
<td>TVA established with flood control as one of its main purposes</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>1936</td>
<td>Flood Control Act asserts federal responsibility in solving all flood problems</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>1944</td>
<td>Flood Control Act extends 1936 Act to include &quot;comprehensive coordinated development&quot;</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>1956</td>
<td>Flood Insurance Act passed but no funds are allocated to establish program</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>1958</td>
<td>Flood Control Act includes water supply as a purpose of federal reservoir construction</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>1960</td>
<td>Flood plain management officially recognized - FP Information Office in Corps of Engrs.</td>
<td>D/S</td>
<td></td>
</tr>
<tr>
<td>1967</td>
<td>Water Resources Council recommends use of Log-Pearson Type IV distribution by federal agencies.</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>1968</td>
<td>National Flood Insurance Act of 1968 established NFIP as we know it.</td>
<td>S</td>
<td></td>
</tr>
<tr>
<td>1973</td>
<td>Flood Disaster Protection Act requires local coordination and public review during flood insurance studies.</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>1976</td>
<td>House Document 465 calls for Unified national program to manage flood losses.</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>1977</td>
<td>Executive Order 11988 requires federal agencies to avoid supporting development within floodplains.</td>
<td>S</td>
<td></td>
</tr>
</tbody>
</table>

Adopted from Arey (1971)
These two changes have increased the scale of federal drainage and flood control projects and the perception of flood control technologies. Development of the federal flood control program has generally been construction oriented, which, according to Arey and others, has focused public attention on construction and diminished perception of the non-structural and less extensive alternatives to large scale construction.

Major changes in the federal programs are also related to the economic state of the nation. Large-scale construction programs occurred during post-war recessions, and the Flood Control Act of 1936 was part of the Roosevelt administration's "New Deal" during and after the great depression. The public became reliant upon the federal government to solve local flooding problems through capital-intensive projects that disbursed federal taxes to the region. The purpose of the Act was as much to provide "good things to do" during the great depression, in contrast to the "leaf raking" of the WPA, as to control floods. Unfortunately it reinforced the notion that the federal government would pay the largest part of project costs and beneficiaries, who were sometimes hard to identify, would not absorb their "share".

The economic effect of this policy has been compared in the literature to non-structural solutions such as zoning controls and floodplain regulation. Federal approaches to flood control changed dramatically in the early 1960's with the Flood Control Act of 1960 which established a flood plain information office within the Corps of Engineers. The next major action was publication by FEMA
(1976) of A Unified National Program for Managing Flood Losses adopted by the 89th Congress. This document has been subsequently revised and updated in 1979 and 1986 and is frequently referred to as House Document 465.

The importance of House Document 465 as a management tool can not be overstated. Its purpose is to alter the typical sequence of events that continues to be flooding, flood losses, disaster relief, structural flood control projects, renewed flood plain encroachment, flooding, and more flood losses. The document points out that although structural solutions have saved lives, protective works alone are not able to keep pace with development within flood prone areas. In many cases, they encouraged unwise use of flood plains with resultant additional losses.

Despite the shift in federal policy to one of apparent enlightenment, there was still a lack of recognition of regional variation in climate, runoff characteristics, and flood plain development problems. This is apparent by the title of House Document 465 which calls for a "unified" national floodplain management program. Conditions of the arid regions are so unlike other regions that a unified program may be an inappropriate goal without adequate consideration of these regional variations.

National Flood Insurance Program

The National Flood Insurance Program is administered through the Federal Emergency Management Agency and is the center of the federal
government's effort to minimize flood losses through non-structural measures. The NFIP began with passage of the National Flood Insurance Act of 1968 and has two major goals. First, the NFIP attempts to reduce flood losses nationally by requiring communities who want to participate in the insurance program to adopt and enforce land use regulations that encourage safe building practices within flood prone areas. Communities that do not comply with these requirements are ineligible for federal heavily-subsidized flood insurance and may be subject to other sanctions and penalties by the federal government. Insurance rates are based on risk premium zones, on a broad basis, not by specific geographic region. The rates for new structures are actuarial, and subsidized. On a broad basis the insurance program may tend to discourage uneconomic development in flood hazard areas. Insurance rates for existing structures are subsidized as much as 90 percent.

The second goal of the NFIP is the transfer of the economic burden of flood losses from the general public, i.e. the taxpayers, to the owners and occupants of developed flood prone land. This is accomplished by federal refusal to provide anything but emergency disaster relief in non-participating communities. The effectiveness of the NFIP has been researched in surveys of community leaders in 1979 and 1983 and reported by Burby and French (1985). Their underlying conclusion is that the FIP will work better if incorporated with other regional land use goals, and with less emphasis on structural solutions. The NFIP has been praised for its emphasis on flood plain land use
management and criticized for its perpetuation of existing unwise land uses. It could also be criticized for an over-reliance on uniform standards and techniques in the development of flood plain management ordinances, flood insurance rate maps, and hydraulic and hydrologic methods. Although there is some sympathy at the federal agencies for desert conditions, the emphasis is to treat Phoenix like Pittsburgh and Tucson like Tampa, even though conditions are radically different.

Flood insurance is big business. According to GAO (1988), there are about two million flood insurance policies written for total coverage of $1.6 billion. The average annual premium is about $260, and can be almost $500 for coastal, high hazard property. Table 2-3 provides a summary of NFIP premiums and revenues as of August 31, 1987. The desert is contained in the "other regions" category. Although the rates shown are the lowest, further analysis is needed to determine if desert dwellers' participation in the program is cost-effective.

Drainage system designers and flood plain managers must become familiar with the technical provisions and terminology of the insurance program. Particularly important is the definition of critical terms such as "flood", "floodway", and "flood hazard zone". These terms and others are defined in Table 2-4, and further definition of terms is contained in NFIP Regulations, Title 44, Code of Federal Regulations.
<table>
<thead>
<tr>
<th>REGION</th>
<th>NUMBER OF POLICIES</th>
<th>TOTAL PREMIUM</th>
<th>AVERAGE PREMIUM</th>
<th>INSURANCE IN FORCE</th>
<th>AVERAGE POLICY VALUE</th>
<th>PREMIUM PER $1,000</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(000)</td>
<td>(000)</td>
<td>(000)</td>
<td>(000)</td>
<td>(000)</td>
<td></td>
</tr>
<tr>
<td>Coastal</td>
<td>1,454</td>
<td>$381,502</td>
<td>$262</td>
<td>$120,673,721</td>
<td>$82,986</td>
<td>$3.16</td>
</tr>
<tr>
<td>% of Total</td>
<td>71.6</td>
<td>72.6</td>
<td>76.9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coastal High Hazard</td>
<td>64</td>
<td>29,796</td>
<td>469</td>
<td>5,200,729</td>
<td>81,793</td>
<td>5.73</td>
</tr>
<tr>
<td>% of Total</td>
<td>3.1</td>
<td>5.7</td>
<td>3.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Great Lakes States</td>
<td>197</td>
<td>50,827</td>
<td>258</td>
<td>10,899,118</td>
<td>55,330</td>
<td>4.66</td>
</tr>
<tr>
<td>% of Total</td>
<td>9.7</td>
<td>9.7</td>
<td>6.9</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Great Lakes Coastal</td>
<td>65</td>
<td>18,552</td>
<td>285</td>
<td>4,868,312</td>
<td>74,681</td>
<td>3.82</td>
</tr>
<tr>
<td>% of Total</td>
<td>3.2</td>
<td>3.5</td>
<td>3.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other Regions*</td>
<td>251</td>
<td>44,457</td>
<td>177</td>
<td>15,299,322</td>
<td>60,954</td>
<td>2.90</td>
</tr>
<tr>
<td>% of Total</td>
<td>12.4</td>
<td>8.5</td>
<td>9.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PROGRAM TOTAL</td>
<td>2,031</td>
<td>$525,134</td>
<td>$259</td>
<td>$156,941,202</td>
<td>$77,277</td>
<td>$3.35</td>
</tr>
<tr>
<td>% of Total</td>
<td>100</td>
<td>100</td>
<td>-</td>
<td>100</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

* Includes the desert Southwest as well as other non-coastal, non-Great Lakes regions.

Table derived from GAO, 1988
TABLE 2-4

DEFINITION OF NATIONAL FLOOD INSURANCE PROGRAM TERMINOLOGY

"Base flood" means the flood having a one percent chance of being equalled or exceeded in any given year.

"Development" means any man-made change to improved or unimproved real estate, including but not limited to buildings or other structures, mining, dredging, filling, grading, paving, excavation or drilling operations.

"Flood" or "Flooding" means:
(a) A general and temporary condition of partial or complete inundation of normally dry land areas from:
(1) The overflow of inland or tidal waters.
(2) The unusual and rapid accumulation or runoff of surface waters from any source.
(3) Mudslides (i.e., mudflows) which are proximately caused by flooding as defined in paragraph (a)(2) of this definition and are akin to a river of liquid and flowing mud on the surfaces of normally dry land areas, as when earth is carried by a current of water and deposited along the path of the current.
(b) The collapse or subsidence of land along the shore of a lake or other body of water as a result of erosion or undermining caused by waves or currents of water exceeding anticipated cyclical level or suddenly caused by an unusually high water level in a natural body of water, accompanied by a severe storm, or by an unanticipated force of nature, such as flash flood or an abnormal tidal surge, or by some similarly unusual and unforeseeable event which results in flooding as defined in paragraph (a)(1) of this definition.

"Flood Insurance Rate Map" (FIRM) means an official map of a community, on which the Administrator has delineated both the special hazard areas and the risk premium zones applicable to the community.

"Flood plain" or "flood-prone area" means any land area susceptible to being inundated by water from any source (see definition of "flooding").

"Flood plain management" means the operation of an overall program of corrective and preventive measures for reducing flood damage, including but not limited to emergency preparedness plans, flood control works and flood plain management regulations."Flood plain management regulations" means zoning
ordinances, subdivision regulations, building codes, health regulations, special purpose ordinances (such as a flood plain ordinance, grading ordinance and erosion control ordinance) and other applications of police power. The term describes such state or local regulations, in any combination thereof, which provide standards for the purpose of flood damage prevention and reduction.

"Regulatory floodway" means the channel of a river or other water course and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height [usually one foot].

"Scientifically incorrect". The methodology(ies) and/or assumptions which have been utilized are inappropriate for the physical processes being evaluated or are otherwise erroneous.

"Technically incorrect". The methodology(ies) utilized has been erroneously applied due to mathematical or measurement error, changed physical conditions, or insufficient quantity or quality of input data.

Source: FEMA (1987)
The definitions of "flood", "floodplain", and "floodway" appear innocuous; however, they raise concern when applied to arid regions. None of the terms have natural significance. That is to say, the concept of a flood in relation to a recurrence interval (i.e., the base flood, 100-year flood, 500-year flood), and the concept of floodway, as used in the program have no physical significance. The magnitude of a flood can be estimated, predicted, or measured, but the significance of a given flood is not in its recurrence, as much as its damage-causing potential. The designation of a floodway, on the other hand, could have some physical meaning to a geologist or geomorphologist, but not as defined by FEMA.

The two concepts of floodplain and floodway are very often misunderstood and misapplied. Figure 2-1 is intended to help clarify the terms and further illustrate their artificial nature. As presented in Chapter 4, neither of these concepts are valid in the desert southwest where (1) ephemeral streams readily change course during flow events, (2) incised channels rarely experience over-bank flow, (3) erosion is more threatening and damaging than flooding, and (4) alluvial fans are numerous and do not fit neatly into any hazard zone. Sheet flow is common on alluvial fans, penoplains, and floodplains; in sheet flow the water can be everywhere and anywhere.

Federal programs addressing flooding and flood control measures are widespread throughout several federal agencies. Table 2-1 indicates some of the
Figure 2-1
Floodplains and Floodways

NOTE: A = AREA OF ALLOWABLE ENCROACHMENT
primary agencies and their responsibilities. Lack of agreement between agencies occurs from time to time and other agencies are asked to resolve conflict.

This occurred in the 1950's when flood frequency analysis became widespread and different agencies preferred different statistical approaches. The Water Resources Council, a now-defunct independent executive agency of the U.S. government, was called upon to recommend a singular method to be used by all agencies in all federally sponsored programs. The WRC (1967) recommended, and some federal agencies now require, use of the Log-Pearson Type III (LP III) distribution unless special permission is granted to use another method. Use of this statistical concept was reaffirmed by WRC in 1976, 1977, and 1981. Log-Pearson Type III is still the recommended probability distribution for use in flood frequency analysis in the United States.

Selection of statistical methods in hydrologic research has a direct effect in the design of drainage systems by controlling the magnitude of the estimate of the design parameter, i.e., precipitation or discharge. For many purposes, the 100-year flood is used as a design standard and the selection of statistical approach is crucial in determination of 100-year flood discharge. It has been demonstrated, by Reich (1981) for instance, that the choice of distribution can result in range of 3.5 to 1 in the estimated discharge. Adoption of the Log Pearson III method was based upon clear understanding that improved methods would eventually be developed and subsequently adopted by the WRC (1976).
This has not been done although the difficulties and deficiencies of the Log Pearson III have been noted time after time.

Federal Emergency Management Agency

The Federal Emergency Management Agency (FEMA), as administrator of the National Flood Insurance Program (NFIP), has profound impact on the design of drainage systems throughout the United States. FEMA requires use of LPIII in most flood insurance studies. Regulatory discharges - those required by regulation - are then used to delineate the 100-year and sometimes the 500-year flood plains within the study area. Insurance rates are based upon designated "zones" within the study area on a national actuarial basis (FEMA, 1987, 1987a). Local agencies use the flood insurance studies to implement local flood plain management ordinances and as a design basis for flood control projects undertaken by the local agency.

Flood insurance studies must follow a format prescribed by the FEMA (1985). Results of the flood insurance studies contain not only insurance rate maps, but also written text, water surface profiles, figures, tables and other documentation. Consultation with local officials and public review and comment of the study are required by the Flood Disaster Protection Act of 1973. Final approval of the study methodology and recommendations are by federal, not local agencies. The importance of statistical methods and the flood insurance program are discussed from the perspective of arid lands in later sections.
Clean Water Act

Another federal program which bears upon drainage system design is Section 404, Federal Water Pollution Control Act (33 USC 1344), also known as the Clean Water Act (CWA). The act prohibits the placement of unauthorized fill or dredged material in "waters of the United States". According to Lueck (1986), this includes ephemeral streams, washes, and arroyos of the arid southwest up to the "n-th order tributary."

The CWA was enacted to maintain the quality of surface waters and riverine habitats. Although effective in 1972, the law was not enforced in regions with ephemeral streams until 1986, when environmentalists became active in the federally-mandated coordination of plans through the Environmental Protection Agency and the Fish and Wildlife Service. In contrast to other federal efforts that tend to emphasize structural improvements, the Section 404 permitting process, together with the flood insurance program, has partial emphasis on non-structural measures. The CWA 404 permitting process is a responsibility of the Corps of Engineers. The Corps has taken a realistic position in reviewing permit applications, and generally only regulates streams that appear as blue lines on the USGS quadrangles. The blue lines indicate larger channels, and in the desert, they are usually dry.

The Corps' approach allows minor alteration to insignificant dry washes without excessive regulatory burden on the agency or the permit requester. Small scale projects are usually covered under a nationwide or blanket permit.
However, larger projects such as bank protection along more major "blue-line" streams can be delayed by environmental groups and federal or local agencies that are required, authorized, or allowed to comment on a proposed project. The 404 process then becomes a political mechanism to attain environmental goals other than fulfilling the program's primary goal of water quality protection.

U.S. Army Corps of Engineers

The Corps of Engineers has grown from a combat support military unit into a large peace time public works design and construction agency. The Corps undertakes construction of large-scale flood control works in major river basins, in addition to its research and engineering activities. Since the early 1970's, the Corps has been involved in urban flood control, including areas of the desert southwest. The Corps' emphasis has always been on structural control measures, using benefit cost analysis (BCA) as one means of evaluating alternative solutions. The BCA also is used in deciding whether the Corps would participate in a project, and at what level of financial participation.

In 1974, Congress passed Public Law 93-251 allowing federal agencies including the Corps to participate up to 80% in non-structural flood control projects. This significant change in federal policy was the result of intensive objection to a structural project in Colorado. The Army Corps of Engineers coordinates the Section 404 program of the Clean Water Act and is responsible
for other regulatory, construction, and research programs. Through its Hydrologic Engineering Center, the Corps became a leader in many phases of drainage system design. Their computer programs in flood discharge analysis, backwater analysis and other areas are widely used by engineers and hydrologists.

Soil Conservation Service

The SCS is a branch of the Department of Agriculture and many of its programs have traditionally been non-structural or a combination of structural and non-structural. The SCS conducts ongoing research related to range land management, watershed management, erosion, sediment transport, and other aspects of hydrology and runoff. The so-called SCS Method is widely used in estimating runoff in urban and rural areas. The SCS also participates in regional storm water management and conservation programs.

U.S. Environmental Protection Agency

The EPA is primarily responsible for protecting and restoring the country's environment. The agency was created after the enactment of the National Environmental Policy Act of 1969, Public Law 91-190. NEPA also established the Council on Environmental Quality which prepares guidelines for the EPA and other federal agencies. Water resources programs include protecting the quality of ground water and surface water, as required by Federal
Water Pollution Control Act Amendments of 1972. The EPA also deals with sewage disposal and treatment methods, and designates sole-source aquifers in regions that rely entirely on a single aquifer for their domestic water supply.

U.S. Department of Transportation

Transportation projects often affect natural drainage patterns, and about 25% of the funds expended for roadway construction are used to provide drainage facilities, including bridges. Elevated roadbeds act as berms that collect surface flows which are carried under the roadway in culverts or bridge structures. Depressed roadbeds intercept cross drainage which needs to be channeled either over or under the roadway.

The USDOT's Federal Highway Administration (FHWA) is responsible for construction of the federal highway system and for funding, construction, and maintenance of state and local portions of the federal system. The USDOT published guidelines and standards for roadway planning, and for design of roadway drainage appurtenances. Administrative procedures exist to minimize conflicts between flood plain management programs and roadway construction practices. It is sometimes presumed that the intergovernmental-interagency coordination provisions of the Office of Management and Budget and successor programs would provide adequate coordination during project planning phases. This alone is generally inadequate because these programs were intended more to minimize redundancy in federal spending than mitigate conflicting agency
goals. For example, the flood insurance program requires residential protection or elevation above the 100-year flood. However, the USDOT requires only 50-year conveyance in culverts. Property upstream of a highway culvert could become inundated from backwater, thereby creating or increasing the lands subject to flood plain regulations. Federal transportation agencies recognize the importance of hydrology and hydraulics, particularly as applied to transportation system components. The Department of Transportation by itself and in conjunction with State DOT's continues to conduct research in both fields and disseminates its findings widely in journals and government publications.

Executive Orders

Presidential Executive Orders affect federal agencies as if they were laws. Executive Orders are published in the Federal Register along with all major decisions, rules, and announcements of federal agencies. Executive Order 11988, published on May 24, 1977, requires federal agencies to (1) avoid supporting development within floodplains, (2) avoid actions located in or affecting floodplains unless no other choice exists, and (3) require that their actions minimize potential harm to or within floodplains.

The intent of the EO 11988 was interpreted for use of federal agencies by the Water Resources Council. According to a study by FEMA (1982), EO 11988 and the 100-year base flood standard have been effective in reducing flood losses and were supported at all levels of government. Executive Order 11990, also May
24, 1977, deals mainly with the preservation of wetlands and therefore has limited application in the arid portions of the country.

Other Federal Programs

Discussion of other federal agencies' role in the design of drainage systems not discussed above are left to the reader's curiosity. However, these agencies are identified previously in Table 2-1, and for purposes of this paper, their role is minor.

State Laws and Agencies

The states have enacted legislation or follow common (unwritten) law to implement or comply with federal programs. For example, the Arizona Department of Water Resources is designated by Arizona Revised Statutes (1979) as the state agency responsible for implementation of the national flood insurance program. Major responsibilities of the agency include technical and financial support to local flood control districts, audits of local programs, and coordination of local programs between jurisdictions.

Other state agencies involved in drainage system design review, approval, or funding include the Department of Real Estate, Department of Transportation, Land Department, and Department of Environmental Quality. These agencies essentially have roles parallel to their federal counterparts. Other states
undertake similar responsibilities in different or differently-named departments. The general responsibility of state agencies is shown in Table 2-5.

**Local Laws and Agencies**

Most drainage system improvements in smaller watersheds are the responsibility of city or county government. Local taxes such as flood control district revenues, general revenues, transportation funds, and bonds are utilized to implement storm water management plans and construction programs at the local level. Urban development and its associated drainage improvements are the responsibility of counties and cities. Local agencies have state counterparts, as shown in Table 2-6. Many local agencies in the southwest and other areas develop flood plain management ordinances, drainage design standards, engineering plan review and approval procedures, basin management plans, and subdivision (development) regulations. Authority to undertake these activities is necessarily vested in the local agencies as a police power by state law or constitution. Because most urbanizing areas in the southwest participate in the flood insurance program, the drainage system designer must comply with local standards and procedures that respond to federal and state mandates.

Many smaller jurisdictions have few, if any, trained staff to deal with drainage or flood plain management issues. In many cases agency employees must "wing it" or seek advice from their peers, counterparts, or consultants.
### TABLE 2-5

**STATE AGENCIES INVOLVED IN FLOOD CONTROL AND DRAINAGE**

<table>
<thead>
<tr>
<th>AGENCY</th>
<th>FUNCTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Resources Department</td>
<td>Coordinates flood insurance program within state, funds state water projects</td>
</tr>
<tr>
<td>Environmental Quality</td>
<td>Protects environment, surface and ground water protection</td>
</tr>
<tr>
<td>Transportation Department</td>
<td>Develops drainage guidelines, conducts research</td>
</tr>
<tr>
<td>University System</td>
<td>Conducts research, educates hydrologists and engineers</td>
</tr>
<tr>
<td>Legislature</td>
<td>Empowers agencies and political subdivisions, funds protective works and studies.</td>
</tr>
<tr>
<td>AGENCY</td>
<td>FUNCTION</td>
</tr>
<tr>
<td>------------------------</td>
<td>--------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Flood Control District</td>
<td>Flood plain management and regulation, protective works and non-structural programs</td>
</tr>
<tr>
<td>City/County Engineer</td>
<td>Reviews engineering plans for private development, design and construct protective works</td>
</tr>
<tr>
<td>City/County Board</td>
<td>Adopts and enforces zoning and flood control regulations</td>
</tr>
<tr>
<td>Planning Department</td>
<td>Prepares land use plans, enforces zoning codes, issues building permits</td>
</tr>
</tbody>
</table>
Some communities use boilerplate ordinances and design standards distributed by professional organizations or agencies. This is obviously a poor practice in the southwest where special considerations such as erosion hazards and alluvial fans must be addressed. It can be generally stated that the smaller jurisdictions cannot retain highly competent engineers and hydrologists to address drainage issues. However, assistance may be available from state or federal agencies, particularly if the community participates in the national flood insurance program.

**Engineering Guidelines and Standards**

Design guidelines and engineering standards are prepared by numerous professional societies and associations. The level of detail is a function of the organization's special interest. For example, the American Association of State Highway Transportation Officials (AASHTO, 1982) is highway oriented, the National Association of County Engineers (NACE, 1986) is directed towards small, maintenance-oriented county highway departments, and the American Society of Civil Engineers offers technical publications that represent state-of-the-art research and design applications. Manufacturers of drainage system components provide designers with guidebooks for the selection, installation, and maintenance of their products (National Clay Pipe Institute (1974), American Iron and Steel Pipe Institute (1980)). Publications of
professional or manufacturing associations are typically general and directed towards the design engineer or applied hydrologist seeking a safe and effective, though not necessarily optimal, solution to a design problem. Obvious exceptions to this rule are academically-oriented research publications of ASCE, the American Geophysical Union, and other associations cited herein. A wide range of information is available to the engineer, with an equally wide range in ease of use, ease of understanding, and regional applicability.

A common characteristic of this literature is its limited usefulness in semi-arid or arid southwestern states. Special considerations of ephemeral streams, alluvial fans, channel bank erosion, rainfall patterns, and so on are either lacking or discussed only briefly. An engineer "transplant" to the southwest may be used to a higher degree of appropriateness of these standard reference materials. This expectation can lead to design decisions yielding less-than-desirable accommodation of storm water flow.

Once aware of the differences between regions, engineers will probably find a lack of consolidated design information for dealing with the desert drainage, and they may be compelled to rely on agency standards. These standards probably will not reflect desert conditions either for the reasons discussed above, and the designer will be compelled to use less-than-desirable design tools.
Alteration to natural drainage paths by construction of engineered facilities occurs regularly and unavoidably. Alteration of natural flow is the subject of surface water law. There is tremendous variation of surface water law throughout the country, and the southwest is no exception. In the southwest, drainage project design is affected by these laws, and also by ground water protection or conservation laws. Legal issues regarding drainage are complex, if not confusing. The designer should become familiar with two general rules of surface water law. They are (1) the natural drainage rule and (2) the common enemy doctrine. The two have been combined in some states to form the reasonable use rule. The natural drainage rule states that:

"...every landowner must bear the burden of receiving upon his land the surface water naturally falling on land above it and naturally flowing to it therefrom, and he has the corresponding right to have the surface water naturally falling upon his land or naturally coming upon it, flow freely therefrom upon the lower land adjoining, as it would flow under natural conditions. From these rights and burdens, the principle follows that he has a lawful right to complain of others, who, by interfering with natural conditions, cause such surface water to be discharged in greater quantity or in a different manner upon his land, than would occur under natural conditions." (Heier v. Krull, 160 Cal 441 (1911)) From AASHTO (1980)

The common enemy doctrine is at the opposite end of the spectrum of logic. It permits each landowner to fend off surface waters as they see fit. Under strict interpretation of this concept, surface flows are truly deemed to be
enemies and each landowner may fight as he sees fit. This is a harsh doctrine and has been combined with the natural drainage rule to form the more tempered "reasonable use rule". This rule states that the possessor of land incurs liability whenever his harmful interference with surface drainage is unreasonable. The question of what is "reasonable" is usually resolved in arbitration or litigation.

The experiences of professional practice can provide the designer with enough knowledge to avoid litigation relative to inverse condemnation, property damage, tort liability, malfeasance, or other civil or criminal violations of the constitutional rights of property owners. Usually, but not always, fulfilling agency design standards is accepted as meeting standards of professional practice. Anyone involved in the design of drainage projects or floodplain management should be aware of the issues, be familiar with the legal principles, and, if they are not immune to personal suit — as are many government employees — they should know a good lawyer.

Engineering registration generally requires working knowledge of local and state rules and design standards. This knowledge is distinctly different from adequate knowledge of surface water laws in the region of practice. Legal aspects of design are not always complex, and they should not be ignored. Although this subject matter is not part of the formal engineering education, some professional associations have introduced the subject through publications and seminars. Recognition that applied hydrology deals with natural forces and
that these natural forces are variable in time and space should foster an appreciation of legal ramifications of project design. The special concerns of the desert should not be overlooked, but rather incorporated into design philosophy for both structural and non-structural drainage projects.
CHAPTER 3
THE PHYSICAL PHENOMENA

Deserts are formed, expanded, and continued by natural conditions distinctly different from non-arid regions. In order to understand these differences and the hydrometeorological conditions unique to desert environments we must appreciate what deserts are. Kirk (1973) provides the simplest definition of the word "desert" as land where average annual rainfall is 10 inches or less. Semi-deserts have rainfall of more than 10 inches but less than 20 inches per year. Using these definitions, nearly one-seventh of the world is desert and another one-seventh is semi desert. All continents except Europe contain at least some desert lands. The North American deserts include nearly 500,000 square miles of the southwestern United States and northwestern Mexico. The North American desert has been classified by Hastings and Turner (1964) and many others into five distinct regions: Chihuahuan, Colorado Plateau, Great Basin, Mohave, and Sonoran. Deserts of the southwest are shown in Figure 3-1.

The definition of "desert" can be further refined by taking temperature into account. In deserts, high temperature, low humidity, and infrequent rainfall prevail resulting in low soil-moisture and sparse vegetal cover. Delineation of deserts of the world can be approximated by identifying areas without a network of perennial rivers. A better method of delineation,
Figure 3-1

The North American Desert and its Subdivisions
as reported by Simons (1980), is the use of a formula that incorporates average annual variation in regional rainfall and temperature. One commonly used method is that of the German climatologist W. Köppen. The method, as described by Simons, has been used to classify major cities in Arizona as desert or semi-desert and as warm or cold. Results of this classification are shown in Figure 3-2, and it can be seen that the four combinations of these two variables lie close to the Tucson area. This illustrates the dramatic changes in weather conditions that can occur between nearby regions. The benefit of such methods, however, is not so much in identifying variations within a desert region, but in defining desert margins. Climatic extremes such as the American "dust bowl" drought of the 1920's can dramatically alter the range of desert land. Permanent deserts have long records of drought due to separation from cloud moisture sources, proximity of mountain barriers, or other natural reasons. Desertification, the creation of new desert land, is often the result of human alteration of the natural environment. New desert can be created by agricultural practices, surface water diversion, and deforestation.

The global range of desert lands is increasing, and the nature of the deserts of the American southwest is changing. Trends have been tracked and reported by others, but for the southwest, the trend is towards desert expansion. Although these changes are of environmental importance, the economic and social significance is not as severe as in other parts of the world
Figure 3-2
Desert Classification - Arizona Cities
where agricultural land and wood fuel supplies are threatened by desertification.

The deserts of America are expected to expand not only in area, but in population. Urban areas of the southwest are growing at an annual rate of two to four percent. The metropolitan areas of Phoenix, Arizona, and Las Vegas, Nevada, grew about 25% between 1980 and 1986 (Urban Land Institute, 1988). As human presence increases, sensitive desert lands give way to subdivisions, municipal ground water withdrawals increase, and water from exotic rivers - perennial streams flowing through arid regions - such as the Colorado River, are regulated and transported to urban population centers and agricultural districts.

It is also of value to understand how deserts are formed. Miller (1973) states that no single factor created the American deserts - the creation is a combination of atmospheric circulation patterns, rain shadow effects, proximity to ocean moisture sources, and cold ocean currents. The mountains of the southwest cause an uplift of moist air, resulting in precipitation on the windward side. The leeward side is deprived of the moisture, receives little precipitation, and is therefore said to be in the rain shadow. The northern portion of the American desert is caused mainly by rain shadow. The Sierra Nevada and Cascade ranges create precipitation and prevent moist clouds from the Pacific from reaching the Great Basin desert. Further south, the Mohave, Chihuahuan, and Sonoran deserts also lie in the rain shadow of western
mountain ranges, and do not benefit much from tropical precipitation. The Sonoran desert is further influenced by the cold ocean currents along western Baja California.

Rainfall reaches all portions of the American desert, and precipitation patterns are relatively predictable. Winter storms from the north and west bring some moisture despite the rain shadow affect. Moisture for summer storms is supplied predominantly by the Gulf of Mexico, and to a limited extent by the Gulf of California. Pickwell (1939) describes how summer storms reach across the Chihuahuan Desert into southern Arizona. As a rule, winter rains predominate in the northern and western desert region, and summer rains predominate in the southeastern portion of the desert. The two primary storm seasons meet in the Sonoran Desert, thereby producing an intermediate zone with two distinctly different summer and winter rainy seasons. Tucson, Arizona, for instance, lies in this intermediate zone and receives about eleven inches of rainfall annually. Tucson's rainfall is fairly evenly divided between winter general storms and summer thunder storms.

The driest regions of the American desert are at the head of the Gulf of California where the Sonoran desert meets the Gulf. Yuma, Arizona and Mexicali, Mexico, receive total annual precipitation of about three inches. In thirty years of record, Yuma has an average rainfall of 0.00 inches for the month of May, even though Yuma is close to the Gulf of California and the Colorado River (Arizona Department of Commerce (1988)). In comparison to
Tucson, rainfall in Yuma is much less because Yuma is farther from the Gulf of Mexico, is distinctly in the rain shadow of the Sierra Nevadas, and is affected by the cold currents of the nearby Gulf of California. Yuma is also at a lower elevation, and this provides additional time for rain from higher elevations to evaporate before reaching the ground.

There is great diversity of rainfall in different parts of the desert. The physical phenomena of the desert should be understood by civil engineers, and this understanding should be applied in protecting residents from flooding.

Desert Rainfall and Precipitation Patterns

Desert rains and flow in ephemeral channels must be separately understood before one can attempt to understand their inter-relationship. Annual rainfall and its spatial variation are a function of several factors. As described by Miller (1973), these factors include elevation, barriers to airflow, distance to moisture source, location (longitude and latitude) and they are commonly used to explain the difference between recording weather stations.

The National Weather Service (NWS) monitors the weather, analyzes the data, and publishes technical reports covering various regions of the country. Generally, storms have either intense precipitation over a small area, or less intense precipitation over a broader region. Thus, storms are classified as thunder storms (or local storms, if there is little thunder) and general storms.
For much of the Southwest, the two storm types occur during distinct seasons depending on the region of interest. The intensity-precipitation relationship also varies by region. Meteorology of the western states is described by Hansen (1977, 1981) and Miller (1973) and others. The seminal work of Hansen is most significant because of its differentiation of summer thunderstorms and winter general storms.

General storms differ from thunderstorms in several ways. Most importantly, general storms have greater areal extent, longer duration, and less intense precipitation. General storms occur at a different time of year and have different directions of motion and different moisture sources. Also, the magnitude of general storms is affected by orography and convection more than thunderstorms are. General storms may contain thunder storm cells, but the intensity of these thunder storms is not as severe as in isolated local storm cells.

**Applied Meteorology in the Desert Southwest**

Meteorology is important to engineers and hydrologists because rainfall characteristics of a region need to be considered during the design of drainage systems. Sometimes site-specific or regional meteorologic data is gathered for the project. More commonly, however, some readily available alternative or "default" methodology is used in lieu of collecting new data. Default methods
may include transferring data from similar nearby watersheds or using a rainfall prediction method developed by or for a governmental agency. For most civil engineering projects on small and intermediate size watersheds, the later situation is typical. Assumptions about the intensity, duration, and areal extent of a rain storm are essential to the design process. And, these assumptions are often subject to more uncertainty than any other because they address natural phenomenon that are not fully understood.

The most widely used default meteorological studies are prepared by the National Weather Service, a branch of NOAA, and consist of precipitation-frequency relationships. According to Miller, the first general study of the precipitation-frequency relationship for the United States was conducted in the early 1930’s by Yarnell (1935). His study provided rainfall maps for 5-minute to 24-hour durations and frequencies of 2- to 100-years. It served as a basic source of information for engineering design and economic analysis until the 1950’s. The maps were based on data from about 200 first-order weather stations using recording rain gauges. In 1940, the Army Corps of Engineers helped install a network of recording gauges to supplement the existing Weather Bureau network. The additional gauges reportedly increased the amount of short-duration (1- to 24-hour) rainfall data used in subsequent research by a factor of 20.

In the 1950’s the Weather Bureau and other federal agencies examined the depth-area-duration relationship as well as precipitation-frequency based on
the augmented data base. These studies showed a need for additional data in the mountainous regions of the western United States. Comparison of these studies with Yarnell's showed a factor of three difference, Yarnell's being usually higher. Hershfield (1961) combined data for the conterminous United States into a single report commonly cited as Technical Paper 40. The effect of topography was considered only in a general sense in all these studies.

The Precipitation-Frequency Atlas of the Western United States consists of a series of volumes, one each for Montana, Wyoming, Colorado, New Mexico, Idaho, Utah, Nevada, Arizona, Washington, Oregon, and California. The approach used in the atlas is essentially the same as in Technical Paper 40. Precipitation-frequency relationships for any location are estimated from generalized maps based on a simplified relationship between rainfall and recurrence. The maps contained in the volumes are for 6- to 24-hour duration rainfall for 2- to 100-year recurrence intervals. There is no attempt in the Atlas to segregate rainfall records by season or areal extent. All records are statistically aggregated using the Gumbel procedure for fitting annual series data to the Fisher-Tippett Type I probability distribution, a special case of the Pearson Type III distribution. The Atlas is the standard default precipitation estimation technique for many agencies and is incorporated in hydrologic methods such as those of Pima County (1979) and the City of Tucson (1982).

It is important to note that, although widely used in drainage design, these default methods are based upon reports that express concern about the
applicability of the methods to regions where seasonal variation in storm patterns occur. TP 49 states that seasonal variations were not presented for states west of the Rockies "...because of the effects of local climatic and topographic influences" and that "[s]easonal probability curves were not derived... because the relatively small number of stations providing the basic data preclude the delineation of the boundaries of areas of representativeness for seasonal probability curves." Similarly, Technical Paper 40 states:

"[A] practical problem concerned with seasonal variation may be illustrated by the fact that the 100-year 1-hour rain may come from a summer thunderstorm, with considerable infiltration, whereas the 100-year flood may come from a lesser storm occurring on frozen or snow-covered ground in the late winter or early spring... No seasonal variation relationships are presented for the mountainous regions west of 105 W. because of the influence of local climatic and topographic conditions. This would call for seasonal distribution curves constructed from each station's data instead of average and more reliable curves based on groups of stations" (TP40, page 7)

Most important is the following statement of the Atlas:

"The maps of this Atlas are based upon data for the entire year. In certain sections of the West, precipitation is highly seasonal. Thus, rainy season precipitation-frequency values approach the annual values... Generalizations about the seasonal distribution of large storms can be obtained from ESSA, U.S Weather Bureau Technical Paper No. 57 (Environmental Science Services Administration, Weather Bureau, 1969). Currently, there is no convenient manner of applying this knowledge to maps of this Atlas, other than subjectively."

There is great variance between desert thunder storms and general storms - not only in season of occurrence, but in depth, area, and intensity of
rainfall. Local storms can have durations ranging from about 10 minutes to 6 hours, whereas general storms can drizzle for several days. The NOAA Atlas contains procedures for developing short duration frequency-recurrence estimates, but they are based on the segregated annual precipitation data that are not separated by season. The procedures rely on empirical methods and the factoring of longer duration (6- and 24-hour) events to obtain estimates of the one-hour event. Then, the one-hour event is again adjusted to obtain values for even shorter duration events. This round-about method is perhaps better than nothing, but its acceptance for design and planning purposes certainly does not stimulate the collection of "better" localized rainfall-frequency data. If one disregards issues about the accuracy of probability distribution functions, it can be reasoned that the procedures of TP 40 and the NOAA Atlas probably underestimate local storm intensity and overestimate general storm intensity for the arid southwest.

Hansen (1977) presents methods to compute estimates of the probable maximum precipitation (PMP) of any watershed up to 5,000 square miles for durations up to 72 hours in the desert southwest. The approach varies markedly from Miller, both statistically and meteorologically. A major difference is that PMP is estimated, not a 100-year rainfall. PMP is defined by the American Meteorological Society (1959) as "the theoretically greatest depth of precipitation for a given duration that is physically possible over a particular drainage basin at a particular time of year". This definition, in
contrast to the 100-year event, is a function of location, time, and area and is frequency independent. PMP is based not only upon historic rainfall records, but also upon the science of meteorology and (alas) the principles of statistics.

In a nutshell, general storms are estimated by separately predicting convergence PMP (rainfall due to atmospheric processes) and orographic PMP (rainfall due to mountain slopes). The two components, calculated for a specific location, basin size, and time of year, are then added to estimate total PMP. The approach used for local or thunder storm PMP is adjusting the most intense rainfall for maximum moisture and then developing a 1-hour, 1-square mile PMP map. Therefore, the approach separates intense thunder storm predictions from general storm predictions. The approach recognizes that the local storm is the greatest potential rainfall threat for small desert watersheds and for short durations on portions of larger watersheds. It recognizes also that thunder storms are isolated spatially and temporally in the southwestern states, whereas less intense thunder storms occurring within general storms constitute the convergence portion of general-storm PMP.

The counterpart of HR 49 is HR 51, which covers the states east of the 105th meridian. The procedure in each report is almost identical, and according to personal communication with Henz (1988) and Hansen (1988), HR 51 is accepted and widely used within its study area, whereas HR 49 generally is not. HR49 is not widely referenced in the literature. This is unfortunate because the methodology of HR 49 seems to be much better suited
for the desert southwest than the procedures and methods of the NOAA Atlas. Several reasons could explain its lack of acceptance: (1) it uses PMP instead of the typical agency-mandated 100-year event, (2) it produces higher, yet more realistic, rainfall prediction for small basins, and (3) it was published after the NOAA Atlas, TP 40, and TP49. Rejection of HR 49 for these reasons is invalid because (1) a reasonable methodology is presented for estimating 100-year rainfall from PMP, (2) the most defensible, not necessarily the lowest, default values should be used, and (3) ordinances, standards, and procedures of governmental agencies should be updated whenever "better" methods come along.

Recognizing that default precipitation values often must be used in drainage design and flood plain management, a direct comparison of various methods might seem useful, but would be inappropriate because the methods can not be compared directly. HR 49 produces PMP as a function of area and duration for a given location. This is shown in Table 3-1 for the Tucson area. The table also shows that for the Tucson region, the local storm dominates basins less than about 100 square miles, and the general storm dominates basins larger than 500 square miles. Basins between 100- and 500 square miles should be evaluated with both local and general storms to see which has the worst impact. The NOAA Atlas 100-year precipitation is shown in Table 3-2, also for a Tucson location. The methods of the NOAA Atlas and HR 49 are tested on hypothetical watersheds differing by orders of magnitude.
### TABLE 3-1

**PMP FOR TUCSON AREA**

#### LOCAL STORM PMP (Inches)

<table>
<thead>
<tr>
<th>AREA (MP)</th>
<th>1/4</th>
<th>1/2</th>
<th>3/4</th>
<th>1</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8.5</td>
<td>10.2</td>
<td>10.9</td>
<td>11.5</td>
<td>15.0</td>
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<tr>
<td>5</td>
<td>7.1</td>
<td>8.8</td>
<td>9.6</td>
<td>10.2</td>
<td>14.0</td>
</tr>
<tr>
<td>10</td>
<td>6.2</td>
<td>8.0</td>
<td>8.7</td>
<td>9.4</td>
<td>13.2</td>
</tr>
<tr>
<td>20</td>
<td>5.4</td>
<td>7.0</td>
<td>7.8</td>
<td>8.5</td>
<td>12.3</td>
</tr>
<tr>
<td>50</td>
<td>3.9</td>
<td>5.5</td>
<td>6.2</td>
<td>6.9</td>
<td>10.8</td>
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<tr>
<td>100</td>
<td>2.8</td>
<td>4.1</td>
<td>4.9</td>
<td>5.6</td>
<td>9.3</td>
</tr>
<tr>
<td>200</td>
<td>1.9</td>
<td>3.0</td>
<td>3.6</td>
<td>4.1</td>
<td>7.7</td>
</tr>
<tr>
<td>500</td>
<td>1.1</td>
<td>1.7</td>
<td>2.2</td>
<td>2.6</td>
<td>5.3</td>
</tr>
</tbody>
</table>

#### GENERAL STORM PMP (Inches)

<table>
<thead>
<tr>
<th>AREA (MP)</th>
<th>6</th>
<th>12</th>
<th>24</th>
<th>48</th>
<th>72</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>7.6</td>
<td>9.8</td>
<td>12.1</td>
<td>14.4</td>
<td>15.5</td>
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<tr>
<td>200</td>
<td>7.0</td>
<td>9.1</td>
<td>11.4</td>
<td>13.6</td>
<td>14.7</td>
</tr>
<tr>
<td>500</td>
<td>6.3</td>
<td>8.3</td>
<td>10.5</td>
<td>12.7</td>
<td>13.7</td>
</tr>
</tbody>
</table>

**NOTE:** To approximate 100-year precipitation, use 40% to 60% of PMP.
## TABLE 3-2

NOAA ATLAS PRECIPITATION
TYPICAL TUCSON LOCATION

**PRECIPITATION (INCHES)**

<table>
<thead>
<tr>
<th>DURATION (HR)</th>
<th>STORM RECURRENCE INTERVAL (YEARS)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td>24</td>
<td>4.2</td>
</tr>
<tr>
<td>6</td>
<td>3.4</td>
</tr>
<tr>
<td>3</td>
<td>3.0</td>
</tr>
<tr>
<td>2</td>
<td>2.8</td>
</tr>
<tr>
<td>1</td>
<td>2.6</td>
</tr>
<tr>
<td>1/2*</td>
<td>2.0</td>
</tr>
<tr>
<td>1/4*</td>
<td>1.5</td>
</tr>
</tbody>
</table>

* Estimates based on Pima County (1979)
(1000, 100, and 10 square mile areas are used). For the HR 49 approach, PMP is estimated, then it is adjusted according to Hansen to determine the 100-year precipitation. The Pima County method is tested only on the small watershed because of areal limitations cited in that methodology. The variation between the methods also illustrates the previous statement that combining rain-fall data of all seasons skews the results of the TP 49 and NOAA Atlas methods.

Without a doubt, accurate long-term local data should be used to estimate precipitation whenever possible (Bax-Valentine, 1988). These data seldom exist for points (i.e. isolated record stations) much less for recording networks. The latter is particularly important for small basins that are effected by local storms. One desert watershed that has been monitored for over twenty years is Walnut Gulch, located near Tombstone, Arizona. The Soil Conservation Service (SCS) records rainfall, runoff, and sedimentation within the 58 square mile study area. Numerous technical analysis have been published by Renard, Osborn, Lane and others based on data from this experimental watershed. Extreme rainfall intensity observations for Walnut Gulch and Tucson are shown in Table 3-3.

**Areal Distribution of Rainfall**

The areal distribution of rainfall is affected by location, season, and other factors. For a given recurrence interval, it is axiomatic that area and intensity are inversely proportional. In other words, for a given location in the
### TABLE 3-3

**PRECIPITATION EXTREMES IN SOUTHEASTERN ARIZONA**

**MAXIMUM OBSERVED POINT RAINFALL (Inches)**

<table>
<thead>
<tr>
<th>Duration (Hours)</th>
<th>Phoenix, Arizona</th>
<th>Tucson, Arizona</th>
<th>Walnut Gulch, Arizona</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4</td>
<td>1.17</td>
<td>2.75</td>
<td>1/8 2.7 3.1 3.52</td>
</tr>
<tr>
<td>1/2</td>
<td>1.27</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3/4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 From various sources (actual duration 35 min)
2 From Osborn and Renard (1988)
American desert and a given recurrence interval, a storm of 1 square mile will be more intense and of shorter duration than a storm covering 10, 100 or 1000 square miles. Conversely, storms of less than 1 square mile could be even more intense. Many rainfall estimation methods, including the NOAA Atlas and HR 49, provide means to adjust for area. Figure 3-3 shows the storm pattern used by Hansen (1977), and it is very similar to the shape used by Osborn (1970) for the area near Tombstone, Arizona.

Rainfall-Runoff Relationships

Runoff is created whenever rainfall exceeds surface storage, infiltration, evaporation, and transpiration capacities of a drainage basin. The generalized relationship between rainfall and runoff is discussed in most hydrology textbooks. Quantification of the relationship is the difficult task assumed by the science of hydrology and many hydrologic methods to evaluate this relationship have been developed over the years. Some are basic and simple; others are sophisticated and complex. All have limitations in their application and none can give an exact "answer." The best that can be expected is a reasonable answer which can be used in decision making with some degree of confidence.

Desert soils range from highly porous sands to impervious caliche and clay layers and rock. Antecedent moisture content (AMC) of a soil is a
Figure 3-3

Idealized Local - Storm Isohyetal Pattern
function of soil type and time between rain storms. The greater the AMC or the more impervious a soil is, the more runoff will result if all other factors are the same. For impervious soils, of course, antecedent moisture content is of little interest compared to well-draining soils. In drainage design, the proposed land use changes within a watershed are important because of their effect on impervious cover, and therefore, upon runoff. Surface storage of rainfall occurs in natural depressions, within drainage channels upstream of the point of analysis, and in man-made detention or retention basins. Natural detention is usually diminished by urbanization and is sometimes replaced with artificial detention.

Surface detention, channel storage, and antecedent moisture content are the primary variables of most mathematical equations used to estimate the amount of runoff and peak discharge from a catchment for a given rainfall. Frequently-used analytic procedures are described in Chapter 4 and include the SCS Method, HEC-1, and the Rational Method. The common element of each method is their reliance on an assumed rainfall for the design storm as model input; therefore, each method consists of two components - rainfall and runoff. They also assume that a direct relationship between rainfall frequency and runoff frequency exists, i.e., a 10-year rain causes a 10-year flood.

Drainage design and flood plain management frequently involves an ungauged catchment where the relationship between rainfall and runoff has not been the subject of field analysis. Actual rainfall records within the
catchment may be short-term or nonexistent. Drainage outlets are infrequently monitored relative to runoff volume and discharge rates. Soil types, as a proxy for measured infiltration rates, are estimated from generalized soil classification studies. Natural and artificial depression storage are estimated from available topographic maps at contour intervals that do not afford much accuracy for this purpose.

The rainfall-runoff relationship must be estimated by use of a hydrologic "model" to estimate basin response to a set of precipitation events. Just as precipitation is assumed to follow some default conditions such as described in HR 49 or the NOAA Atlas, runoff is assumed to follow the response of a hydrologic model. In estimating basin factors such as soil type, moisture content, and storage volume, nothing is "known"; assumptions are usually based on available information according to the best judgment of the designer, or they are mandated by a regulatory agency.

Probability Theory and Statistics

Statistics play an important role in estimating the magnitude of the design storm on ungauged watersheds. The accepted practice is to "fit" available (and often limited) rainfall data to a probability density function or frequency curve arbitrarily chosen by someone. The data are extrapolated to extreme values on specially prepared graph paper, or mathematically. Once
plotted, precipitation for any desired frequency or recurrence interval can be estimated for the distribution function used. Of the many statistical methods available, the federal government suggested the LPIII distribution for use in federal or federally-sponsored programs in the hope that the third parameter would prove useful. Selection of a distribution curve has a direct and important impact on the magnitude of the selected design storm.

A common problem in statistical hydrology is the assumption of direct correlation between precipitation "frequency" and runoff "frequency". This is a simple assumption to make, but it can lead to even more imprecision in the design discharge estimate. It should be clear from the previous discussion about rainfall and runoff that they are distinct natural phenomenon. The only valid generalization is that precipitation might cause runoff, and will do so only if storage and infiltration are exceeded. Infiltration rates, antecedent moisture conditions, and sometimes depression storage and time of concentration can vary over time. A rainfall of a given frequency without regard for both area and duration will not necessarily produce runoff of the same frequency even on a totally impervious watershed having no depression storage or other forms of abstraction. Few catchments meet these conditions, and it is safe to state that the probability of a 100-year storm causing a 100-year runoff is less than 1.

To demonstrate this concept, consider a watershed, say 1 square mile, and a storm with a core of 1+ square mile centered on the watershed with a
maximum intensity and a duration of the time of concentration. The hydrograph would be as shown in the example hydrograph of Figure 3-4. If there is no infiltration, the maximum flood \( (Q_{\text{max}}) \) would be ...

\[
Q_{\text{max}} = i_{\text{max}}A,
\]

Where \( i_{\text{max}} \) is maximum rainfall intensity, and \( A \) is area.

If there is infiltration, the maximum flood would be ...

\[
Q_{\text{max}} = Ci_{\text{max}}A \text{ or } = (i_{\text{max}}-f)A,
\]

Where \( f \) is the infiltration and \( C \) is a coefficient of runoff less than unity.

Now a lesser flood could result from a lesser \( i \) - everything else being the same. Note that \( C \) would be less if \( f \) was the same. A lesser flood would also result from a smaller area storm so \( i(\text{average}) \) is less than \( i(\text{point}) \), or the storm not covering the watershed or the storm not positioned "best" on the watershed. A lesser flood could also result from shorter duration storms which don't result in the entire watershed contributing flow to produce the maximum asymptotic flow rate.

This last effect - that of time - is perhaps the most important, even for quite small watersheds. All of these aspects reduce the \( C \) value - but not as simply as the textbooks say. Note that the same flood can result from different storms of different intensity, area, distribution, and position, and duration and time variation, and infiltration, and slope (and \( Q \) which effects \( T_c \).
Figure 3.4
Example Hydrograph

RAINFALL SUPPLY ($I_{\text{MAX.}}$)

$Q = I_{\text{MAX}} A$

WITH INfiltration ($f$)

$Q = (I_{\text{MAX}} - f) A$

t

TIME

Q = EXCESS RAINFALL

L.A.
Probable Maximum Precipitation

The use of statistical methods in applied hydrology has been a topic of debate for years, and, although many universities require a course in statistical methods in the undergraduate civil engineering curriculum, few engineers are comfortable with the subject. Use of statistical methods to extrapolate data is an uncomforting practice for which there are few acceptable alternatives. One concept that is popular with some agencies and individuals is "probable maximum precipitation", as defined earlier. The estimation of PMP is based more on the science of meteorology and less on statistics than frequency analysis is. However, PMP does not produce the relationship between rainfall and frequency that is usually mandated in project design. The "100-year" storm is not a product of the analysis. There are many detractors of the PMP concept, mostly those like Yevjevich (1968), who are inclined towards probabilistic hydrology instead of deterministic hydrology. Their objection is founded in the statistical concept that every probability must have an assigned value between zero and one. Their objection is therefore one of semantics, and the abuse of the word "probable".

This sensitivity is unfounded because there is an upper limit to the magnitude of natural events in our finite world (at least for practical purposes). Use of the word "probable" in the concept of PMP should not be construed as a technical adjective, but merely as a subjective, non-technical qualifier. PMP
is simply a best guess at a worst-case rainfall for a given area at a given time of year. Others suggest different terminology for the same PMP concept to get away from this issue - words like "large" or "Noah event". Perhaps a nested acronym such as BIG precipitation, meaning "best informed guess" would be satisfactory. Other analytic procedures include the recently reported application of partitioned multi-objective risk method (PMRM) to water resources problems (Karlsson, 1988). Unfortunately, the newer methods seem more complex and less understandable than existing practice, and widespread acceptance and understanding of any one statistical approach is not likely.

Statistical methods are nothing more than an approach for the management and use of available data. Engineers and hydrologists have complained about the short length of hydrologic and meteorologic data for decades, and comments similar to those in Table 3-4 abound in the literature, as well as in the vintage work contained in Appendix A and Appendix B. The situation can not improve until hundreds of years of record are established, or until global hydrology is completely understood. Neither is likely in this lifetime. Tools, however, can be borrowed from other scientific disciplines to help understand extreme past events and the probabilities of their recurrence, as discussed in Chapter 4.
"The great want of well-established data, and of practical rules deduced therefrom, for the guidance of calculations as to the maximum flood discharge from catchments of various characters, is too well known to require much comment... With a view to obtain information on these subjects, I had several rain-gauges prepared about a year ago, of a very cheap but sufficiently accurate construction, and got them fixed up at any stations where I had assistants who could attend to them."

Thomas J. Mulvany, 1851

"The numerous observations and measurements of the rainfall and contemporaneous flood discharge of a number of large sewers which are given in full detail below, will doubtless suffice to point out the necessity of securing the rainfall data needed in the consideration of the discharge from surfaces by means of automatic and self-registering devices, instead of trusting to the judgement of even the most careful and well-trained observers."

Emil Kuichling, 1889

"Practically no detailed information has been published in regard to the areas that may be expected to receive high rates of precipitation for short periods... The lack of sufficient gauges, as well as operating difficulties, made it impossible to obtain records of value on the distribution of intense rains."

Frank A. Marston, 1924

"With rainfall and stream flow, the controlling factors become so complicated that it is impossible to evaluate them, at least with the present limited knowledge of the causes of natural phenomena. One cannot expect, therefore, to determine a law or formula by which to calculate the rainfall at any given date, but some other line of approach to this problem must be sought... [T]he hydrologist can examine the rainfall records and compute the probability of occurrence of a given intensity of rainfall."

H. Alden Foster, 1924
Because hydrology is basically a collection of phenomena related to distributions in time and space of random variables of water quantity and water quality, probability theory is the main branch of mathematics applicable to hydrologic phenomena.

Vujica Yevjevich, 1972

Minimal progress [in the field of hydrology] has been made when methods that were useful before the availability of computers have been programmed for computer use. The greatest progress has come from and will continue to come from efforts which recognize that computers are machines that can augment human minds... We cannot expect to extract much more information from our past measurements.

Stephen J. Burges, 1986
Stream Bed Abstractions

The desert contains countless ephemeral streams, many of which are highly porous infiltration galleries to the underlying aquifer. Flow in ephemeral channels can infiltrate over relatively short reaches of stream bed. Excess rainfall on the upper portion of a watershed may totally infiltrate within the streambed and not reach a downstream concentration point. In sharp contrast, perennial streams have a base flow resulting from constant inflow of runoff from tributary streams and from ground water. Additional storm runoff is added to the base flow, as shown in Figure 3-5, and the time-variant discharge - the peak - travels downstream. By definition, ephemeral streams are usually dry. Either they have no base flow or infiltration is so high that this abstraction can be considered the opposite of base flow, i.e. a negative base flow condition.

The concept of the extreme infiltration loss is presented by Lane (1982), and represents a new emphasis on the importance of this characteristic of desert streams in understanding the rainfall-runoff relationship. Although infiltration also occurs on perennial streams, it is a storage phenomenon which attenuates the hydrograph. Loss of flow is a more significant factor on ephemeral streams with a steepening of the rising limb of the hydrograph. Figure 3-6 shows a normal stream hydrograph (dashed line) and the result of uniform linear channel abstraction (solid line hydrograph). In contrast to a perennial stream
Figure 3-5
Base Flow - Perennial Stream
Figure 3-6
Ephemeral Stream Negative Base Flow
where runoff adds to base flow to create a higher discharge, ephemeral streams have increasing and then diminishing downstream discharge. At some point downstream the flow is often zero. The stream bed base flow (abstraction) can therefore be considered a negative value.

Changes in Watershed Conditions

Watershed conditions change naturally or through human alteration. Natural changes include soil erosion and deposition, channel meander, avulsion, and vegetation changes. These changes can occur almost instantaneously during a stream flow event, or may take millennia to be noticeable. Some changes are cyclic and some are not. Manmade changes include alterations to topography, impervious cover, vegetation, diversion of flow, and "improvement" to natural water courses. Bouvette (1982) describes how extreme changes within an urban watershed of Houston may not only change runoff characteristics of the basin, but also alter local rainfall patterns. One of the more interesting phenomena of the southwestern desert is the formation of arroyos. Arroyos are alluvial channels that have almost vertical banks of collapsible sand mixtures. They are created by very complex sediment transport processes that result from both natural and manmade basin alterations. There are historic accounts that attempt to pinpoint the date that certain streams eroded to become arroyos and to relate the new arroyo with some singular cause. These simplistic explanations have
been generally discounted by more thorough analysis by Cooke (1976) and others. It can be stated, however, that activities such as human modifications to the surface of a watershed or lowering of ground water tables exacerbate this sedimentation process. The newly-formed arroyo is at a somewhat different slope and elevation, and has different cross section to carry flow than its previous configuration. In addition, once formation begins, the arroyo may continue upstream, downstream, or in both directions.

Arroyo formation is not only interesting, it can also be expensive. Roads, bridges, utilities, pipelines, and related infrastructure crossing an ephemeral stream may be threatened by arroyo formation. Tributaries may be affected by concomitant headcutting, and manmade drainage or irrigation structures may not perform satisfactorily after the bed lowering. Arroyos are strictly unique to desert lands, and probably can not be prevented. The best we can expect is to understand the phenomena and plan accordingly in the drainage design process.
Hydrology is one of the oldest fields of scientific study. According to Biswas (1970), large dams were constructed as early as 2800 B.C. Egyptians priests predicted flooding of the Nile by the position of certain stars, and irrigation systems were constructed by many early societies, including Indians of the American southwest. The earliest attempts were directed more towards controlling than understanding nature. These early structural improvements functioned for short periods before being destroyed by unexpectedly large floods. Then, a more detailed inquiry was needed to help determine what went wrong.

Hydrology is a bona fide earth science that seeks to understand and explain naturally-occurring phenomena discussed in the previous chapter. Man is affected by the forces of nature, including flooding. Primitive cultures are primarily agricultural, and they rely on rain or surface flow to produce crops and care for cattle. Advanced civilizations rely upon farmers to produce food crops and government to provide reliable water supplies for agricultural production through the construction of water resource projects. In urban areas, surface waters are considered more a liability than an asset. Storm water runoff is increased by urbanization, and these flows must be safely carried away from cities to some downstream location. Hydrologists, by definition, are
not designers. They are studiers, researchers, and investigators. Engineers, therefore, are routinely asked to design storm water conveyance systems to protect the health and safety of the public and must borrow from, as well as contribute to, the science of hydrology to do so.

**Applied Hydrology**

The application of the earth science of hydrology to engineering design is known as applied hydrology or engineering hydrology. The goal of applied hydrology is to help determine reasonable design parameters for a water resources project. These parameters are then used by a flood plain manager or hydraulic engineer to fulfill the goals of the project. Project scope can range from large-scale multi-purpose reservoir construction to non-structural flood plain management programs. In many design projects, such as water supply or irrigation, low flow is often as important as peak discharge. In single-purpose flood control projects the sole design parameter is peak discharge. Design parameters are always estimates, and the accuracy of the estimates are limited by both data availability and lack of understanding of the physical phenomena effecting the rainfall-runoff relationship. Applied hydrology is traced to the late seventeenth-century, when members of scientific academies reported the theories and findings of their members. Precipitation gauges came into use in the eighteenth-century, and the well known equations
of Chezy, Darcy, Kutter, and Manning, among many others, were developed during this period. Peak-runoff prediction equations were needed in sewer and drainage design, and several were developed in the 19th century before extensive rainfall and stream gauge data became available. The most frequently encountered of these equations, as reported by Hjelmfelt and Cassidy (1975), are:

- Meyers equation: \( Q_p = bA^{0.5} \)
- Talbot equation: \( a = cA^{0.75} \)
- Burkli-Ziegler equation: \( Q_p = Ami(S/A)^{0.5} \)

where...

\( A \) = drainage area
\( a \) = culvert area
\( S \) = stream slope
\( i \) = rainfall intensity
\( m, b, c \) = coefficients

and,

\( Q_p \) = peak discharge

The Rational Method

The most widely used method is the so-called rational method first introduced by Mulvany (1852). The first reported use of the method in the United States was by Kuichling (1889) in a combined storm/sanitary sewer project. The rational method, also known as the Lloyd-Davies equation in
England, is so widely used that virtually every water resource related engineering text and civil engineering handbook explains it. In discussing the history of the rational method, Biswas (1970) also suggests that, in fairness, it should have been called the Mulvany Method.

The Rational Method is an important part of the history of applied hydrology, but its development is not often discussed in the literature. The method was developed to help design sewer systems at a time when combined sanitary/storm sewers were commonly used. Backflow into occupied structures could occur if storm inflow exceeded conveyance capacity. A better technique was needed to establish design parameters, and the familiar equation was developed. The term "rational method" apparently comes from a comment in the concluding section of Kuichling’s work wherein he states...

"In the choice of the several processes of estimating the required capacity of a combined sewer for a populous district, it must be remembered that with the heavy rains of frequent occurrence in this country, the proportioning of sewers by Hawksley’s formula has usually resulted in floodings, and that an extensive experience with the other formulas has not yet been gained. The above investigations, moreover, show that larger quantities of storm-water run off from urban surfaces than is commonly supposed, and hence it is obvious that a more rational method of sewer computation is urgently demanded. Much room for improvement in this direction is still left, and it is sincerely hoped that the efforts of the writer will be amply supplemented by many valuable suggestions and experimental data which other members of the Society may generously contribute." (Emphasis added)
The seminal papers by Mulvany and Kuichling are worthwhile reading even today. However, they are relatively difficult to obtain and therefore have been reproduced in the appendices of this paper.

The rational method is frequently criticized for its limitations and inaccuracy, and other "more sophisticated" computer-based methods are in vogue. When properly applied by an experienced engineer, however, the rational method is as good as the others and about as good as one could expect (Hiemstra, 1967). Criticism of the rational method notwithstanding, the method may be viewed as the basis for all readily accepted hydrologic methods is use today. The SCS method, HEC-1 and regional methods such as those adopted by Pima County are nothing more than modifications of the rational method. Many agencies restrict use of the rational method without prior approval. A notable example is FEMA, which specifies in its Flood Insurance Study Specifications for Contractors (FEMA 1985) that "[e]mpirical equations, such as the rational formula ... must not be used unless approved by the [project officer]."

The rational method is deceptively simple looking and somewhat tricky to apply correctly. In the method,

\[ Q = CIA \]

where...

- \( Q \) = peak basin discharge in cubic feet per second (cfs)
- \( C \) = runoff coefficient (dimensionless)
- \( I \) = rainfall intensity (inches per hour)
\[ A = \text{drainage area in acres} \]

Although the units are inconsistent, a factor of 1.0083 converts inch-acres per hour to cubic feet per second. The conversion factor is usually rounded to 1.0 and dropped from the expression in recognition that other assumptions - precipitation, time of concentration - are not all that precise in any hydrologic method. Application of the rational method is presented in most civil engineering and hydrology handbooks and textbooks. The trick to using it well is learned by experience and with adequate knowledge of the watershed being studied. Watershed area is not difficult to determine with some precision from topographic maps and aerial photographs. Rainfall intensity and runoff coefficients are very difficult to estimate and are usually based on published tables or other default methods. It is important to know the inherent assumptions and limitations of any hydrologic method before it is applied to a design or planning problem. The rational method is no exception. It is interesting to note that the method has remained virtually unchanged since Kuichling's publication and his request for additional improvements by subsequent investigators.

Mulvany and Kuichling discuss the general relationship between rainfall and surface conditions within the watershed. The basic assumption is that runoff is proportional to the product of area and rainfall intensity.
Expressed mathematically...

\[ Q \sim iA \]

where ...

\[ Q = \text{discharge} \]
\[ i = \text{rainfall intensity} \]
\[ A = \text{watershed area} \]

However, most basins contain impervious cover, and the relationship can be modified accordingly...

\[ Q \sim (i - f)A \]

where \( f = \text{infiltration} \)

The relationship can be made an equality by using a constant of proportionality...

\[ Q = C_1 iA \]

where \( C_1 = (i-f)/i \)

and \( C_1 \) varies from 0 to 1.

If \( i \) is the maximum point rainfall rate, then a further refinement can be provided...

\[ Q = C_1 C_2 iA \]

where \( C_2 = i(\text{average})/i(\text{point}) \)
and \( C_2 < 1 \)

If \( i \) varies with time and has a duration longer than the time of concentration \( t_c \), then...
\[
Q = C_1 C_2 C_3 i A
\]

where \( C_3 < 1 \)

And, if \( i \) has a duration less than \( t_c \), then portions of the runoff will not contribute to peak flow...
\[
Q = C_1 C_2 C_3 C_4 i A
\]

where \( C_4 \sim t/t_c \)

\( t = \) duration of rainfall

and \( C_4 < 1 \)

All four of these coefficients are probably time variant, further compounding what is usually assumed to be a simple relationship. In practice, the equation is simplified by reducing the runoff coefficients to a single constant, and a value for \( C \) is usually selected from a handbook, or by "experience". Kuichling suggested using a runoff coefficient that represents the per cent of impervious cover within the catchment. This over-simplification is present in current applications of the rational method.

Major assumptions of the rational method include:

(1) Peak discharge for a catchment is directly proportional to drainage area, average rainfall intensity during the time of concentration, and physical
characteristics of the basin. This is the rational method stated in words rather than an equation.

(2) The recurrence interval of the discharge is the same as the recurrence interval of the rainfall. This is a common assumption in all frequently used hydrologic methods. As presented in Chapter 2, this assumption is not always valid because some basin attributes - antecedent moisture content, for instance - vary with time and the same rainfall can result in significantly different runoff volumes and peak outflows.

(3) The variables C, i, and A are independent and should be estimated separately. This assumption is also simplistic because rainfall intensity can be a function of drainage area and the runoff coefficient probably changes during a flow event as soil moisture increases.

(4) Rainfall duration equalling the time of concentration yields the maximum discharge rate. This further assumes a constant distribution of rainfall over the basin and that there is little, if any, rainfall before or after the design event. This is rarely true because rainfall intensity varies during a storm, both in time and in space.

Advantages of the rational method include its relative ease of use, availability of default precipitation data, and its widespread acceptance in engineering and regulatory practice. Major limitations include its inability to provide runoff volumes or hydrographs, and its frequent misuse. Although the
rational method appears simple, its use requires considerable skill and judgement to produce reasonable and defensible answers.

Variations of the Rational Method

Common modifications to the rational method facilitate the estimation of runoff volume or the development of a runoff hydrograph to help size detention or retention basins. The Modified Rational Method (MRM) assumes the period of rainfall averaging to be the duration of the design storm rather than the time of concentration, and that urban runoff hydrographs can be typified as triangular or trapezoidal. The latter assumes that tributary area increases linearly with duration until the entire catchment contributes to discharge. Application of the MRM is discussed by American Public Works Association (1981) and other references.

The Soil Conservation Service (1975) method is described in Technical Release Number 55 (TR55). This method can be used to estimate both peak discharge and runoff volumes. Basin characteristics are specified in a manner more complex than the simple runoff coefficient, C, of the rational method. In the SCS method, a curve number (CN) is determined based upon the extent of impervious cover and the type of soil within the basin. After the design recurrence interval, basin area, slope, soil type, and percent imperviousness are determined, a series of charts and graphs are used to manually estimate basin discharge rates. Rainfall is estimated by any appropriate means as an
exogenous input to the SCS procedure. The manual SCS method is based on simulations made with the TR-20 computer program (SCS, 1965) and has been widely adapted for local use by governmental agencies.

The SCS method is based on assumptions similar to the rational method. The most obvious change is a more detailed consideration of the physical characteristics of the basin. Application of the method is limited to basins less than 2000 acres (about 3 square-miles) in area. It also assumes that a standard rainfall type is used, such as the 24-hour Type II rainfall hyetograph. Antecedent moisture conditions are assumed to be average, but can be adjusted for drier or wetter conditions.

Other methods, such as the British Road Research Laboratory (BRRL) method, are based on empirical data and provide reasonable results when catchments are small and fairly impervious. The inherent assumptions in all these models, however, are not markedly different from the original rational method and must be used with care, and in recognition of their limitations. None of the methods provides a rainfall input procedure. That critical piece of information is left to the user to find. And although there is often much discussion about the most "appropriate" discharge method to use, there is often little discussion about the equally-important design rainfall. The fact that adequate precipitation data rarely are available for project design is readily admitted - and then ignored. The lack of understanding of natural phenomena is either accepted or it is unrecognized, and debate about the best
hydrologic method can be intense, yet meaningless in the broad sense of true problem solving.

Computer Applications

Computers are a tremendous aid in managing vast amounts of data and in performing time-consuming or repetitive calculations. Computer applications of hydrologic methods have been available from many sources and for many applications. All of the variations described above are computerized, and many local modifications such as the Pima County Method (1979) are coded by consultants, agency staff, or both. The basic problems in the field of hydrology - inadequate data and unknown relationships - are not addressed by computerization. In a landmark publication of the American Geophysical Union (1986), several authors criticize the introduction of computer methods as detracting from basic research and the improvement of hydrologic understanding. Computerization speeds repetitive calculations and iterative processes, and is a valid aid to design and research. However, mere application of computer processing to otherwise unreasonable hydrologic methods tends to falsely lend credibility to the output. Concerns of agencies and engineers too often emphasize arbitrary input parameters instead of assumptions about physical process and the validity of the method. An exception to this concern is the development of the manual procedures of TR 55 by the SCS. The method is based upon computer simulation of historic
data which is used to aid in generalizations about the data which would have been difficult without the aid of a computer. The more common case today is to use a computer to manipulate general data to produce more specific design parameters. This is simply poor science and is similar to concluding that if a carp is a fish, all fish must be carp.

Use of computerized hydrologic methods as management tools is also a concern to some planners and engineers. At a recent conference of the Water Resources Planning and Management Division of the American Society of Civil Engineers, a young consulting engineer presented a paper about a hydrologic model his firm developed for a non-engineer client. The method was reasonably straightforward and would be acceptable for its limited intent in the hands of a knowledgeable user. Diversity of opinion expressed during the subsequent question and answer period indicated that some listeners were interested in the approach, perhaps for their own use, while others were adamantly opposed to the potential of untrained non-technical people having access and actually using a computerized hydrologic method. One attendee even questioned the ethics of the speaker and his employers.

This example is mentioned because it illustrates three uncomfortable trends in the discipline of applied hydrology. First, computers can be misapplied through intent or ignorance. It's not difficult to use the wrong method if a point needs to be "proved" or from lack of knowledge of limitations of some significance. Errors are difficult to catch without thorough
review of input assumptions, model development and "coding, and understanding the meaning of the output. An opinion of reasonableness or lack of it based on experience is sometimes the best that can be expected.

Second, computerization leads to a feeling of more precision in numerical results. Using the concept of significant digits and recognizing that Manning's "n" values, channel shape, and hydraulic radius are not well known under varying flow conditions, estimated discharge from a channel reach probably should not be expressed in five significant digits. It is common to see this assumed precision used as a regulatory consideration or hydraulic design criteria. In a practical sense, applied hydrology is a quasi-technical tool between planning and design, or between legislation and litigation. In consideration of the true state of hydrologic knowledge, computer-generated pseudo-precision should be avoided unless the precision can be honestly and fully explained.

Third, and last, computerization has detracted from research into the basic science of hydrology. Admittedly, numerical methods of optimization, statistics, and theory testing are greatly facilitated by electronic data processing. There is a recognizable difference in computer applications, i.e. data processing in scientific research versus presumed problem solving by automated hydrologic methods.
Regionalization

Hydrologic methods can be regionalized in recognition of special climatic characteristics of a locality. Rainfall and runoff data for a limited area of some hydrologic homogeneity is collected and analyzed. The net result is an adjustment to an otherwise acceptable but more general method. Most commonly used methods such as the rational method and the SCS method are empirical and based on aggregated data or data from another disparate region. This holds true of most of the widely accepted equations in hydrology, such as evapotranspiration, which need local calibration with available data to be most meaningful.

Regionalization requires not only local data, but also a structured and creative view of local hydrologic mechanisms. In the southern Arizona area, the work of Lane (1982), Renard (1984), Reich (1981), Osborn (1970, 1981, 1988), Malvick (1980), and others is notable. Their work is based mainly on or includes data collected at the Walnut Gulch experimental watershed near Tombstone, Arizona. The network of about one hundred rain gauges is supported by stream flow monitoring and sediment transport data. The results of these regionalization efforts are similar; that is, they all provide runoff rates within a confined range. This is to be expected from research using the same data and similar analytic methods. Despite the obvious value of this body of work, it has met only minimal acceptance by designers and regulatory agencies within the general region of southern Arizona. The previously discussed
default methods are utilized much more often than these methods in public works projects and private development.

Probability Density Functions

Climatic history of catchments are non-existent at worst and of short duration at best. In the former situation, the best one can do to solve an immediate problem is assume some degree of transferability of rainfall data from a similar nearby watershed or utilize some type of default values. In the latter situation, the shortcomings of the data are artificially overcome by assuming that the brief historical record is an accurate section of a distribution function. It is assumed that given a section of the distribution and assumptions about what the distribution looks like, that the data can be extrapolated with some level of confidence for recurrence intervals both within and beyond the length of record. That is, it is generally assumed that a twenty year record indicates the magnitude of a twenty-year event, and that when the data is fitted to an appropriate distribution function, the 100-year or 500-year or any other recurrence interval event can be estimated. The probability density function recommended for use by agencies of the federal government, and for participants in federal programs such as flood insurance programs, is the Log Pearson Type III distribution. Use of the method was encouraged by the Water Resources Council in 1967, and reiterated in 1976, 1977, and 1981. The LPIII distribution is based on the work of Professor Karl
Pearson of University College, London during the early 1900's as reported by Foster (1924). According to the WRC (1981),

"Flood events are a succession of natural events which, as far as can be determined, do not fit any one specific known statistical distribution. To make the problem of defining flood probabilities tractable it is necessary, however, to assign a distribution. Therefore, a study was sponsored to find which of many possible distributions and alternate fitting methods would best meet the purpose of this guide. ... The Work Group concluded from this and other studies that the Pearson Type III distribution with log-transformation of the data (Log-Pearson Type III distribution) should be the base method for analysis of annual series data using a generalized skew coefficient..."

The skew coefficient is an indicator of asymmetry of the data and is estimated using a three-phased analysis procedure. The skew coefficient is adjusted to reflect zero flood years, that is, years with no recorded flow above some base gauging level. This condition is common on ephemeral streams in arid regions of the southwest. This situation precludes use of the recommended LPIII distribution because the logarithm of zero is minus infinity. Therefore, a "conditional probability adjustment is recommended for records with zero flood years..." and synthetic statistics are calculated and added to the base data according to recommended procedures.

The numerical results of statistical analysis is dependent upon the distribution used in the analysis. If a given data set is applied to several distributions, different design flows will be produced for a given recurrence interval. The magnitude of the difference is greatest at large recurrence
intervals where extrapolation of the data occurs. Within the data range itself, the variation between distribution functions is less than in the extremes, and one can conclude that the longer the period of record, the less variation there would be. Likewise, if additional data elements are introduced into the data set, the distributions will respond differently to these changes. These attributes are illustrated by Reich (1981) where several PDF's are plotted for the same limited data set and the projected 100-year discharge of the watershed in question varied from about 5000 cfs to over 25,000 cfs.

The WRC recommends that a data base of at least twenty years of annual peak flood flows be used in the LPIII procedure. Assumptions about the data include (1) the local climate does not change during the recording period, (2) the data represent random and independent events, (3) the watershed characteristics are constant, (4) the data represent one type of event and is homogeneous (snowmelt data is separate from rainfall data, for example), and (5) that the data are reliable and accurate.

**Paleohydrology**

Extreme events are sometimes called "outliers" to the statistician and they are considered persona non grata for their disgusting habit of not fitting an assumed distribution pattern. Justification and methods for discounting extreme values from a limited sample abound in the literature. Although
statisticians may reject extreme values, scientists seek them out using the tools of paleohydrology, literally, the study of old water.

The goal of paleohydrology is the determination of rare events relating magnitude with frequency. As described by Gregory (1983) current endeavors include paleoclimatology, paleogeomorphology, paleohydraulics, and paleopedology. Each of these related fields are important to paleohydyrology because determination of historic peak flows by themselves, is not of much value. The peaks need to be related to basin characteristics and climatology of the time of occurrence to help understand the relationship between rainfall and runoff over time. He describes the difficulties confronting interpretation of paleohydrologic data as it might effect contemporary river channels and explains that although uncertainty exists, the field is promising.

Baker (1983) and Patton (1987) further describe some tools available to the paleohydrologist. They include the structure and composition of ancient soils, tree rings, pollens and spores, archaeology, written or oral records, sedimentology, geomorphology, distribution of flora and fauna, topography, and isotopic dating of ancient waters. Used in some combination these tools can help define previous hydrologic conditions and large ancient floods. Related work by Thomes (1983), Maizels (1983), Graf (1983), O’Connor (1985), Ely (1985), Costa (1978, 1987) and others are convincing enough to stimulate discussion about the application of such methods to contemporary desert flood studies. Certainly if one uses such methods to "think back", there should be
a better understanding of the hydrology of today, the relationship between rainfall and runoff, and the magnitude of rare events.

The one-hundred year flood is defined as the flood that has a one-percent chance of happening in any given year. The terminology is unfortunate because it fosters an artificial sense of security that the 100-year event can happen only once every one hundred years. In reality, what is thought to be the 100-year flood could occur much more often, and usually does. Flood data is limited, and hydrologic models so imprecise that the 100-year event is underestimated until its magnitude is adjusted upwards by new data. The US Congress public hearings (1979) discussed the Phoenix, Arizona experience of three 100-year floods in less than 10 years. The estimated magnitude was increased due to statistical acceptance of these "outliers." A detailed paleoflood record could help establish the magnitude and frequency of large floods much better than simply assuming that limited data needs to fit a given PDF. Palaeoflood analysis is the opposite approach for handling outliers - rather than discounting outliers, they are sought out and cherished.

The inverse of frequency or recurrence interval is probability. Using this concept, the "100-year flood" translates to the "0.01 probability flood". This terminology is perhaps more confusing, though less misunderstandable and less imprecise. Nonetheless, the federal government mandates use of the 100-year flood for flood insurance program participants. They do not require structural improvements, nor do they prohibit use of less frequent (bigger) events. The
magnitude of the 100-year flood could be refined through palaeohydrology so that a greater confidence in the projection is developed. Then, flood insurance rates could be more accurately established within and between geographic regions, and design guidelines for structural improvements and non-structural management programs could be refined.

**Applied Hydraulics**

Hydraulics is the practical application of knowledge of liquids in motion. It is distinct from hydrology and is further refined here to mean the design of civil engineering hydraulic structures. In an idealized world, a hydrologist estimates the amount of water a stream will carry in the design storm, and the hydraulic engineer designs the runoff conveyance system based on this estimate. As often, or more often than not, the hydrologist and the hydraulic engineer are the same person.

Understanding the difference between hydrology and hydraulic engineering leads to perception of the differences in precision in each profession. The hydrologist deals with the dynamic, stochastic processes of nature and in so doing can only provide crude estimates for design criteria. The hydraulic engineer must recognize that an estimate should be qualified within a range or confidence interval of some sort, and that reasonable design alternatives should be considered and evaluated. In essence, a sensitivity
analysis of design alternatives can help overcome the imprecision of the hydrologic estimate. At best, the design engineer can say something like "the bridge is designed to pass a discharge of 50,000 cfs without failure, and this discharge is approximately a 50-year event." To say that the bridge will pass a 50-year event without failure is not accurate because of the imprecision in the hydrologic prediction, i.e., no one "knows" what the 50-year event really is. And since the bridge has a fifty-year structural design life, the statement should be that the bridge will "probably" fail or be damaged sometime during its fifty-year life.

Hydraulic Analysis

Hydraulic analysis involves the flow of water, water surface profile, sediment transport, scour, and other related factors in a given flow regime. The analyses are useful for design of structures such as berms, levees, detention basins, bridges, dams, culverts, and storm drains. Non-structural storm water management solutions such as flood plain mapping and structural setback limits are based upon hydraulic analyses. Research and publication in hydraulics is prolific, and the typical undergraduate civil engineering core curriculum contains more on the subject than on hydrology. Use of physical models and flumes are practicable for many areas of hydraulics, whereas hydrologic modelling is extremely difficult. Both fields, however, generally do not emphasize regional differences that affect project design. The purpose, of
this section is to provide an introduction to that differentiation. It is not intended as a comprehensive discussion on hydraulics, but rather as a means towards understanding the relationship between the two fields.

Water Surface Elevations

Hydraulic analysis relies on fundamental understanding of fluid flow as applied in available analytic tools. As discussed earlier, the rational method is commonly used to estimate design flow through a conveyance system. Once a design flow is determined, a conveyance system can be designed. The design flow is used to correctly size the closed conduit to carry the flow. In the case of open channels, water surface elevations must be determined to help locate and design the channel system and its ancillary structures. The most commonly used method of analysis is the standard step method. The U.S. Army Corps of Engineers has adopted the method in its HEC-2 water surface profile program. The Corps of Engineers (1982) described and documented HEC-2 in a program users' manual, and only a brief introduction is provided here. The HEC-2 program is based on the principal of conservation of energy, which states that the total energy upstream is the downstream energy plus any energy losses between stations. The result is a one-dimensional energy equation for gradually varied flow. The loss of energy head is based on channel characteristics such as expansion and contraction, length of various reaches, and friction slope. Input variables include actual or design cross
sections, design flow rates, reach lengths, channel roughness factors, and expansion or contraction indices, and an assumed starting water surface elevation. Using the above equations, the program iterates until the equations are satisfied. Output is water surface elevation at input cross sections and a smooth transition through the channel reaches is assumed.

The HEC-2 program is one-dimensional and does not readily account for abstractions through the channel bed. In the Southwest, abstractions can be significant losses, and bed profiles may be appreciably altered during flood flows. The program has been widely applied to southwestern perennial and ephemeral streams by hydrologists and engineers with little consideration for its limitations. Without doubt, the program is applicable to fully-lined open channels where bed form is fixed and the continuity equation is valid. However, Lane (1982) states that its use in ephemeral streams where channel beds lower during flow and where transmission losses are substantial results in a conservative estimate of water surface elevation and backwater effect. The program is even less applicable in sheet flow areas, over-bank areas, and within alluvial fans. At any rate, HEC-2 is the most commonly used and most widely available computer software for estimating water surface elevations. Its use is a major factor in floodplain delineation, in the Southwest and elsewhere.

A simple yet effective way of determining water surface elevations is the use of Manning's equation or a comparable equation. For a given reach
of watercourse, and known conditions of discharge, slope, and cross section, the elevation in that reach can be readily determined. For many simple design projects, this is all the sophistication that is needed, and the method is appropriate.

Working backwards from a known water surface elevation that occurred during a flood, a hydrologist can estimate the peak discharge. Here, the water surface elevation is known with some degree of accuracy, but all the other factors affecting flow need to be estimated. If the channel has rigid boundaries such as bank protection or concrete lining, the cross sectional flow area is easy to determine accurately. If, however, the channel is natural, then the analyst has the complex task of trying to figure out the cross section, profile, flow velocity, and channel roughness during the peak flow. Any inaccuracies that occur could result in a poor estimate of the flood’s magnitude. In rigid boundary conditions, water surface elevations for a given discharge can be computed accurately, and equal accuracy is achieved in working backwards from flood elevation to determine discharge. This relationship between elevation or stage and discharge is commonly determined along natural or manmade streams for data collection purposes. However, the strength of the relationship decreases as boundary conditions become less rigid.
Control Structures

Structures are commonly used to control the horizontal and vertical position of a watercourse. In the Southwest, river banks are hardened to help prevent erosion into adjoining property. Grade control structures are also commonly used to "stabilize" the bed of the watercourse. The design of these structures for ephemeral channels is dependent upon depth of flow, velocity, sediment transport characteristics, and composition of the natural bed and banks. Often, both types of structures need to be built because changing the banks will eventually alter the bed, unless it too is protected from erosion.

Various materials have been used successfully to minimize bank migration. Included are soil cement, gunite, concrete, interlocking blocks, rock rip-rap, metal jacks, and even old car bodies. Grade control structures are almost always reinforced concrete, placed either vertically, or sometimes sloped downstream. These structures must resisted greater forces than bank protection, and are designed accordingly. Energy dissipators are sometimes needed to slow and disperse the flow, thereby decreasing total energy in the flow and minimizing erosion potential.

Detention and Retention Basins

Drainage improvements often incorporate some type of detention or retention system, intentionally or not. Detention basins are designed to slow the discharge to some pre-determined rate; retention basins decrease the
volume of flow. A combined basin can be designed to control basin outlet rates and volumes to certain conditions such as those that are assumed to occur prior to urbanization, for example. Using detention, retention, and creative outlet design, it is quite possible to provide almost identical pre- and post-development hydrographs for recurrent flow even though the flow characteristics into the basin are very different after development within the basin. Larger floods are affected only minimally by detention or retention basins.

Although it is possible to maintain and control the flow of water through the basin, it is next to impossible to maintain a balanced sediment flow. If the flow into the basin is sediment laden, the basin will fill up over time, and some sort of maintenance is needed to continually provide the design storage volume. Flow out of the basin will be sediment starved, and erosion will occur immediately downstream of the outlet. Outlet erosion needs to be considered during design of the basin, as does the downstream erosive effect of the water. These problems are more pronounced in the arid Southwest where steeper slopes produce higher flow velocities that can transport more sediment. If this sediment is deposited in a detention/retention basin, downstream erosion of the arroyo is inevitable. Chang (1982) provides a discussion about erosion in alluvial channels and presents analytic methods to evaluate long term changes and stable bed profiles based on sediment
transport properties. This methodology has been widely accepted in regions of arid Southwest where the problems of erosion have been recognized.

An additional concern in the design of flood control facilities incorporating storm water retention is that highly recurrent flows do not leave the basin. A basin designed to retain flow will cut off small flows entirely, and diminish the rate and volume of intermediate flows. This could be a concern to riparian properties downstream, particularly if these small flows provide irrigation for natural vegetation and wildlife. Bearing this in mind during project design could lead to exclusion of retention as a design alternative, or providing some type of flow bypass for small events.

Sediment Transport

Sediment transport is probably the most often ignored drainage design consideration, and it is also one of the most important. The fact that sedimentation is often ignored is evident in scour around bridge piers and abutments, at the outlet of detention basins, at roadway drainage crossings, and at drainageway inlets to major channels of the arid Southwest. The catastrophic effect of scour was evident in Tucson’s flood of October 1983 (Saarinen, 1984; Reich, 1984) and numerous other desert floods. In the Southwest, erosion is a major cause of damage during rare flow events, probably more so than inundation.
In the above discussion regarding detention basins, it is evident that sedimentation is critical to the design of the project, as well as its operating costs, and the definition of project limits, both upstream and downstream. Sediment transport is a critical component of drainage system design, and more than any other factor forces a systems approach to comprehensive design. Drainageway design that considers only the flow of water is much easier to design, much less adequate, and much less rational than an approach that includes sedimentation as well.
Flood control and drainage projects result in many social benefits, most notably the protection of public health and safety from the forces of rapidly flowing water. These benefits are not achieved without significant public and private investment for planning, engineering, land acquisition, construction, and maintenance. Physical alteration or protection of natural drainage ways is common in civil engineering projects, and yet the benefits and costs of the drainage portions of these projects are usually not examined. This chapter presents an informal survey of public officials that demonstrates the general lack of formal or even informal economic analysis in state and local drainage projects.

The practicing civil engineer has been conditioned by regulatory policy that mandates design for the "100-year" flood and the assumption that structural solutions are preferred over non-structural approaches to flood plain management. Project design is frequently limited in definition, goals, scope, and range of alternatives. For example, a typical subdivision project might contain a natural channel that needs to be considered during the development process. The conditioned goal of the design engineer/hydrologist is to meet the standards set forth, and standard solutions regularly accepted, by the local regulatory agency, e.g., to comply with the national flood insurance program.
The range of alternative designs is severely restrained by these parameters and is limited to different channel cross sections, materials, and hydraulic structures that meet the perceived goals. The analysis becomes nothing more than cost estimates of various construction materials and techniques, without quantifying the benefits or consideration of a range of design goals. The apparent least-cost alternative is often selected. However, the economic efficiency of project alternatives, if any, has not been addressed.

One of several useful tools in evaluating the economic performance of any set of alternatives is benefit-cost analysis, abbreviated herein as BCA. BCA is nothing more than a systematic accounting procedure that weighs project benefits against project costs. When the benefits exceed costs, i.e. when the B/C ratio is greater than unity, a project may be warranted. The higher the ratio, the better a specific design appears in relation to the other alternatives. The words "may" and "appears" are underscored to emphasize that BCA is only a tool in the decision-making process. Its proper application requires interpretation of results, not blind acceptance of the alternative with the highest numerical rating. BCA can be confusing and complex, and be difficult to apply fairly to a multi-purpose water resources project. For example, construction costs may also provide local economic benefits, and the inexperienced or contriving analyst may sway the results by putting this big-dollar item on the wrong side of the ledger. A simple example of BCA is provided later in this chapter and it shows how BCA can be used to raise
important issues that otherwise remain ignored. The example is comparable
to that used by Riordan (1978) but is more relevant to the desert Southwest.

BCA has several important purposes. It can be used effectively to (1) chose
between alternatives, (2) to establish project scale, (3) to chose between projects,
and (4) to help evaluate the economic impact of flood control policies.

Benefit-cost analysis is a standard tool of economic evaluation that has a
reputation for being misapplied and, even more recently, it has been neglected
by designers of state and local drainage projects. This chapter discusses the
need for and uses of BCA or similar techniques, and why it should be applied
in drainage projects.

An Informal Survey

The use of alternatives evaluation by government agencies in Arizona
and Nevada was the subject of an informal telephone survey of county and
city flood plain managers, administrators, and engineers. Responses
demonstrated that BCA is rarely used by the agencies on non-federal projects,
that the 100-year flood is the universal design standard, that the damage
potential of less frequent, or more frequent, floods is not considered, and that
projects are not prioritized based on performance indicators.

These results are not too surprising in the light of state statutes that
empower local agencies to resolve drainage problems. Of the desert states,
only Nevada requires some type of accounting and pay back from flood control projects. Nevada Revised Statutes 543.580.2 says, in part...

"The chief engineer and general manager [of a flood control district] shall make and file reports from time to time, which must show... [a]n estimate of the cost of each project or work of improvement including...[a] comparison of the total cost of the proposed works with an estimate of condemnation and relocation or replacement of property within the boundaries of the flood plain."

The section requires a district to verify that construction cost is less expensive than condemnation of flood-prone property before building a flood control structure.

Most survey respondents also indicated authority for both public and private drainage improvements. Several respondents mentioned that they should be conducting the types of analysis suggested in the questions, but do not. The survey and aggregate responses are shown in Figure 5-1.

**Using BCA to Choose Between Alternatives**

The structured evaluation of alternatives relies on the quantification of costs, benefits, and intangibles for each alternate design or approach. Project financing techniques are also important, particularly if the project has several participants.
In the design of drainage ways or flood control works, does your agency routinely use BCA or similar methodology to...

| a) evaluate alternatives? | {YES 1* NO 6} |
| b) establish project priorities? | {YES 1* NO 6} |
| c) size projects? | {YES 1* NO 6} |

Do you routinely design flood control or drainage projects to a fixed standard such as the "100-year" flood, standard project flood, or the like?

| a) "100-year" flood | {7} |
| b) SPF | {0} |
| c) Other | {0} |

Do you routinely evaluate the effect that a rare hydrologic event may cause? For example, if you design for the "100-year" flood, do you assess the impact of a "150-year" or larger flood?

{ NO 6 YES 1** }

My responses address...

| a) public works of my agency | {2} |
| b) private works approved by my agency | {1} |
| c) both a and b, above | {4} |

Note: Two respondents commented that BCA was used on federally sponsored projects in their community.

* For dams and related structures

** Sometimes evaluates the 500-year flood

FIGURE 5-1

INFORMAL BCA SURVEY
Project Costs

In the Southwest, consideration of costs for drainage improvements of local and regional scale is typically limited to the cost of construction. A better evaluation would also include the value of right-of-way, the present worth of project maintenance over the design life of the project, the cost of mitigating environmental or cultural resource degradation, and an assessment of otherwise intangible disbenefits. The latter category includes such considerations as visual impacts, and, although they may seem difficult to quantify, Peterson (1986) presents viable analytical methods. Other cost categories may exist and should be determined and assessed for each specific project. For example, Laursen (1985) suggests designing some bridge approaches to be damaged or lost during floods smaller than the 100-year flood. The costs of repair and inconvenience would need to be considered in the evaluation process.

Project Benefits

The selected alternative should have attributes beyond being the lowest cost to construct. However, for any project where a BCA is not conducted, the basic question "Am I getting a good deal?" is unanswered. Certainly, the proponents of "standard design" may suggest that "You get your project approved expeditiously by the regulatory agency", or "You get
those irate citizens off your back". These are benefits of a sort, but the answers don’t identify whether or not there is a "better" solution.

Benefits of drainage improvements include the usual flood control benefit of (depending on your perspective) increased land value, damage reduction, or development potential. But in the Southwest, where storm runoff should be considered an asset rather than a liability, every project is potentially multi-purpose. This is the significant difference of drainage projects in the arid regions where water is a valued, scarce resource and not just a nuisance to be transported downstream.

The assignment of value to harvested water is not a conventional approach in BCA. But, according to French (1988) "the great majority of [BCA] have not been performed in areas classified as arid." Cluff (1985) has long been a proponent of harvesting storm water runoff in southern Arizona, but due to federal, state, and local commitments to water importation via the Central Arizona Project, his views have met continuing resistance from those who take the position that the supply and allocation of CAP water is assured and sufficient to meet long term needs. Benefits, therefore, should, as a minimum, include the flood damage reduction and the value of harvested water. This, in itself, is a radical departure from the present practice of designers and regulatory agencies in the Southwest. An example of past consensus of flood plain management in the arid region is presented by Weisz (1973), in which land use planning and flood control in the Tucson area are
addressed. Weisz pays particular attention to BCA, but fails to consider the value of increased runoff associated with urbanization. Clearly, the motives and benefits of flood control projects in arid regions are different from humid regions. The analysis can be similar, but the determination of what is a cost and what is a benefit may be different.

Project Financing

Project financing can be public or private, the difference being that public financing spreads the cost over direct and indirect beneficiaries whereas private financing is provided by only the direct beneficiaries. There are few if any drainage projects that can be categorized as truly "private", except in terms of initial capital investment. Maintenance, repair, and other secondary costs are routinely passed on to government agencies. Project financing needs to be addressed carefully in the BCA because costs can be inadvertently excluded, double counted, or mis-allocated during the analysis. The effects of inflation and the time-value of money also should be carefully considered for the results of the analysis to be meaningful.

A Suggested BCA Methodology

It should not be inferred that all federal flood control programs and procedures are inappropriate to the Southwest. The federal government's Methodology for Evaluating Alternative Economic and Environmental Plans for
*Water and Related Land Resources Implementation Studies* (US WRC, 1983) is an exemplary tool that can be readily adapted to non-federal projects. The approach defines general planning considerations and economic procedures and establishes accounts for national economic development, environmental quality, and regional economic development. Other effects may be addressed in separate accounts, as warranted. The WRC procedure is utilized by federal water resources agencies including US Army Corps of Engineers (Petersen, 1984), Bureau of Reclamation, Soil Conservation Service, and Tennessee Valley Authority. A key element of the WRC procedure is the allocation of project costs to the beneficiaries by project purpose. The "without project" condition, also referred to as the "do nothing" alternative, is the basis for economic comparison. This multi-dimensional analysis departs from the previously discussed restrained one-dimensional or univariate analysis practiced by engineers and local agencies for non-federal projects. Federal guidelines do not necessarily limit flood control or water resources projects to a perceived set of design standards. Instead, USWRC (1983) states that alternative plans may either "be in compliance with existing statutes, administrative regulations, and established common law; or ... propose necessary changes in such statutes, regulations, or common law." The procedure encourages consideration of non-structural and combination structural/non-structural alternatives in the generation of alternative plans. This progressive position is in contrast to the
state or local agency perception of federal policy, particularly as interpreted in the adopted ordinances for these agencies.

Additional Considerations

Logical decision makers base their recommendations on how well an alternative meets their needs, and not on the overall economic performance of the alternative. In public projects, costs can be spread among a wide population base. Conversely, in privately funded projects, only project benefits can be spread among a wide population base. This leads to some difficulty in deciding what is the "best" alternative to construct. However, in the usual contemporary situation, this issue does not arise. Benefits are neither evaluated nor assigned; therefore, no one knows who gains and who loses.

According to recent article by Cochran (1988), one must consider the nature of riparian habitat and its irreplaceability in the design of drainage ways and flood control systems. This is a direct analog to wetland protection afforded by federal law to perennial stream systems. The environmental issues raised by such discussions should not be ignored because they represent a developing trend in flood control projects to provide flood protection with a minimum of environmental change. In other words, the standard engineering solution of "building something" is no longer readily acceptable to the general public or to our professional colleagues in other related disciplines.
The result of this emerging philosophy is that aesthetics, habitat preservation, and environmental sensitivity are important factors in project design. These factors, more than many others, tend to spread the benefits of drainage projects over a broader base than merely the project "owner", particularly in development-related projects.

Maintenance costs can be excessive in a poorly designed project, and the BCA analyst needs to be aware of the magnitude of these costs. A design that ignores sediment transport characteristics of an ephemeral stream could easily result in excess scour or sedimentation that lead to functional or structural failure. In private projects, these considerations seem unimportant to the owner; in public projects, these concerns often go unrecognized. The ultimate maintenance experience is failure-induced reconstruction, the cost of which is invariably a public expense. Another public expense is the cost of flood damage repairs associated with an extreme event that exceeds the project's hydraulic design criteria. The magnitude of damage for an event larger than the design flood is an unasked, and therefore, unanswered question. BCA can help address this problem by evaluating the economic efficiency of projects designed for events larger and smaller than the design flood, and then selecting something that seems optimal and rational. This concept also is in contrast to routine design procedures.

Land, like water, is a finite resource and can be an expensive component of drainage projects in urban areas. The cost of right-of-way for
a project is readily estimated by an experienced appraiser, but the true land
cost of the project may exceed the obvious acquisition. Indirect but
quantifiable costs include use restrictions to adjoining land, erosion hazard
setbacks, and inherent commitments to future drainage improvements by
upstream or downstream property owners.

The indirect economic impact of flood control actions was the subject
of a recent US Supreme Court decision (1986) which declared that Los Angeles
County's application of a local flood plain management ordinance prohibiting
a church's reconstruction of flood damaged structures resulted in a
compensable temporary "taking" of the Church's private property in violation
of the Fifth and Fourteenth Amendments. The ordinance, the Court ruled,
went "too far" and denied any economic use of the property. Constructed
within a flood plain prior to adoption of the local ordinance, the property was
destroyed in a flood that was exacerbated by a prior upstream forest fire and
by cloud seeding.

This case, along with other recent related Court findings resulted in
a Presidential Executive Order 12630 requiring federal agencies to consider the
legal implications of day-to-day project-related decisions. In the absence of
basin-wide flood plain management strategies, inverse condemnation issues
can be complex and potentially litigious, but are often totally ignored by local
agencies, developers, and design consultants.
A Qualitative Example

A simple hypothetical example illustrating the potential significance of BCA is presented to help clarify the previous discussion. The example involves watershed changes associated with an urban development, and although hypothetical, it is not far-fetched. In the rapidly growing arid Southwest, this example is appropriate because most watershed changes — construction of a new subdivision, or a new roadway, or an open channel aqueduct — result from urbanization. The example involves a square 160 acre parcel of native desert traversed by an ephemeral channel. The channel has obvious physical evidence of lateral migration; therefore the local floodplain ordinance requires a 50-foot building setback from the 100-year flood plain. The ordinance allows construction of structural bank protection up to, but not within, the floodway to replace the erosion setback requirement. Any construction within the floodway violates the local ordinance and jeopardizes the community's participation in the federal flood insurance program.

General conditions of the site are shown in Figure 5-2, in which the flood plain and floodway are determined by FEMA-approved methods, and the erosion setback is a local initiative derived from local considerations. The flood plain contains natural vegetation which helps foster riparian wildlife and animal migration. Riparian habitat of this type has been assigned a value through university research and political action. The value is often less than
Figure 5-2

Project Site - Example
the fair market value of the property. Further, the region is largely dependent on ground water for its municipal water. Water flowing through the site is eventually recharged by natural forces. Development of the site is expected to increase runoff into the channel by 500 cfs, and there is adequate channel capacity downstream to accommodate this increased flow. The increased imperviousness of the development will also increase the volume of runoff from the site.

The developer purchased the property with existing zoning that authorizes up to four homes per acre. The intent is to provide a quality development that meets market needs and maximizes profit for the developer. Each new home sold represents a profit of $5000, and the development is expected to sell quickly. A reputable planning and engineering firm has been retained to conduct a hydrology study and prepare other plans and reports necessary to produce a subdivision plat for the parcel.

Two Scenarios

For purposes of this example, two scenarios are presented. The first is the usual case in which the consultant's primary goal is to meet the client's needs while complying with local ordinances and standards of professional practice. After some head scratching, it is determined that the lowest cost acceptable solution is construction of reinforced concrete bank protection at the regulatory floodway. The naturally wandering channel is replaced by a
straight channel. Bank protection would be placed away from the floodway, but the internal circulation of the subdivision mandates two bridges across the wash. This alternative reclaims about 12 acres from the flood plain and erosion hazard setback, and provides 48 extra home sites, each with $5000 profit. The cost of the bridges is minimized by shortening them as much as possible.

The hydraulic performance of other construction techniques was evaluated in the hydrology and hydraulics studies. Bank protection of various types, also following along the floodway, were evaluated. Each of the other construction techniques exceeded the $50 per lineal foot ($132,500) cost of the selected concrete alternative. Construction plans are prepared and approved by the local regulatory agencies, a subdivision plat is recorded, homes are built and sold. Roads, bridges, and drainage improvements are built under agency inspection, and accepted as public facilities upon completion.

In the second scenario, nothing changes except the analytic technique. The local agencies have become enlightened and now require a simplified BCA along with the hydrology/hydraulics studies and engineering plans. The new procedure requires consideration of maintenance costs, the value of augmented recharge, and the value of lost riparian habitat. The procedure requires separate calculations of an overall BCA, an evaluation of the BCA solely to the developer, and solely to the public. The analysis is completed for the previously acceptable alternative, and is summarized in Table 5-1. Here, the
TABLE 5-1
EXAMPLE BENEFIT COST ANALYSIS

<table>
<thead>
<tr>
<th>ACCOUNT</th>
<th>PRIVATE</th>
<th>PUBLIC</th>
<th>COMBINED BENEFIT</th>
</tr>
</thead>
<tbody>
<tr>
<td>48 Additional homes @ $5000</td>
<td>$240,000</td>
<td></td>
<td>$240,000</td>
</tr>
<tr>
<td>Aquifer Recharge*</td>
<td></td>
<td>$25,000</td>
<td>$25,000</td>
</tr>
<tr>
<td>TOTAL</td>
<td>$240,000</td>
<td>$25,000</td>
<td>$265,000</td>
</tr>
</tbody>
</table>

COSTS

<table>
<thead>
<tr>
<th>COST</th>
<th>PRIVATE</th>
<th>PUBLIC</th>
<th>COMBINED BENEFIT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction</td>
<td>$132,500</td>
<td></td>
<td>$132,500</td>
</tr>
<tr>
<td>Maintenance*</td>
<td></td>
<td>$75,000</td>
<td>$75,000</td>
</tr>
<tr>
<td>Habitat Reduction ($10,000/acre)</td>
<td>$120,000</td>
<td></td>
<td>$120,000</td>
</tr>
<tr>
<td>TOTAL</td>
<td>$132,500</td>
<td>$195,000</td>
<td>$327,500</td>
</tr>
</tbody>
</table>

BCA 1.81 0.13 0.81

* indicates present worth
overall BCA is 0.81 with a private BCA of 1.83 and a public BCA of 0.13. In this analysis, it becomes clear that, although the alternative meets local and federal floodplain guidelines and the developer's profit goal, the drainage way design is not desirable from the public perspective. Other alternatives need to be generated and evaluated with the intent of finding a better solution than one that merely "works."

In the example, a critical factor is the assumed value of the riparian habitat which can be matter of intense argument - much more than for the other items in the BCA. However, just the arguments, in this analytical context should be useful and could lead to new alternatives that could optimize the ledger in all three BCA accounts. Perhaps the developer would be willing to protect the riparian habitat in exchange for a somewhat higher overall density that yielded the same number of lots. Perhaps the local government would acquire the habitat for public use through right-of-way negotiation or eminent domain. A better solution is agreed upon, and the project is constructed. The analytic approach used in the second scenario has forced consideration and recognition of issues and policies ignored in the first scenario.

**Equitability Index**

The basic questions of who benefits from and who pays for flood
control projects has been raised previously by Laursen (1985, 1985a) and others. The issue is equitability, or a general balance between what a project participant pays for and what they receive. Agencies of the federal government use a cost allocation method that attempts, albeit indirectly, to assign project costs in proportion to benefits accrued. In comparison, the informal survey indicated that benefit-cost analysis used by local agencies are limited and probably do not normally assess costs to the beneficiaries in a prescribed manner.

The success of a project is dependent on many factors including (1) the perceived need for flood damage reduction, (2) the quality of design, (3) the range of alternatives considered, (4) the breadth of project purposes in fulfilling public and private need, (5) the availability of project funding, and (6) the political acceptability of the project. Each of these factors is important, and this brief listing can be considered to be arranged in increasing order of importance with political acceptability being the most crucial to the project. The last three factors relate directly to the often-ignored concept of equitability. Obviously, funds must come from one or more accounts or sources, hopefully in some sort of parity with project purpose, and attributed so that politically-oriented decision makers will make a "good" decision.

One way to help assess parity between financial participation and benefits received is what is introduced herein as an "equitability index". This index is simply an indication of the correlation between the amount of
funding, which may be either money or in-kind contribution, and the benefits, which may be either loss-reduction or value-added, associated with that funding. The equitability index is defined by the following relationship, which is not unlike the statistical concept of coefficient of variation.

\[ EI = \frac{BC}{\sqrt{\frac{\sum (BC_i - BC)^2}{n}}} \]  

Equation 5.1

where...

- \( EI \) = equitability index
- \( BC \) = benefit-cost ratio for the project
- \( BC_i \) = benefit-cost ratio for participant \( i \)
- \( n \) = number of project participants

The EI represents the inverse of dispersion of the project participant B/C ratios (\( BC_i \)) around the overall project B/C ratio (\( BC \)). Because of the inverse relationship, a small dispersion produces a large EI which represents a high level of equitability in that project alternative. Using this relationship, an EI of zero means the project is perfectly inequitable, that is, the beneficiaries do not bear any project costs, and the funding sources receive no benefits. (A public project with an EI of zero would be the quintessential "pork barrel"). Conversely, an infinite EI means that each participant has exactly the same B/C ratio, which is also identical to the overall project B/C ratio. The likelihood of achieving a perfect balance or perfect imbalance is
slight in any project with more than a few participants; therefore, the meaning of intermediate values between zero and infinity is of practical interest.

Consider a hypothetical situation in which all the BCi's are either greater than or less than the project BC by exactly the same amount, say 10%. Each BCi is either .9 BC or 1.1BC. Then, Equation 5-1 reduces to the reciprocal of the difference, as shown in Equation 5-2.

\[ EI = \frac{1}{1-d} \]  \hspace{1cm} Equation 5-2

where \( d \) = a dispersion constant

and \( 1-d \) = variation of BCi from BC

If each BCi is exactly 10% different from the overall BC, then the EI is exactly 10. An acceptable EI is one that represents a reasonable range of dispersion for a given situation. This is a value judgement by the project analyst. If a dispersion greater than 50% is unacceptable, then an EI more than 2 would be sought. Likewise, an EI of 5 indicates that the BCi's are generally within 20%.

To be meaningful, both the BC ratio and the EI for each project alternative or each project competing for funding must be considered. The benefit-cost ratio indicates the return on investment while the EI indicates the distribution of benefits and costs. It is quite possible to have perfect
equitability in an uneconomical project, just as it is possible to have a warranted, yet inequitable project. Table 5-2 illustrates the continuum of project acceptability considering both indices. In the table, several alternatives are evaluated, each with a BC of about 2.0, but with different EI's. Assuming that each alternative is technically feasible, alternatives D, E, and F are clearly better than A, B, and C. Depending on actual circumstances, the decision to choose between E and F could be based on which factor is more important, the economic performance of the project (BC) or the economic return to each participant (BCi). The point is to use both analytic tools to help decide what is most desirable, on the basis of benefits, costs and their distribution. Projects can be reconsidered in the light of this continuum to help assure that a reasonably optimal solution is selected for implementation.

To further clarify the EI concept, consider an example project with an infinite EI, such as structural bank protection designed to prevent lateral migration of an ephemeral stream where the only beneficiary is the adjoining property owner, who is also funding and maintaining the project. If this same project is instead financed totally with public funds, the EI would be zero. More realistically, a project of this type might be paid for with a combination of public and private funds (such as with an improvement district scheme) and maintained at public expense if the watercourse will be publicly owned. In this case the EI would probably be greater than one, depending on the level of public funding and the average annual expense of maintaining the project.
### TABLE 5-2

EQUITABILITY INDEX EXAMPLE

<table>
<thead>
<tr>
<th>ALTERNATIVE</th>
<th>BC, FOR PARTICIPANT</th>
<th>PROJECT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>A. PERFECT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>INEQUITABILITY</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B.</td>
<td>0.5</td>
<td>0.9</td>
</tr>
<tr>
<td>C.</td>
<td>1.25</td>
<td>2.0</td>
</tr>
<tr>
<td>D.</td>
<td>1.5</td>
<td>1.75</td>
</tr>
<tr>
<td>E.</td>
<td>1.75</td>
<td>1.85</td>
</tr>
<tr>
<td>F. PERFECT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>EQUITABILITY</td>
<td>1.75</td>
<td>1.75</td>
</tr>
</tbody>
</table>
In a single-purpose project such as this, the EI would not be too difficult to estimate, and the assignment of some level of public participation may be politically expedient. However, in a multi-purpose project not only is the benefit-cost ratio elusive, but so is the EI. Modifying this simple example to include wildlife habitat preservation, public recreation, or ground water recharge makes the analysis much more difficult. The analyst needs to reach some agreement on the value of habitat, recreation, and ground water and assign the costs to project participants in some reasonable manner. Then, a funding method must be established that either allows beneficiaries to pay their fair share, or allows the transfer of the financial burden to some participant who is willing to pay more than a fair share. This benevolent participant, more often than not, is a governmental agency with an actual or perceived responsibility for solving drainage problems and an obligation to protect public safety.

The development-related example cited in the previous benefit-cost discussion can be enhanced by application of the EI concept. In that example, the overall BC ratio is 0.81, with the private party's BC ratio estimated at 1.81 and the public BC ratio estimated at 0.13. In this case, the EI is 0.95 with a BC ratio of 0.81, and the project can be considered undesirable because it is both uneconomical and inequitable. The project approach clearly needs to be revised to address not only the marginal benefit, but also the distribution of benefits and costs to both participants. In so doing, the nature of the project
could improve markedly from the single-purpose structural design approach mentioned previously to a multi-purpose, mutually beneficial, multi-beneficiary project that fits the hydrologic and surficial systems of the desert. These simple analytic tools can provide drainage solutions that not only meet flood insurance requirements, but also "make sense" in their physical setting.

**Using BCA as a Decision Making Tool**

The purpose of BCA and the equitability index concept is to encourage project designers and administrators to think about the project and to evaluate alternatives in some rational manner. The sole criteria for local drainage projects should not merely be "protection from the 100-year flood", but rather application of affordable and implementable design and management principles that support local needs.

**Project Financing**

Impact fees, developer fees, exactions and other creative funding methods have been used in various parts of the United States to assign costs to entities that create a need for public services. This concept can be used to finance drainage projects and to allocate costs fairly between public and private parties. Combined with benefit-cost analysis and the equitability index, creative financing programs can be developed that diminish the need for
government to be either financially benevolent or incapable of protecting the public from flood damage.

Establishing Project Scale

The sizing or scale of a flood control project can be optimized by evaluating three or four alternatives each based on a different flood recurrence interval, then plotting the results, as indicated by AASHTO (1982). The optimal solution is at the knee of the curve, as indicated in Figure 5-3. This approach assumes a linear relationship between recurrence interval, project costs, and project risk, as expressed on an annual or amortized basis. It is more likely that a non-linear relationship exists to reflect proportionately higher costs for more flood protection. This condition is shown in Figure 5-4. Again, inflation and interest rates enter both sides of the equation and need to be adequately considered. Petersen (1984) and others plot B/C ratios or a proxy of the ratio to yield a similar curve identifying optimal project size. This concept is shown in Figure 5-5. Although all three concepts indicate that an optimal design exists, there usually is a range of acceptable designs centered around the optimum. Often, there may not be much difference in the benefit-cost ratio between an 50-year and a 150-year project, and some other factor such as the E1 could be used to select a project within this range. If nothing else, these procedures at least force evaluation of a range of design values.
Figure 5-3

Linear Benefit Cost
Figure 5-4
Non-linear Benefit Cost
Figure 5-5
Modified Corps of Engineers Method
Design Flood Frequency

In benefit-cost analysis, a reasonable range of acceptable solutions needs to be fairly evaluated in a consistent manner in order for the results to be meaningful. Many engineering design problems lend themselves to application of this approach. Included are roadway corridor and alignment studies, structural systems evaluation, and many manufacturing decisions. Applied hydrology is constrained by the perception that the federal flood insurance program requires planning and design for the "100-year" flood, or that structural solutions are preferable. These perceptions are part of many agency drainage design standards and flood plain management ordinances.

The Water Resources Council (1967) recommends a procedure for estimating the recurrence interval or frequency of floods based on local flow records. Other agencies recommend procedures for estimating runoff based on historic or assumed precipitation patterns. These guidelines are adopted by local agencies in watersheds where stream flow data is not available. These default assumptions become the basis of local hydrologic analysis required for local projects. The effect on project design is that creative definition of the problem is prescribed by local rules and design guidelines that are adopted without much question of their applicability. Even worse, the effect of floods exceeding the design event is often totally ignored by regulatory agencies and project designers.
The 100-year flood has become the standard project flood in the arid Southwest, as well as most of the United States. It is used in flood plain delineation and integrated in land use planning and development regulation. Although the 100-year flood has become the sole standard of limited analysis, BCA for drainage design alternatives must address a broader range of alternatives. The 100-year flood clearly can not be considered sacrosanct if analysis of alternatives is to be conducted.

Three flood categories should be examined: frequent (2-year, 5-year), rare (100-year), and maximum expected flood. Damage - but not a catastrophe - can be permitted in a maximum flood that is not likely to happen. Non-major damage can be tolerated in a rare flood. For instance, the 100-year flood has a 25% chance of happening in 25-years (the life of a mortgage), a 50% chance of happening in 50-years (the life of a house, more or less), and a should happen at least once in 100-years. Little damage, but some inconvenience, would be acceptable from frequent floods- but not from every thunder storm.

Osborn and Laursen (1973) discuss the natural limits of precipitation or flooding within catchments and state that "among all the uncertainties in the art and science of hydrology, there is one certainty - neither the rainfall intensity nor the runoff can possibly be infinite." They chose the terminology "maximum expected peak discharge" or MEPD as the practical limit of runoff rather than the infinite runoff assumed by commonly used distribution
functions such as Log-Pearson III. If their approach is used, or the more conventional probable maximum flood (PMF), the design flow could be expressed as a percentage of the likely worst case situation rather than as the 100-year flood. The abscissas (x-axis) of Figure 5-3 through Figure 5-5 could be expressed as some per cent of MEPD or PMF rather than using the concept of frequency as a design criteria. The false sense of security associated with 100-year design would be diminished, and a more realistic design descriptor introduced. The analysis of risk and uncertainty associated with project design could still be evaluated in BCA, but without the common misconception that the 100-year design flood could only occur once every 100 years.

Choosing Between Projects

Just as BCA can be used to evaluate, compare, and size alternatives, it can also be used to help establish project priorities, i.e., help choose between projects competing for the same funding. Project prioritization requires consideration of when a project should be implemented, as well as other factors such as probability of implementation, political significance, and so on. The agency survey indicates that flood control projects, like project alternatives, probably are not prioritized systematically.

Evaluating Flood Control Policies

The final step in the logical chain of potential uses for BCA is policy analysis. Alternative policies such as on-site versus regional stormwater
detention systems could be evaluated prior to implementing a new policy. Similarly, a community may want to know if its participation in the National Flood Insurance Program is fair to insurance policy holders. Evaluation of flood control policies, prior to implementation, could help minimize the effort needed to prioritize projects or choose between alternatives by establishing base values and other parameters to be used in the analysis.

The hypothetical example cited earlier used an assumed value for increased runoff volume produced in urbanization. The type of project constructed depends on the value assigned. If the value were high, the owner may want to keep the water on site for landscape irrigation or other uses, and not pass it through to recharge the public aquifer. On the other hand, if the value were low, paving the bed of the channel, which could decrease recharge, may be an acceptable element of project design, especially if the runoff would recharge to the aquifer somewhere downstream. Policy evaluation can benefit from this kind of sensitivity analysis using the BCA approach.

**The Benefit of Benefit Cost Analysis**

Benefit cost analysis and similar economic performance indicators are regularly used in federal water resource projects, and in other related and un-related fields. In the foregoing examples, application of a simplified BCA
to a relatively simple design problem and introduction of the EI concept resulted in the forced recognition of design issues and public policy issues that may have otherwise been unnoticed or ignored. Admittedly, the numbers and conditions of the urban development BCA example were manipulated to prove a point. They could, however, have been manipulated to show that the developer was getting a bad deal and did not know it. BCA has other potential uses in evaluating projects, priorities, and policies. The point here is simply that BCA is a valuable tool that can help raise, clarify, and resolve important issues and help refine project design. The final outcome is hopefully more informed decision making that yields a better and more creative overall drainage project.

An often-repeated criticism of BCA is that there are unquantifiable benefits or costs which should be considered. However, BCA and EI done fully can provide the comparison between alternatives that permits a reasoned rather than emotional choice between alternatives - and, thereby, quantification of the unquantifiable, say for the money value of looking at a cottonwood tree rather than soil cement. Even if the quantification is not precise, and even if only a description of impacts is provided to decision makers, it is much better than excluding these factors from the project design and alternatives evaluation process.

A major benefit of BCA/EI is that it provides a framework in which the laws of nature and the rules of man can be rationally and simultaneously
addressed. On a broad scale, floodplain management and engineering design policies may be evaluated. Then, community needs can be defined and prioritized. Projects to fulfill these needs may then be identified, prioritized, and approved. Funding for the projects' implementation, construction, and maintenance can be assessed with similar techniques. Finally, project performance can be monitored and compared with project goals in an ongoing evaluation procedure.

The survey provided in this chapter demonstrates that this level of analysis probably is not conducted at the local and regional levels, and probably not even at the state level. Without such analyses, it is impossible to determine whether our intentional and unintentional investment in drainage systems is "profitable". We are unable to tell whether elected officials, governmental agencies, and drainage project designers are representing the public interest. Clearly, the benefit of BCA/EI is its value in identifying the best floodplain programs and policies, as well as selecting the best projects and people to undertake them.
CHAPTER 6
RATIONAL DRAINAGE DESIGN

Flood plain managers in the Southwest are becoming aware of special hydrologic problems that exist in the desert and have begun a search for more acceptable solutions. This new direction from organizations such as Arizona Floodplain Managers Association (AFMA, 1987) is encouraging, but sometimes professionals are too close to the problems to find the rational solutions they are seeking. Their view may not be broad enough to visualize the political-technical framework in which they function, or they may simply be part of the problem rather than part of the solution.

The purpose of this chapter is to further identify some of the more important ways that desert flood plain management can be made more rational; that is, finding means to fit design and management policies to the desert system. Several concepts, problems, and approaches are categorized into four general areas: hydrology, hydraulics, sedimentation, and institutional issues.

Hydrology

In drainage design and flood plain management, the most important goal of hydrology is to establish flood flow criteria to serve as the basis for delineation of flood plains and design of hydraulic structures. This goal can
not be rationally attained without consideration of desert storm patterns, watershed characteristics, the relationship between rainfall and runoff for the watershed of interest, and the effects of man-made and natural changes in the watershed. Ideally, design flows would be based upon a solid historical record of rainfall and runoff data, and a perfect knowledge of the relationship between the two for each basin of interest. Without this "perfect knowledge", the hydrologist can only estimate the design discharge, and the preferred methods of estimating today are based more on statistics and assumed distribution functions than science.

Statistics, Science, and Computers

The literal definition of the word "hydrology" is the study (-ology) of the earth's water (hydro-). It is an earth science that in recent years has been diverted from disciplined understanding through data gathering, research, and experimentation and the other things that "-ologists" do, to an emphasis on statistical analysis based on incomplete or artificial data sets. This emphasis on statistics is obvious in the journals of hydrology-related organizations and professional societies. Criticism of this diversion has come from several bold and articulate sources, including Klemes (1986), Lindlsey (1986), and Pilgrim (1986). Hydrology is also an important part of hydraulic engineering; since hydraulic engineers had to know enough to design and operate water systems, they found out almost enough. A broad concern for the direction the field of
hydrology has taken is expressed in a special publication of the American Geophysical Union [1986] which contains the views of several of these writers.

The fact that many regulatory agencies use default hydrologic models rather than models based on local rainfall and runoff data is partially attributable to an over-emphasis on statistics. Field calibration of these models is impossible or uncommon because small and intermediate-size urban catchments are ungauged. There perhaps is unfounded confidence in the models because they may (1) result in hydraulically infallible, over-designed structures, (2) be so newly adopted that failure producing events have not yet happened, or (3) be computer-adapted, thereby reinforcing a false sense of accuracy, validity, and sophistication.

Almost every community, including those of the desert Southwest, has an official weather station that collects basic atmospheric data such as precipitation, humidity, wind speed and direction, and barometric pressure. This data is useful to weather forecasters, contractors, farmers, and picnickers but is of limited use in predicting flood elevations, stream discharges, and as general engineering design criteria without related stream discharge information. Even crude, easily obtained discharge estimates based on debris lines and the slope-area method (Baker, 1986) are virtually unavailable for urban catchments.

Larger ephemeral watercourses that flow through urban areas may be monitored by federal, state or local agencies because of floodplain encroachment or other safety-related reasons. In desert regions, such large watersheds are
affected mainly by long duration, low intensity storms, and their runoff response may not be characteristic of urbanized watersheds. Very few rural watersheds such as Walnut Gulch (USDA, 1983) are intensely monitored. Transposition of data to nearby urban areas may be possible, but great care must be used in its urban application. Osborn (1971) discusses that transposition is risky because of potential variation in rainfall between nearby regions.

The emphasis on statistical analysis rather than scientific inquiry can also be attributed perhaps to impatient analysts who can not bear to wait one- or two hundred years for complete rainfall and runoff data, and who also can not resist using their mathematics for *something*. That something has been statistical analysis of whatever real or synthetic rainfall and runoff data may be readily available. With the advent of computers, this approach has been compounded (and confounded).

The computer is an invaluable tool for carrying out repetitious calculations. One such type of repetitious calculation is evaluation of alternative designs, as discussed in Chapter 5. Unfortunately, not many practicing hydrologists and design engineers seem interested in using the computer in design analysis, although notable exceptions are evident, but not prevalent, in the literature.

Another area in which computer application could be immensely beneficial is trying to understand the meaning of data outliers. Most analysts try to "fit" extreme and seemingly out-of-place data to some type of assumed
distribution pattern such as the LPIII. If the outlier does not fit the distribution, or can not be made to fit, it is discarded or otherwise discounted as anomalous if one uses the procedures recommended by WRC (1983). The significance of the extreme value is seemingly ignored because it does not fit in. Its significance, however, is its extremeness, i.e., its relationship with some large, and perhaps limiting natural occurrence (Baker, 1983). Those who investigate the origin of outliers rather than merely discard them are likely to benefit from the insight gained. Commonly used probability density functions have infinite tails, as shown in Figure 6-1, but nature must have limits (Hansen, 1977). If one considers the natural limits of precipitation rates and rainfall duration caused by moisture availability and other factors, the distribution would become asymptotic to this limit, rather than infinite. Failing to recognize the existence of limiting values will invariably skew the estimate of the standard 100-year event, and, for that matter, any extrapolated recurrence interval. Figure 6-2 shows the effect of considering probable maximum events in the magnitude/recurrence relationship. The difference has design and policy implications of obvious importance to flood control and drainage decision makers.

Applied Hydrology in Desert Communities

The common method of estimating peak runoff in urban desert watersheds uses no specific local data and relies entirely on default values, as depicted graphically in Figure 6-3. Some would object to this statement and
Figure 6-1
Limitless - Tail PDF
Figure 6-2
Limited - Tail PDF
Figure 6-3

Conceptual Framework for Default Runoff Estimation Technique
claim that local data is included in the NOAA Atlas, but given the tremendous variability of climatology throughout the Southwest, this inclusion is not necessarily beneficial. Very little field data is needed, and what is required can be obtained from USGS quadrangle maps or, better still, recent aerial photographs of the watershed. General information about the time of concentration, area, and imperviousness are estimated. Rainfall rates are determined using the NOAA atlas, and coupled with the watershed data, the rational method or some modification of it, is used to estimate peak runoff. Stream flow data for urban watersheds is usually not available to correlate the estimate with experience.

The USGS (Roeske, 1986) fortunately has monitored both rainfall and runoff on eight urban watersheds in the Tucson area. The length of data collection varies somewhat between the basins, but averages about eight years. The data can therefore be considered preliminary, but it is never-the-less useful in evaluating the relative performance of different runoff models in comparison with measured data. In other words, these data do not provide a perfect understanding of the rainfall/runoff relationship for the watersheds, but are much better than no data at all. Data from three of the eight watersheds have been obtained from the USGS and are used to represent small, intermediate, and large basins typically encountered in urban design projects in the Southwest. The basin areas range from about 24 acres to 6100 acres, and are described more fully in Figures 6-4 through 6-7. Changes within the watersheds over the period
<table>
<thead>
<tr>
<th>WATERSHED</th>
<th>AREA (sq.mi.)</th>
<th>SLOPE (ft./ft.)</th>
<th>MEAN ELEVATION (MSL)</th>
<th>AVERAGE ANNUAL RAINFALL (IN.)</th>
<th>CHANNEL LENGTH (MI.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Craycroft Wash Tributary</td>
<td>.038</td>
<td>0.18</td>
<td>2770</td>
<td>12.0</td>
<td>0.41</td>
</tr>
<tr>
<td>Alamo Wash</td>
<td>9.56</td>
<td>0.0014</td>
<td>2588</td>
<td>11.0</td>
<td>6.40</td>
</tr>
<tr>
<td>Cholla Wash</td>
<td>1.27</td>
<td>0.011</td>
<td>2531</td>
<td>11.0</td>
<td>3.26</td>
</tr>
</tbody>
</table>

Figure 6-4
Watershed Location and Description
Figure 6-5
Craycroft Wash Tributary Watershed Map
Figure 6-6
Cholla Wash Watershed Map
of record have not been monitored, but aerial photographs with topographic information are available from a commercial firm, and these photographs could provide detailed information about changes in land use, slopes, channel improvements, and the like. For the analysis presented here, data from only a portion of one year is used, and any changes within the watershed during this short period are considered negligible.

Rainfall and corresponding runoff hydrographs for local storms on two of these watersheds are shown in Figure 6-8 and Figure 6-9. These events all occurred in water year 1984, which is cited as a wetter-than-usual period. It represents the year with the most rainfall of the eight years of record. It should be observed that there is virtually no correlation between flow events on the basins during the summer rainy season. When it's raining and the channel flows in one basin, the other basins are usually experiencing neither rainfall nor runoff.

On the other hand, Figure 6-10 shows rainfall and runoff for the general storm of 1-3 October 1983. A detailed account of this disastrous storm is presented by Saarinen (1984), and is also reported by Reich (1984), and others. The general storm is very similar to storms of about the same period during 1977, and documented by the Corps of Engineers (1978). These general storms have large areal extent, and the distribution of rainfall was almost uniform.
Figure 6-8
Craycroft Wash Tributary Runoff - Water Year 1984
Figure 6-9
Cholla Wash Tributary Runoff -
Water Year 1984
Figure 6-10

October 1983 General Storm Hydrographs
throughout the Tucson area. This is evident in the stream flow hydrographs for
the three Tucson basins, which responded similarly throughout the three day
event.

Runoff data from various Tucson-area storms are summarized in Table
6-1 and compared with predicted values obtained by the City of Tucson and
Pima County hydrology methods. Computer printouts and manual calculations
for this chapter are contained in Appendix C. These similar methods are
modifications of the rational formula, and the City method is a simplification
of the County method. Both rely on the NOAA atlas for precipitation default
values, and inherently assume distribution by the Log-Pearson III method, in
compliance with federal mandate.

Of the three methods, each takes its turn predicting the highest value
for one of the watersheds. Comparatively, the rational method is lowest for the
smallest watershed, and highest for the largest watershed. The City and County
methods are consistently close numerically because they are based on the same
procedure, but since the City method is useful only for watersheds less than two
square miles, it could not appropriately be used for the Alamo Wash. The City
method defers automatically to the County method whenever the input area
exceeds model limitations. The County method should not be used for areas
larger than 10 square miles. This also is continued in Appendix C.
### TABLE 6-1

$Q_{100}$ ESTIMATES - VARIOUS METHODS

$Q_{100}$ (CFS)

<table>
<thead>
<tr>
<th>WATERSHED</th>
<th>CITY OF TUCSON</th>
<th>PIMA COUNTY</th>
<th>RATIONAL METHOD*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Craycroft Wash Tributary</td>
<td>175</td>
<td>170</td>
<td>97</td>
</tr>
<tr>
<td>Cholla Wash</td>
<td>2,396</td>
<td>2,446</td>
<td>1,136</td>
</tr>
<tr>
<td>Alamo Wash</td>
<td>N/A</td>
<td>5,343</td>
<td>7,923</td>
</tr>
</tbody>
</table>

(watershed is too large)

*Using NOAA rainfall values and handbook value for runoff coefficient, $C$. 
Regionalized Runoff Methods for Ungauged Watersheds

Another approach at estimating peak runoff from a catchment relies on two factors: (1) watershed area and (2) a relationship between flood peaks and their recurrence within the region. The conceptual framework for such methods is shown in Figure 6-11. Other than watershed area, information about basin slope, channel characteristics, imperviousness, and time of concentration are not needed. It may be expedient in some cases to use regionalized runoff methods to estimate peak discharges from a catchment rather than the methods just described. These regionalized methods for Arizona include Malvick (1980) and Boughton and Renard (1984). A common element in both of these papers is the use of stream flow records over a broad region to create a mathematical or graphical relationship between runoff and watershed for various recurrence intervals. Again, most of these methods require assumptions about probability density functions, and Log Pearson III is again prevalent.

Malvick's approach, which does not use a PDF, resulted in a family of curves for different recurrence intervals based on 3500 station-years of record for 143 gauging stations throughout Arizona, plus 11 stations within the Walnut Gulch watershed. The length of record ranged from six to 69 years, varied from site to site, and some station records were discontinuous. Malvick deleted 38 recorded peak discharges to "avoid biasing the rare events." He also notes that "few, or perhaps none, of the greatest floods were measured with a current
Figure 6-11
Conceptual Framework for Regionalized Runoff Methods
The Malvick method is discussed by Boughton and Renard, who indicate that his method results in estimated discharges that are generally lower than those of other researchers. They suggest that the general shape of the curve plotted from his equation envelopes their own data from Walnut Gulch and other southern Arizona gauging stations, and that Malvick may also under predict Walnut Gulch runoff. They further suggest that if Malvick's curve is shifted one-half log cycle to the left it much better matches the higher estimates of the other methods. The differences could be explained by Malvick's decision to delete the 38 outliers, as mentioned above or for numerous other reasons. However, Carmody (1980) provided a detailed investigation into the same outliers and the observations that established them. Carmody concluded that improper observation and reporting lead to the extreme data, and since Malvick apparently used Carmody's results, then the differences must be explained by climate variations between the study areas or some other reason. The results of shifting Malvick's curve, as suggested, is shown in Figure 6-12.

Boughton and Renard discuss that peak discharges can not be determined precisely and with great certainty. They incorporate a finite-tail PDF whose limit is based on flood data. Comparison between watersheds and between methods of estimate is valuable. Accordingly, Figure 6-13 also shows a modified-Malvick estimate for discharge on the three gauged urban watersheds
FROM: MALVICK (1980)
BOUGHTON & RENARD (1984)

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Figure 6-12
Malvick and Modified - Malvick Discharge Chart
<table>
<thead>
<tr>
<th>WATERSHED</th>
<th>R.I.</th>
<th>MALVICK</th>
<th>MODIFIED MALVICK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Craycroft 2 yr.</td>
<td>2</td>
<td>3</td>
<td>11</td>
</tr>
<tr>
<td>Wash Triburary 10</td>
<td>10</td>
<td>11</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>40</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>MEF</td>
<td>100</td>
<td>250</td>
</tr>
<tr>
<td>Cholla 2</td>
<td>2</td>
<td>75</td>
<td>180</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>200</td>
<td>700</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>1000</td>
<td>2100</td>
</tr>
<tr>
<td></td>
<td>MEF</td>
<td>2500</td>
<td>5000</td>
</tr>
<tr>
<td>Alamo 2</td>
<td>2</td>
<td>350</td>
<td>600</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>1200</td>
<td>2600</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>4000</td>
<td>8500</td>
</tr>
<tr>
<td></td>
<td>MEF</td>
<td>10000</td>
<td>20000</td>
</tr>
</tbody>
</table>

Figure 6-13
Peak Flood Estimates
(Malvick & Modified Malvick Method)
discussed above. Results of this comparison show that the modified method produces values about twice as high.

An Improved Peak Estimating Method for the Desert Southwest

The comparisons provided above, the literature, and professional experience indicate gross simplification in the flood peak estimation procedure, reliance on limited data, questionable assumptions about data distribution, ignoring desert storm types and patterns, deletion of potential valid data, inclusion of invalid data, or some combination of these factors. A more rational method for estimating flood peaks in desert catchments is needed for several reasons, the most important of which is improved confidence in the estimate itself. Present methods are too simplistic to be credible, they ignore too much of what is known, and their continued acceptance discourages basic research about desert hydrology. If the estimates are right, it is for the wrong reasons, and this itself should encourage consideration of a more soundly-based approach.

The method developed here relies on suggestions introduced in previous chapters, and is shown schematically in Figure 6-14. The most important considerations are (1) relating rainfall intensity to both time of concentration and watershed area, (2) differentiating between local and general storm patterns, (3) defining a regional "design storm" based on realization of natural limits to rainfall rates, (4) fitting the design storm over the watershed
Figure 6-14
Conceptual Framework for Desert Runoff Method
and determining the worst-case runoff that could be expected based on such factors as direction of storm travel, likely antecedent moisture content, and surface and channel abstractions; and finally (6) using a pre-determined fraction of the design storm runoff for engineering design and flood plain management purposes. A stepwise procedure for using this approach follows.

Step 1] For the watershed in question, determine watershed area; estimate time of concentration by the best means available, preferably from rainfall and runoff data; identify any significant natural or manmade detention basins within the watershed.

Step 2] Determine from HR 49 whether the basin will experience more intense rainfall for its area and time of concentration from either the local storm or the general storm. If the differentiation is unclear, i.e. for basins between about 100 square miles and 1000 square miles, follow both parts of Step 3, otherwise use only the appropriate sub-step.

Step 3a] If basin rainfall is governed by thunderstorms, determine from HR 49 (or local data, if available) the general pattern and distribution of probable maximum precipitation within the design storm. Prepare a design storm diagram at the correct scale to overlay a map or aerial photograph of the basin.

Step 3b] If the basin is governed by the general storm, determine probable maximum precipitation intensity for the basin from the general storm section of HR 49.
Step 4] Determine from local meteorological data the temporal distribution of rainfall during rainy seasons and the probable antecedent moisture content resulting from previous storms or early portions of the design storm relative to the basin characteristics. This will help estimate the imperviousness of the basin, the runoff coefficient, and whether or not retention within the basin will change peak runoff rates.

Step 5] Using a suitable rainfall-runoff model, or better yet a known relationship, evaluate the peak runoff for the design storm under likely conditions of storm travel, and for local storms, at various positions within the watershed. Consider the effect of imperviousness, high antecedent moisture conditions, and detention or retention within the watershed.

Step 6] Reduce the probable peak discharge from the design storm by an appropriate factor to achieve a design discharge value for engineering and flood plain management purposes. Use a factor of about 0.6 to approximate the 100-year discharge, as suggested by HR 49 for local storms. This could be referred to as the 60% Maximum Expected Flood or 60% MEF.

Step 7] Compare the results of this approach with available data from the watershed or a nearby gauged watershed and look for reasonableness in the estimate. If gauged data is not available, the results can be compared with Malvick, Modified Malvick, or another method that
seems appropriate for the region. Document the assumptions, analytic technique, and recommendations.

This seven-step method has been applied to the three gauged urban watersheds in the Tucson area, previously described in Figure 6-5 to Figure 6-7. Results are shown in Table 6-2. It can be generally concluded that the results lie between the MEF values for Malvick and Modified Malvick methods shown in Figure 6-12. The 60% MEF is comparable with the 100-year discharge estimates by these methods, which is approximately 40% to 45% of the MEF estimated by those methods. It is recognized that the PMP rates are much higher than reported observations for even point rainfall in southern Arizona, where the highest rate observed by Osborn and Renard (1988) is about 3 inches in one-half hour or about 6 inches per hour. Their observations, particularly for the 50-year and 100-year point rainfalls, are significantly higher than estimated by the NOAA Atlas, and would be much lower than PMP estimates from HR 49. However the period of observation really is not very long, nor is all data recorded in short enough time increments to determine the probable maximum intensity of extremely small, intense local storms.

A personal observation of one inch in less than five minutes occurred in September 1988 in the Tucson Mountains. This intensity/duration relationship is approximately what Osborn and Renard report for Walnut Gulch in 100-year rainfall, and is also similar to estimates prepared by Peterson (1986) for Billings, Montana. This represents an intensity of about 12 inches per hour. The basin
TABLE 6-2
DISCHARGE ESTIMATES
DESERT RUNOFF METHOD

WATERSHED

<table>
<thead>
<tr>
<th></th>
<th>CRAYCROFT TRIBUTARY</th>
<th>CHOLLA WASH</th>
<th>ALAMO WASH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Governing Storm</td>
<td>Local</td>
<td>Local</td>
<td>Local</td>
</tr>
<tr>
<td>Time of Concentration</td>
<td>10</td>
<td>30</td>
<td>50</td>
</tr>
<tr>
<td>Runoff Coefficient</td>
<td>.20</td>
<td>.15</td>
<td>.18</td>
</tr>
<tr>
<td>Area (M12)</td>
<td>.038</td>
<td>1.27</td>
<td>9.56</td>
</tr>
<tr>
<td>PMP (in)</td>
<td>8.5</td>
<td>10.2</td>
<td>8.9</td>
</tr>
<tr>
<td>i (in/hr.)</td>
<td>34</td>
<td>20.4</td>
<td>10.6</td>
</tr>
<tr>
<td>MEF (CFS)</td>
<td>165</td>
<td>2,490</td>
<td>11,700</td>
</tr>
<tr>
<td>60% MEF (CFS)</td>
<td>99</td>
<td>1,494</td>
<td>7,000</td>
</tr>
</tbody>
</table>

Notes: Tc and C determined from USGS Data PMP determined from HR49
drained rapidly, and no damage was observed. However, if this storm were centered over a more urban area, flooding of local streets and some inconvenience would have occurred. The *Green Valley News* (1985) reports that a storm in Green Valley, Arizona, dropped more than 3.1 inches in less than 30 minutes on July 19, 1985. It produced a very localized flash flood that caused a fatality when a motorist tried to drive through a dip section. The PMP estimates provided by HR 49 may seem high, but they are not unbelievable when compared with these storms and the intensities shown in Table 3-3.

Because of the similarity in the estimates - they all lie within an order of magnitude - one might ponder which is the best to use. The answer depends upon data availability. If basin area and related descriptors are known and no gauging data are available, either of the Malvick family of curves seems reasonable. The Modified Malvick method would perhaps be preferred when a more "conservative" number was needed. However, if local rainfall and runoff data is available, or if there is a level of comfort in estimating the runoff coefficient and using PMP estimates, then the Desert Runoff Method presented here may be more desirable. The major question is what data is available, what assumptions feel right for the task at hand, and which method best allows the incorporation of these assumptions and data.

The Desert Runoff Method, as used here, relies on very limited data from one abnormally rainy summer and one general storm to calculate time of concentration and runoff coefficient. The times of concentration were about
what would be expected, but the runoff coefficients at first seemed too low. However, field inspection and prior knowledge of the three basins verified the reasonableness of these calculated values. In all three watersheds, walls and other basin alterations provide a lot of abstraction, and there is extensive opportunity for infiltration, both on the surface and in the channels. The effect of urbanization of desert watersheds is not fully understood, and Boughton and Renard (1984) report that the urbanized Rodeo Wash in Tucson has an estimated 100-year flood only one-third the magnitude of a similar size undeveloped watershed in Walnut Gulch. One could conclude that the C-value for desert watersheds should be taken from the lower range of table values for the given land use category, and could also conclude that components of the C value such as abstraction, retention, and spatial averaging of rainfall also reduce the value. Such a conclusion is somewhat speculative without additional analysis, but there are not too many other variables that are reasonably subject to such investigation.

Although the numerical results of the method presented here are not markedly different than some of the other methods discussed, this similarity should not lead to complaisance in selecting a method to use. It is just as important to select a method that incorporates correct assumptions as it is to get a "correct" answer. The key is to find and use a flood estimating technique that does both.
An Idealized Flood Estimation Procedure

Discrepancies between hydrologic methods will continue until a procedure that gives the "right" answer is devised. This can be achieved by a precise understanding of rainfall, runoff, and their interrelationship in a drainage basin. However, this kind of understanding may only be possible from an extensive, long-term research program of monitoring watersheds of the desert Southwest. Data needs are extensive and expensive. Until the research has been completed, we will continue to debate what is the "best" estimate of runoff for any given purpose and hope that the methods we choose prove to be acceptable in the long run. Until an ideal method is devised, we should aggressively pursue watershed monitoring programs and question our current approach, and then base our decisions on the best available information. This clearly is not the rule of practice today.

It is also possible that a perfect understanding of the rainfall/runoff relationship can not be achieved because it might be too complex. If one accepts that this relationship is controlled by natural, physical processes and that there are many things about rainfall and runoff that control a flood, then one should not expect to derive a means to estimate a single number from a given set of initial conditions for the physical system. The concept of a system's extreme sensitivity to initial conditions is referred to as chaos by physicists, and it is the subject of extensive research in that field. If watersheds behave in a chaotic fashion, then the best we can expect to predict is a range of values for
runoff from a given rainfall. Perhaps that range of values is already present in the literature of hydrology, and future research to derive a perfect hydrologic model is unwarranted. Perhaps a good enough estimate of potential floods is possible if the most important controlling factors are known and understood well enough.

Finally, the correct analysis and design procedure should consider (1) what would happen in a maximum expected flood, and (2) what would happen in the 100-year storm. If there is no catastrophe in the MEF, and only minor damage in the 100-year flood, it does not matter if the design flood is really the 25-year or the 500-year flood. What does matter is that the level of protection anticipated from a project is actually provided, and that the effects of bigger and smaller floods are considered. An idealized flood estimation procedure simply strives to provide an appropriately sized project based on a reasoned design.

**Hydraulics**

Once the hydrologist estimates design flows, the hydraulic engineer and flood plain manager use these estimates in project design. Hopefully, the engineer and manager feel comfortable that the flow values that they are given are valid and based on reasonable assumptions for their geographic region. Open channel flow, rather than closed conduit systems, is commonly used in
the desert. Hydraulic analysis of open channel flow relies on momentum, energy, and continuity relationships as well as the ubiquitous Manning and the seldom-used Chezy equations.

Channel Roughness

Channel roughness is one of the most numerically important factors in the latter two equations, and one of the most poorly understood during flow conditions. For instance, in the Manning equation (described in Chapter 4), the highest order relationship of the four variables are area \((A)\) and channel "roughness" \((n)\), both of which are raised to the first power. Hydraulic radius \((r)\) is area divided by wetted perimeter, so that area, \(A\), is cumulatively the highest order variable. The value of \(Q\), in other words is directly (although inversely) proportional to the value of \(n\). \(Q\) is affected more by small changes in area, and less by larger changes in wetted perimeter and slope.

Cross sectional area can be approximated by direct and indirect methods during or after a flow event in an ephemeral channel. Velocity can be accurately estimated only during an event, and only with sufficient instrumentation in the hands of trained personnel. Slope can be estimated and observed during flow events, or afterwards from debris or other physical indicators. The channel roughness coefficient, however, can only be calculated based on determination of the other parameters or estimated (guessed at) based
on judgment. Further, these elements appear as constants in the equation; however, they change in nature during a flood event.

In the desert Southwest, natural channel bed and slope characteristics change with the seasons, with changes in the surface condition of the watershed, and perhaps also with ground water withdrawal. In the latter case, a lowering of the water table may increase both infiltration and in-channel deposition. Today, there is also increasing emphasis by desert communities to retain channels in their natural time-variable state to the extent practicable during urbanization and flood control projects. This makes the job of the hydraulic engineer more difficult because the roughness coefficient (and the cross-sectional area) of earthen channels is a variable compared with the relatively constant roughness and shape of fully improved channels.

The accurate determination of roughness coefficients is crucial to project design, and although reasonably accurate estimates are possible for lined channels, it is a matter of experienced and educated guessing for unlined channels. Resources to help the designer estimate natural channel roughness are available for perennial streams, and most civil engineering manuals give reasonable estimates in tabular form. Most published n-values are the average of guesses of several "experienced" engineers, not measurements. The USGS (Barnes, 1967) publishes guidance that contains photographs of typical channels to further assure the reasonableness of the selected values. These references, however, are not directly applicable to the desert Southwest, and the designers
are left with their own estimates or mandates of the local regulatory agency. The USGS (Aldridge, 1973) also began research on the n-values for ephemeral channels, but the findings were never published or released for widespread use.

Determination of channel roughness is important not only because of its significance in hydraulic design, but also because the Manning equation is commonly used to rate a channel (Simons, 1962). If the channel rating or stage-discharge relationship is incorrect because the n-value is not accurate, data regarding stream flow will also be inaccurate. If inaccurate stream flow records are used in determining a rainfall-runoff relationship, then the relationship could be misunderstood (Carmody, 1980), and use of the relationship for engineering design and flood plain management could result in improperly sized structures or unwise land use. These considerations are summarized in Figure 6-15. Control sections with known channel characteristics are useful in determining discharge rates, but this alone does nothing to assure that channel roughness assumptions are correct.

The n-Value Paradox

The paradox of n-value determination is shown in Figure 6-16. It is interesting to note that if n-values are consistently chosen incorrectly, i.e., the same condition is always assigned the same wrong value, then a drainage project may be appropriately designed. If the n-value is correct in either gauging
STREAM GAUGING

If the n-Value is Assumed Too... Then the Discharge Estimate is Too...

...Low ...High
...Correct ...Correct
...High ...Low

CHANNEL DESIGN

If the n-Value is Assumed Too... Then the Channel Size will be Too...

...Low ...Small (under-designed)
...Correct ...Correct
...High ...Big (over-designed)

Figure 6-15

n-Values In Gauging and Design
n-VALUE ESTIMATES...

...IN CHANNEL GAUGING

<table>
<thead>
<tr>
<th></th>
<th>TOO LOW</th>
<th>CORRECT</th>
<th>TOO HIGH</th>
</tr>
</thead>
<tbody>
<tr>
<td>TOO LOW</td>
<td>MAY BE OK</td>
<td>UNDER-DESIGNED</td>
<td>VERY UNDER-DESIGNED</td>
</tr>
<tr>
<td>CORRECT</td>
<td>OVER-DESIGNED</td>
<td>OK</td>
<td>UNDER-DESIGNED</td>
</tr>
<tr>
<td>TOO HIGH</td>
<td>VERY OVER-DESIGNED</td>
<td>OVER-DESIGNED</td>
<td>MAY BE OK</td>
</tr>
</tbody>
</table>

NOTE: The matrix illustrates the effect of incorrect estimates of channel roughness on post-construction channel adequacy.

Figure 6-16

n-Value Paradox
or design and incorrect in the other, then the project based on a Manning-rated channel flow record will be either over-designed or under-designed. If the n-value is high in one use and low in the other, then the channel will be largely over-designed or under-designed in comparison with other cases in the paradox. Certainly, more measurements are needed, both in the field and in the laboratory, to define channel roughness for desert conditions. In addition, practitioners need better training on the importance of selecting n-values, understanding the time variance of natural channel conditions, and what happens if a "conservative" number is used.

The Long Contraction

Erosion of alluvial channel banks is a problem that desert communities are struggling to resolve in some manner acceptable to diverse special interest groups. The use of structural bank protection such as soil cement or metal sheeting results in increased erosion protection at the expense of project appearance and habitat change. Some communities in Arizona now promote combined erosion protection and habitat restoration/preservation in the same project. This multi-purpose approach can work well in the reaches between bridges and other hydraulic structures, but at the bridges themselves, the primary design goal is still to protect the structure from erosion-induced failure, and to train the direction of flow through the bridge openings rather than at abutments or approaches.
In Tucson and other desert communities, major floods destroy bridges, and government responds by replacing the bridge with a new one that hopefully will not fail. The new bridge may be designed with scour in mind, as well as an awareness that the upstream reach of the watercourse must be designed to protect the bridge abutments from erosion. The designer of a combined bridge and bank protection project such as this can use the long contraction to significantly reduce project costs with little additional risk of structural failure.

The long contraction, as presented by Laursen (1986), and Petersen (1986) can be used to reduce channel width upstream of and through a bridge resulting in somewhat greater velocity and depth of flow through the bridge structure. Additional scour will occur through the contracted reach, and the depth of scour is primarily a function of geometry in the long contraction and the type of bed material. The relationship is shown in Figure 6-17, and the general relationship, based on simultaneous solution of the Manning equation and the DuBoys equation for sediment load is as follows. Notice that the hydraulic radius, $R$, is replaced by depth of flow, $y$, in a simplifying assumption for wide rectangular channel sections.

$$Q = By(1.49/n)y^{2S^{1/2}} \quad \text{(Manning equation, modified)}$$

$$Q_s = BK (t_0 - t) \quad \text{(DuBoys equation)}$$
Figure 6-17

Long Contraction
where... $Q = \text{discharge}$

$Q_s = \text{sediment load}$

$B = \text{channel width}$

$y = \text{depth of flow}$

$K = \text{a coefficient dependent on sediment size}$

$t_o = \text{boundary shear or tractive force}$

$t_c = \text{critical tractive force, dependent on sediment size}$

Solving these equations with the assumptions that (1) the Manning $n$ is the same in the normal and contracted reaches, (2) $t_o$ is much greater than $t_c$, which is probably valid for moderate to high flows, and (3) both discharge and sediment discharge are the same in both reaches, then...

$$\frac{y_2}{y_1} = (\frac{B_1}{B_2})^{0.74}$$

and

$$\frac{S_1}{S_2} = (\frac{B_1}{B_2})^{0.7}$$

Using these relationships and recognizing that bridge construction and maintenance costs are an almost-linear function of deck area, a hydraulic engineer, with the help of a structural engineer, can design the bank protection/bridge project to meet multipurpose goals upstream and downstream of the long contraction, and emphasize bridge protection and structural cost reduction through the contraction.
Using basic optimization techniques as a aid in the design process, a recursive equation or linear programming model can be devised to incorporate competing design criteria (Taha, 1971). The equation is solved using matrix algebra. Simple problems may be solved by hand, but more complex problems benefit from the use of a computer. In this way, an optimal solution is defined based on the restraining considerations. Here, the object function is determined by defining an objective function and restraining equations.

The long contraction occurs naturally along ephemeral channels where rock outcropping occur, and where soil conditions make bank material difficult to erode. Despite this fact, the long contraction may be difficult to implement as an element of project design. In the Tucson area, two recent attempts to design new and replacement bridges with this element were denied. The first case involved replacement of the Cortaro Road bridge over the Santa Cruz River which failed in the flood of October, 1983. The bridge is located in unincorporated Pima County, near the corporate limits of the Town of Marana. Designers felt that the Town would not look favorably on such a design, and that an impending inter-governmental agreement for flood control would be hindered. The replacement bridge and associated bank protection were designed and constructed normally.

The second example involves the Vail Road bridge over the Pantano Wash on the far eastern side of unincorporated Pima County. The project engineers considered using a long contraction as a cost saving measure, but
were dissuaded by environmental considerations. The amount of upstream bank protection was cut back to minimize construction intrusion into a wildlife habitat area that was considered worthy of preservation. Reduction in the length of upstream bank protection made the long contraction unfeasible, and it was not used. In neither of these cases was the extra cost of the longer bridge a design consideration.

Bridges and Roadway Approaches

An alternative or additional design strategy in the above situation is to allow roadway approaches to the bridge structure to intentionally erode away during some large flow event. This concept can protect the bridge itself by providing additional flow capacity in the over bank region, thereby lowering scour through the structure and hydrodynamic forces in the direction of flow. This concept is shown schematically in Figure 6-18. The break away approach can either supplant or augment the long contraction design - they are not mutually exclusive ideas. If the concept is used with a long contraction or not, it will prohibit travel across the bridge until the approaches can be repaired. The cost of repair and the indirect costs associated with temporary re-routing of traffic need to be considered during design, but savings from reduced construction costs can be used to fund other projects of local importance. This concept, of course, is not recommended for every bridge across a given watercourse because a major flood event would eliminate all vehicular access
Figure 6-18
Break Away Approach Concept
across the river until repairs are made. The less important routes could have break away approaches, with full structural protection being provided at more important roadways. Also, emergency personnel need to be advised about the nature of such an approach design so that they are prepared to close the roads prior to approach failure.

Sedimentation

Sediment transport is frequently neglected during the planning and design of drainage systems, although erosion can undermine structures and induce failures, and sediment deposition can raise water surface elevations and cause flooding as well as structural failure. Long term sediment transport conditions need to be an integral consideration in the design process. Sedimentation must be considered during drainage design because (1) a channel that is fully lined can still fill with sediment and not perform as designed; (2) a channel that has bank protection can fill or scour and not perform; and, (3) a natural channel can fill, scour, and move laterally. The designer needs to recognize that a balanced discharge and sediment discharge can occur only under one set of width, depth, and slope conditions. As discharge and sediment transport change, so do channel geometric requirements. The key is to design drainage systems to accommodate changing flow and sediment transport rates.
To ignore the inter-relationships of discharge, sediment transport, and channel geometrics is counting on luck, rather than "good" design practice.

Bank Erosion Protection and Grade Control Structures

The previous discussions about bank protection beg the question "If the banks are protected and do not erode, where is the sediment supply?" The answer is both simple and complex. During most flow events, the channel bed will supply the sediment otherwise provided by the banks, leading to upstream erosion and downstream deposition on the bed. Over a series of events, a new channel profile will evolve, representing a new steady-state regime for the watercourse. The new channel profile will be flatter, thereby lowering velocities and increasing depth of flow above the deepened channel bed. The net change in water surface elevation needs to be evaluated to determine if it exceeds some acceptable elevation. In normal, natural streams bank rebuilding (though slower) probably equals bank erosion. With bank protection both are probably inhibited. In a single flood, banks can erode, but the sediment eroded results in bed aggradation.

The same argument holds for grade control structures which are designed to maintain a certain bed profile to control erosion and water surface elevations. It is not uncommon to incorporate grade control structures into bank stabilization projects in an attempt to limit the bed erosion caused by reducing the sediment load previously supplied by the banks. This needs to be done
with care because little is known about sediment transport processes in desert channels, although case histories of fluvial channel adjustments in both width and profile have been reported by Chang (1982).

Recent studies of the configuration of grade control structures indicate that the sloping sill is more acceptable than vertical structures (Laursen and Flick, 1983) because scour is reduced mainly due to the geometry of the sill. However, the recommendations to use sloping sills have not yet been widely implemented in Arizona projects.

Alluvial Fans

Alluvial fans are common at the base of desert mountain ranges where steep mountain streams discharge from an apex onto moderately steep alluvium. Fans represent unique challenges in drainage design because flow paths downstream of the apex can change during and between runoff events, and because flow rates generally decrease downstream as channel abstractions occur.

Two major concerns are apparent in developing desert communities on alluvial fans. First, is the difficulty in designing drainage systems when the flow paths have a habit of changing location abruptly, and each channel needs to be designed to carry larger combined flow. Second, is establishing reasonable flood plain management policies when the flood prone areas and erosion hazard area can change periodically and erratically. Both considerations are important in alluvial fans subject to urbanization, and they represent a difficult challenge.
to hydrologists, flood plain managers, civil engineers, and land planners.

The path of mountain runoff can be controlled by constructing drainage channels from the apex of the fan to some logical downstream point, such as a major natural channel, and channelizing direct runoff on the fan to these constructed channels. This fully-structural approach is capital-intensive and may only be economically feasible where land values are high. Lined channels also may aggrade during a large flood and cause flow to "jump out" of the channel. An alternative strategy is to limit development so that natural flow changes on the fan have minimal effect on manmade changes on the fan. Low density development could be accommodated by orienting homes and related structures in the direction of flow, providing sufficiently high floor elevations to limit flooding, and minimizing the berm effect of roadways by allowing recurrent flow over the top of the roads, rather than collecting and re-routing flow under the roadway in culverts. Both solutions are expensive: in the first case, the "engineered" solution has a large construction price, and in the second case, high-value land uses can not be realized because density restrictions would limit profitability.

The issue of flood plain management on alluvial fans of the desert is critical in the Tucson region. Federal agencies such as FEMA (1985) have adopted methods for defining flood plains on alluvial fans that are not well suited to desert conditions. Application of these procedures by FEMA to portions of the Tucson region have subjected numerous properties to
requirements of the flood insurance program when their inclusion in the program is seriously questioned by local authorities (Pima County, 1988).

Institutional Issues

The technical, design, and policy-related issues discussed herein should be considered in the light of numerous other institutional considerations that can impact informed decision making. These considerations can be categorized as (1) legal, (2) ethical, and (3) environmental, and each category represents diverse opinions from professions that relate, at least indirectly, to the design process. The difficult task is finding an opportunity to teach designers that these considerations exist and must be considered, and somehow help them incorporate such issues into project design, for the good of the project, as well as for their own.

Legal Considerations

The design of drainage systems is governed by legislation, case law, and rules of professional conduct as well as the laws of nature. The designer unquestionably has a fiduciary duty to maintain and protect the public health and safety. Legal considerations regarding this fiducial role vary markedly from place to place and over time. Therefore, the designer is well advised to document design assumptions, methodologies, project goals, and constraints
throughout the design and approval process. At a minimum, the designer should be able to demonstrate that the selected design is founded on state-of-the-discipline knowledge applied in accordance with accepted professional practice.

The arid Southwest is subject to federal, state, and local control that simply must not be ignored. The difficulty of incorporating these considerations into project design lies mainly the designer finding they exist and understanding their importance. The designer needs to discuss design assumptions and goals with the client, with affected property owners, and in most cases also with regulatory agencies. If drainage patterns are going to be altered, it should be done in a manner consistent with local flood plain ordinances and state statutes. If permits are required from local, state, or federal agencies, they should not be intentionally or unintentionally overlooked. Generally, a design that meets permitting requirements will be legally defensible, but this alone does not mean that the design is optimal, in the best interest of the client, or free of liability.

Ethics

The designer needs to retain control of the project from a technical standpoint to the extent practicable, and not let legal and permitting agencies impose standard (or old) concepts and practices, especially when they are not necessarily optimal, or perhaps not even "correct". This raises the issue of professional ethics. Should the designer blindly assume that agency-mandated
procedures are "good enough", or should the designer critically review these procedures prior to embracing them during project design? After all, engineers or other professional registrants who stamp a set of plans attests to the quality of the work, not merely to their involvement and responsible charge of the design process. This question is difficult to answer, and certainly depends on the circumstances. If the hydrologic method has been formally adopted by a regulatory agency, and, when applied properly, it produces results that lead to acceptable design, it probably can be used with confidence. If the method fails to meet either of these criteria, the designer is well advised to be skeptical, and at least should carefully document why a particular method was used.

In our litigious society, engineers and designers with errors-and-omissions insurance have an attractively deep pocket for plaintiffs to try to stick their hand into. Lawsuits against engineers as well as governmental agencies abound after catastrophic floods or the failure of hydraulic structures. Quite simply, engineers who are neither familiar with hydrologic methods nor fully document their work, need both a good lawyer and lots of insurance coverage.

Environmental Issues

Finally, the issue of environmental protection along ephemeral channels of the Southwest has recently become a design issue. The basic problem is to
find a way to provide flood protection without degrading riparian habitat, wildlife corridors, equestrian paths, or trails along the watercourse. In other words, the designer may be asked to provide flood protection without altering natural drainage corridors. The 404 permitting process is currently the singular emphasis and regulatory tool that helps assure that environmental groups have a say in the drainage design process. Although some local floodplain management ordinances may also attempt similar regulation, the penalty for local violations is small compared with the potential fines of up to $50,000 per day or two years in jail for violating the Clean Water Act (Corps of Engineers, 1984). Environmental scrutiny may seem frivolous to the project engineer, but it can have obvious personal impact.

It is possible for environmentally astute individuals to abuse the provisions of the 404 process to attain personal goals. A project can be delayed for long periods of time while remote issues are addressed in response to environmental questions of inflated significance. This can forestall decision making, construction activities, and the delivery of project benefits. Environmental delay tactics need to be procedural rather than substantive to cause great delay. A wise designer will be aware of the issues and comply with both the letter and the intent of the Clean Water Act, even though it seems grossly mis-applied in the desert Southwest.
Maintaining the environmental attributes of a watercourse is difficult on both public and private projects, mainly because it is hard to quantify the value of the natural state. Unimproved land is of value only if it can be used for something, and residences can usually be built only where floods are unlikely. Any property adjoining a watercourse is prone to flooding or erosion damage unless it is somehow protected. If the land is to be built upon, the riparian habitat will change or be destroyed. The example in Chapter 5 shows how these issues might be considered in the design process, but cooperation and flexibility by the landowners, their consultants, and the regulatory agencies are needed. The attitude of "not in my back yard", or NIMBY, is widespread in neighborhoods desiring to thwart what they view as an intrusive project.
This dissertation presents a synthesis of the current practice of applied hydrology in the desert Southwest. The Tucson, Arizona, region is cited as a typical example of how drainage systems are designed, and the technical and policy basis for design decisions are reviewed. The emphasis throughout these discussions is on smaller watersheds which receive most of the rapid urban growth now prevalent in the Southwest. Numerous recommendations to improve the design process are considered, categorized in this chapter as (1) technical recommendations, (2) policy recommendations, and (3) societal/professional recommendations. Then, for the sake of discussion, the recommendations are presented again in the form of drainage regulations for a hypothetical desert community. This transformation leads into concluding remarks, which may be equally valid in non-desert regions as well as the desert Southwest.

**Technical Recommendations**

Technical recommendations address the engineering aspects of drainage design, as well as application-related fields of science. These recommendations emphasize that the arid and semi-arid Southwest is distinctly different from other regions of the country and that the Southwest requires a different
drainage design process. It is not concluded that existing drainage systems are
either grossly under-designed or over-designed. It could easily be that the
designs are reasonably good. However, it is concluded that inherent
assumptions of the design process are not sound because, for the most part, they
ignore the desert physical system. Rainfall patterns and surface conditions are
so different in the desert - and so variant throughout the desert - that each local
region of the Southwest needs independent analysis founded on the best
information available. Each region also needs a rainfall/runoff data gathering
program to assure that the design procedures used locally meet local needs. If
existing methods used by drainage designers yield reasonable results, it is only
because numerous simplifications and errors in logic balance out. In other
words, if the answers are right, they are "right" for the wrong reasons. The
purpose of the following recommendations is to help arrive at design decisions
that are "right" for the right reasons.

Rainfall Patterns

The meteorology of the Southwest is described in Chapter 3, and it is
evident that many desert locations experience distinct summer and winter rainy
seasons. During the summer rains, very intense local storms (sometimes
thunderstorms) can produce flash flooding on small drainage basins. General
storms of the winter have a broad areal extent, long duration, and lower
intensity rainfall. These storms can produce floods on large drainage basins.
The literature indicates that local storms produce flooding on basins smaller than 100 square miles, and general storms produce flooding on basins larger than 500 square miles. Basins of intermediate size need to be evaluated carefully to determine which type of storm produces more severe flooding. The literature also demonstrates that local storm intensity is a function of two variables - area and duration. For instance a 30-minute storm on a 50 square mile watershed would be more intense than a 3 hour storm on the same watershed, both storms having the same recurrence interval.

These two factors - seasonal variation and the area/intensity/duration relationship - are frequently ignored in the design of desert drainage systems. Further, information in Chapter 6 shows that the intensity of small local storms is probably greater than predicted by the most commonly used method - the NOAA Atlas. Local design methods should be improved to inherently consider these factors. This can be achieved by collecting rainfall data within each region and using it in the design process, or at least using it to verify the accuracy of present methods. If the present method is the NOAA Atlas, it is further recommended that another method such as the Probable Maximum Precipitation provided by HR 49 be used. This suggestion has recently been made by Petersen (1986), Bax-Valentine (1988), and Osborn (1988). The main point is to use rainfall intensities suitable for the storm type, area, duration, and location of the watershed in question.
Runoff Rates

Local rainfall records are usually much better than streamflow records. Although many larger ephemeral streams are gauged, few of the small urban watersheds are monitored. This lack of flow record makes understanding the rainfall-runoff relationship even more difficult than usual. In this study, data on three urban watersheds in Tucson were examined, and the runoff coefficients (the "C" in the rational method) were determined to be lower than what is usually assumed in the design process. It was also discussed that stream bed infiltration can be considered as a negative base flow, and that some flows may not leave the watershed due to channel abstractions. Clearly, much better stream flow data is needed to understand why the coefficients are low, what effect negative base flow can have on discharge and sediment transport, and how changes within the watershed affect runoff rates. Local agencies develop and implement a small watershed stream flow gauging program and collect data that will help understand the rainfall/runoff relationship within their jurisdiction.

n-Values

Channel roughness needs to be accurately, or at least consistently, determined in the channel gauging and drainageway design processes. The n-value paradox discussed in Chapter 6 demonstrates that research into the roughness characteristics of ephemeral streams is sorely needed, and that results must reach practitioners to be useful. Channel roughness research should be
undertaken as an integral part of the stream flow data collection mentioned above.

Statistical Methods

Statistics has been used to both explain and discard extreme data, and to expand limited data sets. Computers have been used to manipulate limited data sets to the point where their limitations are obscured, and the quest for better data has become torpid. Most of the statistical methods used in hydrology, including the ubiquitous Log-Pearson Type III, do not recognize the finite limits of the natural system, and infinite floods are often predicted at very small recurrence intervals. Although the various methods tend to agree within the range of the data and slightly beyond, they tend to vary widely in extrapolated regions. If, however, natural limits in rainfall and runoff are used to constrain the outer bounds of these probability density functions, more acceptable extreme values would result.

This approach was used by Malvick (1980), as discussed in Chapter 6, to provide a family of curves relating drainage area and discharge at different recurrence intervals. Malvick's method, and a modification based on data from an intensely-monitored watershed, are compared with predicted values by other Tucson-region methods, and it clearly produces reasonable results. Methods like Malvick's should be developed for various regions of the Southwest and be used
for design in ungauged watersheds and as a means of checking the results of more sophisticated methods.

Paleohydrology

The study of the relationship between rainfall and runoff can be greatly enhanced by incorporating paleoflood research. Field data can be used to extend the data set relative to discharge, precipitation, and watershed conditions. This would be particularly beneficial for projects that are designed for very large floods, or whenever the duration of the data is limited. In the latter case, the paleodischarges could help select an upper bound to flood discharge, or at least help select a more suitable probability density function. Paleohydrology should be recognized as a valuable tool in applied hydrology and also needs to be incorporated in local flood studies.

Sediment Transport

Damage from desert floods is often caused by bank retreat during peak flows, not by inundation of the overbank area. Using definitions from the flood insurance program, such events do not qualify as floods, but rather as erosion hazards. Erosion is also responsible for bridge failure and damage to other structures and public works within and along the channel. Fully-lined channels can fill with sediment, channels with bank protection can erode or fill in, and natural channels can erode, aggrade, or move laterally.
Sediment transport in ephemeral channels is a complex process wherein the channel is continually striving for a state of sediment equilibrium. Although alterations of the natural watercourse may intend to help balance the transport process, long-range changes need to be considered. Mining of stream beds for fill material and for sand and gravel can radically alter the balance. Only limited tools are presently available to help answer questions regarding the long term effect of in-channel mining. None-the-less, the designer needs to consider the effects and consequences of altering the sediment supply in a natural channel.

Sediment transport can be a useful tool, as described in Chapter 6, and can help minimize construction cost for bridges and other structures. The long contraction can be utilized in conjunction with bank protection to decrease the length of bridges. The concept of the break away approach can be used to increase hydraulic capacity by allowing a portion of the bridge approach to erode, thereby increasing available flow area. Although additional research is needed in the overall field of sediment transport on ephemeral channels, enough is presently known to design these types of facilities with as much confidence as regular bridges and approaches are designed today.

Consideration of sediment transport processes should become an integral part of the drainage design process, and the effects of deposition and scour need to be considered during project evaluation.
Policy Recommendations

Policy recommendations address the ways in which drainage designs are evaluated and how decisions are implemented. They generally address the limited range of alternatives considered, and the lack of evaluation of the few alternatives that may be produced.

Range of Alternatives

The informal survey provided in Chapter 5 indicates that the 100-year flood is the singular design criteria for most local drainage projects. Designers often ignore other recurrence intervals for design purposes, and they ignore the damage potential of floods of larger or smaller recurrence intervals. This limitation is attributed to presumptions about the role of the 100-year flood in local compliance with and participation in the National Flood Insurance Program. The NFIP does not preclude using other recurrence intervals, but it does require protection from the 100-year "base" flood. This can be achieved by means of structural or non-structural solutions, or some combination of solutions.

Misconceptions about both the flood insurance program and federal dominion over local drainage design have limited the range and creativeness of drainage design alternatives. We have attempted to allow the rules of man to supercede to laws of nature. The cost-effectiveness of drainage systems and the level of protection from something bigger than the 100-year flood are ignored.
Project designers should consider structural and non-structural flood plain management techniques for a range of recurrence intervals, and not be blindly limited to design for the standard 100-year flood.

Benefit Cost Analysis

BCA is commonly used by federal water resources agencies to evaluate project effectiveness and to assess alternative designs. Similar methods are rarely used by local agencies, and there is no way of telling whether or not a local project is warranted as designed. BCA forces consideration of many facets of the project that are otherwise ignored. The designer may find that the procedure itself opens the door to creativity.

BCA should be used in conjunction with an array of project alternatives to optimize design and to help select the best alternative to meet local needs.

Equitability Index

The equitability index (EI), introduced in Chapter 5, provides a simple means of comparing how fairly project costs are distributed among project beneficiaries. The question of who benefits from and who pays for improvements is important, but is seldom directly addressed. Combined with BCA, the EI is a valuable tool that can help further refine the design process and help identify levels of financial participation by each project beneficiary.
Also, both can be used together to establish project priorities and project scale and to evaluate the advantages of flood control policies.

It is, therefore, recommended that the EI be used to add meaning to benefit-cost ratios whenever BCA is used in project design or evaluation of alternatives.

National Flood Insurance Program

It is widely assumed that participation in the National Flood Insurance Program (NFIP) is cost-effective and beneficial to the community at large. Federally subsidized insurance premiums are paid by property owners, and the cost of capital improvements for flood control programs are financed mainly through local ad valorem taxes. Maintenance of flood control structures and the preparation of plans and reports to comply with provisions of the NFIP can be extensive. For the desert Southwest, it is questionable whether participation in the flood insurance program is either cost effective or equitable. It is, therefore, recommended that desert communities address the entire range of costs and benefits associated with participation in the program and assess the equitability of the current financing structure.
Professional Recommendations

The final category of recommendations deals with the profession of civil engineering and engineering hydrology, society's trust in these disciplines, and the educational system that trains the professionals. Of the three categories, this affords the best long-range opportunity to improve the overall quality of drainage design in the desert Southwest.

Civil Engineering and Engineering Hydrology

These professional disciplines have responsibility to design drainage systems based on the best available information. Instead, the design is usually based on mandates established by the local agencies, including such criteria as the rainfall/runoff relationship, precipitation rates, and emphasis on structural design. The engineer recognizes that if a project is to move forward, agency approval is required, and it is usually more expedient to use the mandates than to contest them. Some engineers design a drainage project without physically visiting the site and do not view the project during flow conditions. This is neither good practice, nor does it add to the credibility of the profession.

If a designer recognizes that better methods exist, he must "prove" their merits compared with the usual design methods. However, liability for the design rests with the designer regardless of what rainfall/runoff methods are used. It seems more prudent to try to utilize the better method than to
perpetuate the old method solely for the sake of expedience. It is, therefore, recommended that the civil engineering profession take a lead role in improving the hydrologic and hydraulic methods approved for use within jurisdictions of the Southwest and help develop a research and monitoring program to derive these methods.

University Education

The undergraduate civil engineering curriculum at many universities is void of adequate instruction in hydrology as well as sediment transport. There is little formal training available in desert hydrology, and practitioners are left to their own means to discover appropriate problem-solving techniques. Universities should review the needs of design professionals relative to the hydrology of arid lands and strive to improve the quality of undergraduate training in this field.

Desert Drainage Regulations

The foregoing recommendations can be viewed from the perspective of rules and regulations for the design of drainage systems, floodplain management, and land use controls near ephemeral channels. The following "regulations" are in a format similar to those currently used in the Southwest to comply with federal and state guidelines for participation in the National Flood Insurance
Program. The primary emphasis of the rules presented here is desert hydrology without undue emphasis on insurance programs.

Purpose

These regulations are intended to protect public safety and welfare by facilitating the planning, designing, constructing, and maintenance of drainage systems with full recognition of hydrologic constraints and opportunities within the desert.

Scope

These regulations apply to all public and private improvements that alter the flow of surface runoff across a property line.

Responsibility

Public agencies shall plan, design, construct, and maintain public drainage systems, and approve the construction of private systems. Public agencies shall prepare basin management studies and land use plans for all zoned property within the area of jurisdiction. The private sector shall conform with basin management plans during development of private property, and provide funding proportionate to benefits received. A private party shall not be required to provide more than a "fair share" of the drainage system, but may do so voluntarily.
Design Goals

Drainageway design shall mitigate the dangers of rapidly flowing water and the occurrence of erosion damage, and reduce inundation damages to a cost-effective minimum. This may be accomplished by structural or non-structural measures, or by some combination thereof. Design recurrence intervals shall not be mandated, but, instead, they shall evolve during the alternatives analysis process. Runoff shall be considered a valuable asset, and physical characteristics such as transmission losses and erosion shall be used beneficially during the design process.

Alternatives Analysis and Project Funding

Benefit Cost Analysis shall be used as an aid to project design and as a means to select or refine an optimal alternative. Further, BCA may be used to establish project priorities and evaluate alternative plans and programs. Costs shall be both direct and indirect, and benefits shall be both loss-reduction and value-added. Multi-purpose or multi-participant projects shall be evaluated using the equitability index, and financial participation shall be in proportion to benefits accrued.
Hydrologic Methods

Rainfall/runoff relationships for a catchment shall be based upon the best available information, including field measurements within the catchment. If gauge data are not available, local rainfall data or PMP shall be used in conjunction with a rainfall/runoff relationship appropriate for desert conditions. Existing and proposed conditions shall be evaluated relative to peak discharge rates and the volume of runoff, based on the maximum expected flood, a rare flood, and a frequent flood. These rates and volumes will be used to optimize project design to the extent practicable. Seasonal precipitation patterns shall be an important consideration, and the dominant storm type for the basin in question shall be used for project design. Runoff estimates shall be validated for reasonableness against a regional method, such as Malvick, prior to project design.

Drainageway Design

The project engineer shall prepare plans and specifications within the confines of the above stated design goals and the general goals of the basin management plan. Recurrence intervals, cross sections, materials, and other design elements shall not be mandated as a condition of plan approval. Catastrophes shall be avoided, and the probability of property loss from rare floods shall be optimized.
Data Collection

The public agency, in conjunction with other interested parties, shall undertake a cooperative program to evaluate local rainfall/runoff relationships. A network of recording rain gauges, streamflow gauges, and sediment transport measurement devices shall be installed throughout the jurisdiction and tributary areas. Data shall be readily available, and study findings shall be disseminated to design professionals. Cooperative research activities shall address channel roughness, infiltration, time of concentration, runoff coefficients, and the effects of urbanization on these factors. Human intervention within the watersheds shall be regularly monitored. Paleoflood studies shall be used to establish an upper limit to flood magnitude, and to help negate the need to use a probability density function for data extrapolation.

Flood Control Financing

Financing of flood control projects, including annual maintenance costs, shall be based upon actual benefits received. Property taxes based on valuation (ad valorem taxes) may be used only if all properties benefit in proportion to property value. Major construction projects shall be funded solely by the public and private beneficiaries.
Flood Insurance

The community shall be self-insured and flood insurance shall be provided as part of local flood control taxes. The issuance of a building permit shall indicate compliance with local floodplain management programs.

Annual Report

The public agency responsible for flood control programs shall prepare an annual report of progress, construction activities, management programs, and future needs. The report shall be presented at a public hearing, with solicitation of public comment.

Cooperative Task Force

The agency, professional designers, and university officials shall meet regularly as a cooperative task force to exchange viewpoints and help meet the needs of practitioners, educators, and elected officials. The task force shall help prepare the annual report, and advise local government, as appropriate.

Conclusions

This paper addresses the problems confronting professional engineers and hydrologists who routinely design drainage systems in the desert Southwest. It is apparent that the laws of nature are often superceded by the rules of man
in the design process and that the range of project alternatives is usually extremely limited, often including only the 100-year flood.

Despite a long history of requests for better rainfall and runoff data and a better understanding of the rainfall/runoff relationship, design practitioners utilize hydrologic methods developed for perennial streams in humid parts of the country as part of the National Flood Insurance Program. In so doing, many hydrologic methods used for desert watersheds could be considered "technically incorrect" or "scientifically incorrect", as these terms are defined by FEMA. We can do a better, more "rational" job of planning, designing, and financing drainage projects in the Southwest. We can better apply what we know, and do a much better job of data collection, basic research, and dissemination of information to practitioners, elected decision makers, and the public. Efforts similar to those of Mulvany and Kuichling from more than 100-years ago need to be repeated today on watersheds of the Southwest to help gain a better understanding of flooding in the desert.
APPENDIX A
MULVANY (1852)
had directed their attention to the avoidance of the loss of power from back-water by this method, seemed to suppose that the water behind the wheel remained perfectly still, and that each float-board had to contend against the whole of this resistance after it had reached the lowest point in its revolution. But this was not the case, as the back-water had a receding motion communicated to it by the forward action of the float-boards; and in a well-constructed gravity wheel the floats were so formed as to retain the fluid upon them only so long as to be effective, and then rise out of the water in the direction of the backward current communicated to the tail-water.

"On the Use of Self-Registering Rain and Flood Gauges, in making Observations of the relations of Rain Fall and of Flood Discharges in a given Catchment," by Thomas J. Mulvany, Mem. Inst. C.E.I.

My chief object at present is to describe to the Institution a mode which has been adopted, as I think with advantage, of making observations as to the fall of rain, and the consequent discharge of water from catchment basins. It will be necessary, however, that I should make a few preliminary remarks on the general subject of catchment basins, and the circumstances which affect the amount and rate of flood discharges, for the purpose of explaining the objects which I had in view in resorting to this particular mode of observation, and to point out what I expect to be its useful application.

The peculiarity of the mode adopted consists in the use of self-registering gauges, so arranged as to represent in the form of a diagram the particulars of the fall of rain at any period during its occurrence, and the variations of the discharging stream as to height of surface and rate of discharge. I believe that the principle of registry which I have made use of has been applied to tide gauges, although I have not seen any of them; but I am not aware that it has been before made use of to record the facts as to rain-fall. At all events, I have only adopted the simplest and cheapest means of registry which suggested itself. The gauges which have been fitted up have worked very satisfactorily, but it is only an experiment which has been tried as yet, and I have not had time or opportunity to improve much upon the first idea.
am anxious, however, to bring the subject, even in a crude state, under the notice of the Institution, with the twofold hope, first, that the system of observation may be thought worthy of adoption by others who are interested in such investigations—and, secondly, that some improvements may be suggested, either in the construction of the apparatus employed, or in the mode of recording and analysing the results of the registry—which suggestions may lead to the establishment of a general and uniform system of collecting these data, on which calculations of an important nature may afterwards be founded.

The great want of well-established data, and of practical rules deduced therefrom, for the guidance of calculations as to the maximum flood discharge from catchments of various characters, is too well known to require much comment. In fixing the details of a design for any important work of river improvement, many intricate problems present themselves, in the solution of which the Engineer will find that he can receive but little assistance from books, or from the recorded experience of others; and if he relies upon observations to be made with special reference to the case under his consideration, he will find that to lead to correct conclusions such observations should be spread over a very inconveniently long space of time. For instance, if he has occasion to execute works of a particular class on a portion of a mountain river, and visits the site of the works for the first time in a dry season, he will probably find a very wide and crooked river bed, with a small thread of water running through it;—he may ascertain with the assistance of the valuable maps of the Ordnance Survey the extent of district which is unwatered by the river—the general levels of the mountain ridges from which the water descends, and, by some personal inspection, with the help of the map, he will make himself acquainted with the general nature of the surface of the country—whether it be of a porous nature, so as to retain for some time a considerable portion of the rain water which falls on it, or rocky and precipitous, and therefore calculated to discharge the water quickly and with little deduction. But after having ascertained all these facts, and feeling satisfied that each of them must have an important effect on the result as to the maximum flood which he is called upon to provide for, he has no faithful guide, that I am aware of, to help him to a conclusion as to the amount of effect on the discharge due to each or all of these conditions; he is, in fact, left to guess at the result after all, and unless he happens to have had some previous experience of similar
cases, his guess will probably be very wide of the truth. If he takes pains to satisfy himself by observations of the result of any flood which there may be an opportunity of observing, he may probably be much misled by assuming that what he sees, and what may indeed be a very large discharge of water, is a maximum discharge, which it may not be, and to produce which a combination of circumstances as to full of rain and the peculiar character of the catchment may be required, that may not occur more than once perhaps in two or three years, but which it is nevertheless necessary that he should provide for.

We are provided with some data as to the average and maximum fall of rain per diem or per annum in various localities; from which, coupled with observations that have been made as to evaporation and absorption, certain conclusions have been arrived at as to the proportion of the rain water which is usually carried off by the rivers of the country; and in this way rules have been deduced for calculating the discharge which may be expected from a catchment of a given area after a maximum fall of rain. The calculations usually made on this principle show that from 1½ to 1¾ cubic feet per minute may have to be discharged from each acre.

Now this result, although arrived at in an empirical manner, is found to be tolerably near the truth in catchments of an average character, neither mountainous nor very flat, and within certain limits as to extent. But though the calculation may be depended upon for such cases, or possibly for the ultimate result of a large catchment, consisting of a number of subdivisions which vary much in character and extent, it will by no means apply to each of those subdivisions. As an instance of the various results which may be found in different portions of the same catchment, I may refer to the Woodford River, in the counties of Leitrim and Cavan. The discharge of water has been carefully measured in various parts of the catchment of this river when different floods had attained to their greatest respective heights, and it has been found that at Ballyconnell, where the river carries off the water of 90,000 acres of country, the greatest discharge in ordinary winter floods was about 60,000 cubic feet per minute, or two-thirds of a cubic foot for each acre; and in a late very high flood, which may be considered as a maximum, it was found to be as much as 101,000 cubic feet per minute, or about 1½ cubic feet from each acre; while, in the upper portion of this catchment, there is a subdivision of about 5,000 acres, belonging to a mountain river,
from which the discharge in floods has been several times found
to be from 50,000 to 53,000 cubic feet per minute—that is, from
10 to 10$\frac{1}{2}$ cubic feet from each acre. On the other hand, in more
extensive districts, the maximum discharge has been found not to
exceed half or one-third of a cubic foot per acre per minute.

The following statement of a few cases of maximum discharges,
which have been carefully ascertained, will show at a glance the
great difference in the results, occasioned by the circumstances of
the catchment:

<table>
<thead>
<tr>
<th>River</th>
<th>Extent of Catchment</th>
<th>Maximum Discharge per minute</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shannon, at Killala, abt 5,000,000</td>
<td>1,000,000</td>
<td>0.33</td>
<td>Measured previous to the commencement of Shannon works.</td>
</tr>
<tr>
<td>Lower Erne, at Belleek, 674,000</td>
<td>677,711</td>
<td>0.67</td>
<td></td>
</tr>
<tr>
<td>Upper Erne, at Delturbet, 300,000</td>
<td>257,771</td>
<td>0.83</td>
<td>Measured during the very high floods of January, 1861.</td>
</tr>
<tr>
<td>Woodford, at Ballyconnell, 99,000</td>
<td>101,036</td>
<td>1.12</td>
<td></td>
</tr>
<tr>
<td>Yellow River, Co. Leitrim, 5,000</td>
<td>52,125</td>
<td>10.48</td>
<td></td>
</tr>
</tbody>
</table>

Mr. Beardmore, in the introduction to a very useful book of
tables published by him last year, mentions that at Glencuse, in
the Pentland Hills, on the 8th of August, 1846, a discharge of
24,180 cubic feet per minute was produced for four successive
hours, after a fall of 1.88 inch of rain, the extent of the catchment
being 3,820 acres. This is equivalent to 6.33 cubic feet per
minute for each acre. He also quotes some statements made by
Mr. Phillips in his evidence about the London sewers, which show
that after thunder storms the discharge is generally found to be
from 25 to 35 cubic feet per minute from each acre of urban
drainage, and that in the storm which occurred in August, 1846, it
was calculated that as much as 60 cubic feet per minute was dis-
charged from each acre, by some of those sewers.

The results as to discharge recorded above, in the cases of the
Erne, Woodford, and Yellow rivers, were all produced by the
same fall of rain, and, therefore, the variations in the results are
wholly owing to the natural circumstances of the catchments.
There are equally important variations, however, to be observed
in the rates of fall of rain in various localities; and as the circum-
stances of the rain-fall are of as much importance as the nature of
the catchment in the determination of the flood discharge, the necessity for collecting accurate information on this subject may be seen, by comparing the following results of observations already made.

It is stated that 1½ inch of rain in 24 hours, may be considered as a maximum fall, on the general surface of this country, but I have not found anywhere a statement of what may be taken to be a maximum fall in one hour, or in any given number of hours. Taking the average, however, of the above-mentioned quantity, which is '062 inch per hour, it is equivalent to 3'77 cubic feet of water per minute on one acre of ground.

The rain-gauge which I am about to describe, has, however, registered many greater rates of fall within the last month, the greatest of which was at the rate of '187 inch per hour, or 11'65 cubic feet per minute on an acre.

Referring again to Mr. Beardmore's book, it is there stated in the words of Mr. Homersham, that "it is not an uncommon circumstance for 3 inch of rain to fall in hilly districts in one hour." Now this fall would be equivalent to 18 cubic feet per minute on the acre.

It is also there stated, that on 1st August, 1846 (the period already alluded to as having caused such an enormous discharge in the London sewers), there fell in London, in less than three hours, from three to four inches of rain. Now if we take it that four inches fell in three hours, it would be equivalent to 80'46 cubic feet per minute per acre.

And finally, as an example of the difference of circumstances between this country and others as regards rain-fall, I may refer to the fact, that in India it has been found that the average yearly fall of rain at Bombay, for ten years, was 78'1 inches, the greatest fall in any one year, 113 inches, and the greatest fall in 24 hours, 16 inches. If we take only the average of this fall in 24 hours, it will be equivalent to 40 cubic feet of water per minute on each acre, and it is probable, that during a portion of the time, the fall was much more intense than the average.

With such plain facts as these before us, as to the great variations both in the absolute amount, and the comparative rates of rain-fall under different circumstances, and in the effects produced by the natural formation of the earth's surface on the discharge of floods, it is clearly impossible to depend for success in some calculations, upon observations of the average annual or daily fall of rain, or upon measurements of average discharges, or isolated records of
maximum discharges from particular catchments. A widely-spread
and long course of observations is required as to the fall of rain
in all parts of the country, not merely the daily fall, but the
maximum rate of fall for any given portion of time, from a minute,
or an hour, to a day; and an equally extensive course of observa-
tions as to the discharge of water from catchments of various
characters, taken in connexion with the observations of the rain-
fall, so as to afford the means of comparing the effect with its
cause, and showing, not alone the greatest or least quantity of
water discharged, but the whole quantity in any given space of
time, and the rate of discharge at any time that it may be
required.

A great mass of authentic data of this sort must be obtained
with care, and arranged with system, before any successful
attempt can be made to generalize the results, or to form anything
like a satisfactory set of principles for the guidance of calculations.
I have no doubt, however, that if this information be properly
collected, it will afford the means of attaching a certain value, in
the calculation of discharge, to each ascertained condition of the
catchment as to its physical formation, and the circumstances
relating to rain-fall, so as to enable us to ascertain with sufficient
accuracy for any practical purpose, the maximum discharge to be
provided for, or the minimum supply which can be depended upon.

The first matter of importance to be ascertained in the case of a
small or a mountainy catchment, is the time which a flood requires
to attain to its maximum height, during the continuance of a
uniform rate of fall of rain. This may be assumed to be the
time necessary for the rain which falls on the most remote
portion of the catchment, to travel to the outlet, for it appears
to me that the discharge must be greatest when the supply
from every portion of the catchment arrives simultaneously at
the point of discharge, supposing, as above premised, the rate
of supply to continue constant, and this length of time being
ascertained, we may assume that the discharge will be the greatest
possible, under the circumstance of a fall of rain occurring, of
the maximum uniform rate of fall for that time. Thus, if the
catchment be of such a nature as that the rain from all parts of
it will reach the outlet in three hours, and if we assume that
one inch of rain is the greatest quantity known to fall in the
locality at a constant rate in three hours, then we may conclude
that the greatest possible flood will be produced in that catchment,
by such a fall: while twice that quantity of rain, falling in twenty-
four hours, which would be looked upon as a maximum fall, and
which would produce a maximum discharge in a different sort of
catchment, would, if spread evenly over the twenty-four hours,
produce but a very slight effect, comparatively speaking, on the
discharge in this case.

This question of time, as regards any catchment, must depend
chiefly on the extent, form, and rate of inclination of its surface;
and, therefore, one great object for investigation is the relation
between these causes and their effect; so that, having ascertained
the extent, form, and average inclination of any catchment, we
may be able to determine, in the first place, the duration of con-
stant rain required to produce a maximum discharge, and conse-
quently to fix upon the maximum rate of rain-fall applicable to
the case. It is evident that, as the space of time is reduced, the-
amount of the maximum rate of rain is increased. Certain rates
of rain may be found to continue for one, two, or three hours, that
would never be found to last for six or twelve hours; and a certain
maximum rate may be observed for twelve hours, that would never
continue for a whole day.

The next matter of importance is to ascertain the effect which
the various conditions of the earth's surface—as to geological for-
mation, degree of cultivation, &c.—may have on the retention of
a portion of the water which falls on it; and thence to determine
the proportion of the rain-fall which may be calculated upon as to
be carried off by the river or drain, under different circumstances.

If observations were made, in the way I have alluded to, on the
rain-fall and discharge from a great number of catchments, of all
varieties of size and character, it appears to me that by selecting
a number of cases which may happen to be similar in most of
their conditions, but to differ in some one—such as the rate of
inclination of their surfaces—and by carefully comparing the
results in these cases, correct conclusions may be drawn with
regard to the laws of that particular element in the calculation;
and so of the others. These investigations may be materially
aided by having small and suitable catchments selected specially
for the purpose of experiment, as in the case I am about to describe.
Of course the more simple the circumstances of the catchment
selected the better for the purpose of the experiment. And if two
or more cases could be selected for observation, nearly similar as to
form and extent, but varying as to the inclination and nature of
surface, the comparison of the observations in those cases would
be most useful.
OF CIVIL ENGINEERS OF IRELAND.

With a view to obtain information on these subjects, I had several rain-gauges prepared about a year ago, of a very cheap but sufficiently accurate construction, and got them fixed up at any stations where I had assistants who could attend to them. Arrangements were made for registering the fall of rain, not only daily, but hourly, during the continuance of any important fall, and places were selected wherever convenient for measuring the discharge of floods from catchment basins of known extent. These latter observations were to be made hourly, also during the rising and falling of the flood, and in connexion with the observations of the rain-fall.

This system of observation, however, although very useful in lieu of a better, is very difficult to carry out fully, and is far from being perfectly satisfactory in its results; I therefore proposed in certain cases to adopt these self-registering gauges, which should insure a faithful record of the fall of rain, and of the fluctuations of the flood, at whatever hour of the day or night they might occur. The apparatus used for the purpose is very simple and inexpensive, and has been found to work in a very satisfactory manner.

The above drawing will show the way in which the registry of the rain-gauge is effected. The measuring vessel into which the water
from the gauge is led, and in which the float works, must, of course, be made to bear a fixed proportion in point of area to that of the receiver of the rain-gauge. In the present case the proportion is one-fifth, so that a fall of one inch on the earth's surface is represented by a rise of five inches in the measuring vessel. The paper for the diagram registry is ruled accordingly, being divided by horizontal lines into spaces of half an inch, each such space representing one-tenth of an inch of rain-fall; and it is divided by vertical lines into spaces representing the distance which the board is moved in one hour. The gauge is set at noon each day, and does not require any attendance for twenty-four hours.

The only gauge of this description which has been fitted up yet, is in the Drainage Office at Keonbrook, on the line of the Shannon and Erne Junction Navigation. It was put up by Mr. Leonard, the Resident Engineer on that portion of the works, who has given a good deal of consideration to the making of the required observations, and has followed out his experiments with great success. He selected a suitable place, very near the house where the rain-gauge is fixed, for observing the discharge from the catchment of a small stream. This catchment is favorably circumstanced for the purpose of experiment, being of a simple form, and tolerably uniform rate of inclination, and having no flat under-flooded land, nor any lakes, except one small one of four acres in area, situate near the outlet.

Mr. Leonard having ascertained that the rain-gauge acted so well, proceeded to put up a self-regulating flood-gauge at this place, which works in exactly the same way as the rain-gauge, but he adopted a very great improvement, which had not suggested itself to me. He determined to pass the stream over a waste-board, carefully constructed, and of a certain length (ten feet, in this case), and thereby was enabled, by calculating the discharge according to the height passing over the waste-board, to prepare the paper for the diagram registry in such a way as to indicate the rate of discharge at any period of time. This gauge has now been at work for a couple of months, and has been found to answer all its purposes perfectly. The following copies of the diagram registries of the two gauges will explain the way in which they present the information:
These diagrams show the particulars of the rain-fall at Keonbrook for five days (January 12th to 16th, 1851), and the particulars of the discharge during the same period, from the adjoining catch-
ment of the Ballytrely stream. It will be at once seen that they afford the means of recording almost every circumstance connected with the fall of rain, and the corresponding discharge. As regards the former, it can be seen not only how much rain fell, but at what time the rain commenced, how long it continued, and at what rate it fell during that time, or during any portion of the fall. While as regards the discharge, it is recorded what was the rate at which the stream was discharging before the rain began; how soon it became affected by the rain; how long it continued increasing, and to what maximum it attained; how long it was gradually diminishing, until it fell to its former rate, or began to be affected by a fresh flood; and, finally, what was the total quantity of water discharged per diem, or during any given number of hours. To ascertain this last fact, it is of course only necessary to note the average rate of discharge per minute for each hour of the period required, add them together, and multiply the sum by 60, which will give the total quantity of entire feet of water discharged during that period.

If this mode of making observations should appear to others to be as efficient and valuable as I believe it to be, there can be no difficulty about its general adoption, as it involves a very trifling cost. The rain-gauge may be fitted up anywhere for less than a pound, exclusive of the value of the clock, which may be supposed to be required for other purposes; but even if procured for the purpose, it need not cost more than another pound, or very little more. A common Dutch clock will answer, which will keep anything like fair time for twenty-four hours.

I think it would be very desirable that rain-gauges should be constructed on this principle, whenever they are required to be used; whether in connexion with observations of flood discharges or not; as it affords the means of obtaining the information, to the necessity of which I have already alluded, relative to the maximum rates of rain-fall for any given space of time. It is quite practicable to make them give as detailed a registry as can possibly be required, both as to depth of rain and the division of time; the first, by altering the proportions between the rain receiver and the measuring vessel; and the latter, by increasing the size of the pinion attached to the clock.

The system of gauging the discharge over a weir or waste-board is, unfortunately, not always practicable; but it can be applied in a great number of cases: in most small catchments which may be selected for experiment, and in many large districts, where perma-
nent weirs are already erected. Of course it cannot be adopted, except at a great expense, in large mountain streams, where no weirs at present exist; and yet these are cases in which such observations should be frequently and carefully made. In most of these cases, however, the stream may pass through a channel of uniform section for some distance, and a gauge may be fitted up at that place to register the rise and fall of the surface of the stream; recourse must, however, be had to actual measurement for the velocities at various heights. In one case of this sort I endeavoured to attain the object of a correct registry as nearly as possible by fixing two gauges at a stated distance apart, and by having the height of water accurately registered at both places, with reference to a common datum, I intended to record not only the actual depth of water in the channel, but also the rate of inclination of the water surface, and to determine the quantity of discharge by calculation. The gauges in this case, however, did not act satisfactorily; the current was too strong in the river to admit of the floats being placed there, and two wells were made a short distance in from the side of the channel, in which the gauges were fixed. These wells, however, soon became filled up with matter brought down by the floods, and although that might, I think, have been avoided by proper arrangements, yet, as I found there would be, after all, so much calculation required to ascertain the discharge, and as I am not satisfied of the universal correctness of the formula usually adopted for calculating the discharge by means of a given section and rate of inclination, I gave up the idea of fitting up the gauges in the way I had at first intended—thinking it better to run the chance of having careful observations made every hour or half-hour during the rise and fall of the floods, which pass off quickly in that place. It is, however, a very desirable matter to be considered, how the self-registering flood-gauge may be best applied in those cases where it is impracticable to measure the discharge by means of a weir.

If any extensive system of observations should be undertaken on the principles to which I have alluded, it would be very desirable that a general form of registering should be adopted for recording the results obtained from the diagrams; and that such form should contain all the important information, and yet be as simple as possible. I have carefully tabulated the results of the rain and flood-gauges at Keonbrook, for the period from the 30th of December to 31st January last. I selected this period because there were four days previous to the 30th December on which there was no rain.
whatever, and as the stream was about at its lowest state when the rain of the 30th December commenced, I considered it a favorable time to begin the comparison between the rain-fall and the discharge. This form of registry, an extract from which is annexed, is, I am aware, open to many modifications, and I should be glad to receive any suggestions as to the best means of simplifying its form, or extending its utility.

It has occurred to me also that the general results of these registries may be with great advantage brought tangibly before the eye by means of diagrams. I have prepared a diagram (vide Plate No. 1.) showing the average rates of rain-fall upon, and the average rate of discharge from, each acre per diem for the period already mentioned, from 30th December, 1850, to 31st January, 1851; each day is represented in the diagram by a narrow column, in which is drawn, to an uniform scale, the average rate of rain-fall and of discharge for each day—unity being one cubic foot per minute on each acre. The rate of rain-fall is distinguished from the rate of discharge by different shading; and the difference between these two rates, being the effect due to absorption and evaporation, is thus brought strikingly before the eye. If this diagram were continued, so as to show the comparative amounts of rain-fall and discharge in a given case throughout a whole year, or a number of years, it would afford valuable information as to the effect of evaporation at different seasons, and under various circumstances.
### Proposed Form of Registry

<table>
<thead>
<tr>
<th>Date</th>
<th>Rainfall</th>
<th>Number of Hours</th>
<th>Rate of Fall of Rain per Hour</th>
<th>Remarks to Particular Falls of Rain</th>
<th>Quantity of Water Discharged per Minute from Each Drainage Area (in Inches of Water)</th>
<th>Discharge per Minute from Whole Catchment</th>
<th>Equivalent Average Discharge per Minute from Each Catchment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan 8</td>
<td>0.45</td>
<td>857,000</td>
<td>1.130</td>
<td>110.74</td>
<td>200.60 65 75 140</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan 10</td>
<td>0.20</td>
<td>200,000</td>
<td>0.20</td>
<td>656.50</td>
<td>615 250 412 722</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan 11</td>
<td>0.22</td>
<td>420,000</td>
<td>0.35</td>
<td>420.00</td>
<td>600 233 295 551</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan 12</td>
<td>0.29</td>
<td>563,500</td>
<td>0.05</td>
<td>549.00</td>
<td>405 250 381 700</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan 13</td>
<td>0.35</td>
<td>669,500</td>
<td>0.05</td>
<td>660.50</td>
<td>600 240 400 805</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan 14</td>
<td>0.28</td>
<td>607,000</td>
<td>0.05</td>
<td>499.50</td>
<td>460 180 284 520</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan 15</td>
<td>0.23</td>
<td>1,109,000</td>
<td>0.133</td>
<td>1,109.00</td>
<td>1,100 225 810 1,500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan 16</td>
<td>0.20</td>
<td>757,000</td>
<td>0.133</td>
<td>400.00</td>
<td>200 440 623 1,500</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan 17</td>
<td>0.24</td>
<td>456,000</td>
<td>0.065</td>
<td>261.50</td>
<td>415 185 305 498</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jan 18</td>
<td>0.24</td>
<td>456,000</td>
<td>0.065</td>
<td>261.50</td>
<td>415 185 305 498</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Remarks to particular Falls of Rain:
- Greatest height of Flood on 14th at half past 6 a.m.: 1.130 feet. Discharge per minute 722 cubic feet per acre. Discharge for the 3 hours preceding the half past 6 a.m. was 551 cubic feet per acre. Discharge for the half-hour preceding the half past 6 a.m. was 284 cubic feet per acre. Discharge for the half-hour after the half past 6 a.m. was 284 cubic feet per acre. Discharge for the 3 hours after the half past 6 a.m. was 284 cubic feet per acre. Discharge for the 1 hour after the half past 6 a.m. was 284 cubic feet per acre. Discharge for the 1 hour before the half past 6 a.m. was 284 cubic feet per acre. Discharge for the 1 hour before the half past 6 a.m. was 284 cubic feet per acre. Discharge for the 1 hour before the half past 6 a.m. was 284 cubic feet per acre. Discharge for the 1 hour before the half past 6 a.m. was 284 cubic feet per acre. Discharge for the 1 hour before the half past 6 a.m. was 284 cubic feet per acre.
Mr. G. Yeates remarked, that the greatest quantity of rain he had ever observed to fall in the twenty-four hours in Dublin was from one inch and a quarter to one inch and a half; this was on the 6th of August, 1849: on that occasion the rain commenced in the evening and ended in the morning; so that the time during which the rain actually fell was not more than ten or twelve hours; and almost all the public rain-gauges in the city indicated different quantities. With reference to the diagram registries, he himself had made use of a somewhat similar form of table for recording the variations of the barometer.

Mr. Mallett conceived that the importance of the subject, in its various relations, could not be too highly estimated, and as far as it became an engineering question he should remark that there had existed no positive knowledge upon the subject of the fall of rain, the discharge of rivers, and the amount of absorption and evaporation that took place in a given time. Up to a late period the only data afforded were the results of experiments made by Lawton under circumstances that do not apply. The plan of registering adopted in the case submitted to the Institution was a most beautiful one, and exhibited the results of the falls and discharges in a most clear and satisfactory manner. It certainly was new to him, but like Columbus's egg, in acknowledging the beauty and aptness of the illustration, they were forced to wonder that the plan had not been previously adopted. He thought that a series of experiments, similar to those made in the present instance, would be attended with the most important results; but he considered that the experiments should, in order to afford a true basis for calculation and conclusions, be carried out in extensive districts in which the slopes were moderate. The results of experiments made in mountainous districts, where on one side there might be a heavy fall, and on the other scarcely any, would of necessity be abnormal, and not afford a basis for accurate calculation.

The Chairman said he had been for a long time collecting information upon the subject treated of in the paper read that evening; and had lately received from the engineers in various parts of the country the results of observations which they had been enjoined to make with the greatest care. Until the present meeting he was not disposed to give credence to several statements that had been made with regard to the amount of water which had been represented to have been discharged from each acre in certain cases. The facts, however, that had been just brought forward, and the accuracy with which they had been obtained, led him to suppose
that the calculations, which he had supposed to be mistaken, had been well founded in the other instances. The results of the information afforded by the different engineers would form the subject of a future communication to the Institution.

[11th March, 1851.]

WILLIAM T. MULVANY, Esq., V.P., in the Chair.

On the Use of Screw-Pumps for Unwatering Works; by THOMAS J. MULVANY, Memb. Inst. C.E.I.

In the execution of some river drainage works, involving a large amount of pumping, my attention has lately been particularly directed to the consideration of the cheapest and most effective means of getting rid of the water, as, in all such cases, but more particularly in those where the work has to be executed in a considerable depth of water, the expense of pumping forms a very important item in the whole cost of the work. Having tried several varieties of pumping machines, I have been led to select the screw-pump as by far the cheapest and most effective means of raising water, which has come under my notice.

It is remarkable that, although this sort of machine is by no means unknown in many parts of England and Ireland, yet it is very seldom used by Engineers or Contractors on public works in this country at least; and it must be admitted that this fact might fairly be adduced as strong *prima facie* evidence that it has not been found to be an economic machine. With this impression upon my mind, I should have been very slow to advocate the use of the screw-pump on merely theoretic grounds, and I therefore think it right to premise that the statements I have to make as to the advantages of this machine are not the results either of calculation, or of mere experimental trials, but of observations made with regard to the regular working of pumps which have been now in use, under my own inspection, for several months.

The pump to which I will now call your attention was made some years ago for one of the contractors on the Shannon works. It consists of a hollow cylinder, 22 feet long and 2 feet diameter inside, the barrel being formed of pieces of lagging 2 inches thick and about 3 inches wide, secured together by hoops of iron keyed in the manner adopted in the hoops of ship masts. In the centre of the cylinder is a solid shaft of timber, octagon shaped, and 6 inches diameter, and the space between this centre shaft and the
APPENDIX B
KUCHLING (1889)
THE RELATION BETWEEN THE RAINFALL AND THE DISCHARGE OF SEWERS IN POPULOUS DISTRICTS.

By Emil Krichling, M. Am. Soc. C. E.

WITH DISCUSSION.

The most important question which arises in the construction of a sewerage system whose function is also the removal of the surface drainage, is with regard to the amount of storm water that will find its way into the sewers; and therefore a brief review of the modes of solution adopted in different places may be of interest. The proportion of rainfall contemplated to be admitted into the sewers varies within wide limits and depends largely upon the intensity and duration of the rain, the relative impermeability and slope of the surface, and the facility with which the storm waters can be diverted into suitable natural channels. either directly from the roofs and street gutters, or indirectly by means of storm-overflows from the sewers; the element of initial cost is also of vast significance in determining the limits, and, by being too narrowly considered, frequently leads to reductions of drainage capacity which must be supplemented a few years later by the construction of special relief conduits, whose expense is often much greater than the original amount saved, together with accrued interest.
Arguments to the effect that a municipality can better afford to pay the damages caused by occasional sewer overflows, than to pay the interest on the additional expense involved by the provision of a more ample original capacity, are very common, but they never allude to the financial value of the annoyance, the sanitary dangers or the depreciation of property which always follows where the sewerage is defective. To quote from a report of a distinguished American engineer in regard to the improvement of the sewerage of a certain large city: "Your engineer has been aware for several years of the importance of improving the sewerage system; and the frequent complaints of householders in certain localities of the city have caused the most careful investigations to be made from time to time. The flooding of basements and cellars depreciates the value of property and endangers the lives of those occupying the flooded dwellings. The offensiveness caused by sewage flooding drives away tenants. The damage done to property cannot be estimated by the soiled goods and injured walls, but must be measured by the permanent prejudice created against the locality." These words are likewise applicable to many other cities than the one referred to, and hence, before accepting the cogent arguments which appeal only to the economical instinct, a careful analysis of the problem with special reference to probable future conditions should be made, even though it be attended with many complications and difficulties.

In the absence of definite experiments of their own, our American engineers have generally based their practice of sewerage upon the results of certain gaugings of the storm-discharge of a few sewers in London, which were made many years ago, and while the subject was still in its infancy. For some unaccountable reason, the details of these gaugings have not been published; and in reading the meager descriptions which are sometimes quoted, one is often at a loss to know whether the percentage of the rainfall so discharged refers to the maximum, or to the average for the entire period. The familiar case of the Savoy street sewer, which is alleged to have given on June 20th, 1857, from an unusually heavy rainfall for London of 1 inch in one and one-fourth hours, a maximum flow of 0.34 cubic feet per acre per second, while the sewers of several adjacent and similar districts yielded only from one-fourth to one-seventh of this amount, is one of the few instances which are unambiguous in statement; but as some of these gaugings which were made by the London commission of 1857 have been shown to be
grosely inaccurate, the scientific value of the whole series is greatly impaired. Two different gaugings by Mr. Bazalgette, of the Savoy Street and Ratcliffe Highway sewers, for rainfalls of 2.90 inches in thirty-six hours and twenty-five hours respectively, refer specifically to the aggregate volume of rainfall discharged during said periods, and not to the maximum rate of discharge at any instant, the percentage so computed being given at 64.5 and 52.0 respectively. From these and a few other results, the distinguished engineers, Messrs. Bidder, Hawksley and Bazalgette, felt "warranted in concluding, as a rule of averages, that 0.25 inch of rainfall will not contribute more than 0.125 inch to the sewers, and that a fall of 0.40 inch will not yield more than 0.25 inch to the sewer." The two rainfalls mentioned are referred to as "the heaviest and most continued of the year 1857," and the two sewers are in the most densely populated portion of the city, the former "draining a locality strictly urban and of steep inclination," while the latter serves a "locality only moderately inclined." In the discussion of the Main Drainage of London, at a meeting of the Institution of Civil Engineers in 1853, Colonel William Haywood, the engineer to the sewer commissioners of that part of London called "The City," stated that in 1857, with a rainfall of 2.75 inches in thirty-six hours, the London Bridge sewer discharged 53 per cent. of said fall in the same time, and in 1858, with a rainfall of 0.24 inches in 1.5 hours, the same sewer discharged 74 per cent. of the fall; also that in June, 1858, "the Irongate sewer, which drains an area entirely paved and built over, discharged as much as 94.5 per cent. of a rain storm of 0.54 inch in five hours," while two months later the same sewer "discharged only 78 per cent. of a rainfall of 0.48 inch in 1.67 hours." Presumably these figures are the maximum percentages observed on these occasions, although nothing further than their extraordinary magnitude warrants such a conclusion, if the rainfall happened to be uniform in intensity. None of the reports, however, give any clue as to the character of the rainfall, the manner in which it was observed, or the location of the gauge, and hence the anomalies of measured percentage of discharge may be easily explained. Another familiar English authority on this subject is John Roe, who was for many years surveyor of the Holborn and Finsbury sewers, and who stated that during the continuance of a rainfall of 1.0 inch per hour, from 41 to 54 per cent. of the precipitation will reach the sewers, according to the amount of garden land or lawn upon the drainage area.
Upon the foregoing indefinite data principally, which may be found quoted more or less extensively in nearly every treatise on sewerage and in most of the elaborate reports, engineers have hitherto been content to rely; and it has thus come to be in some measure traditional that about 50 per cent. of the rainfall will run off from urban surfaces during the progress of the storm, while the remainder may follow at leisure. The greatest depth of rain for which provision should be made in planning sewers of American cities has likewise been commonly taken at one inch in an hour, "as its frequency, when compared with such falls as two and three inches in the same time, renders its consideration a more practical question; and the possibility of this rate of fall occurring for shorter intervals of time is so apparent from past observations on the sea-board and in the interior of the country, that we may regard it as a very proper maximum. It has been adopted as such in England, and, so far as we can learn, also in this country."

This view still continues to prevail extensively, and but few engineers have ventured to step outside of the beaten path. On this basis, also, the sewerage systems of two large cities, which have long enjoyed the distinction of being the best works of the kind in the United States, have been designed. It was considered that, inasmuch as the English gaugings indicated that only from 50 to 75 per cent. of the volume of rainfall ever entered the sewers at all, the provision of a capacity for discharging a depth of one-half inch of rain over the whole area uniformly in one hour was very liberal, and would afford an ample margin for the contingencies of future growth. The fact was, however, soon established, that as these cities grew in population and in the number of buildings and improved streets, the main sewers proved to be inadequate to carry off the storm-water due to the heavy showers of comparatively short duration, and to remedy the evils resulting from deluged cellars in the flooded districts, special storm-sewers have been constructed and additional ones are contemplated in both cities. In one of these cities an attempt was made to restrict the amount of water admitted into the sewers by throttling the street inlets in such manner as to compel a portion of the rainfall to escape by flow through the gutters; but while the sewers may thus be somewhat relieved, the annoyance and damage caused by flooded streets is substituted, and may ultimately become an intolerable nuisance.

The same experiences have likewise been observed in a large number of other cities, both in this country and abroad, wherever rain storms of great intensity for relatively short periods of time are more or less frequent, and where the principles above set forth have been adopted in computing the size of main sewers unprovided with storm outlets. It is conceded that in the majority of these cases, notably in the two cities named above, no fault can be found with the execution of the sewerage works, and hence the failure must be attributed to the assumptions with regard to rainfall upon which the calculations are based.

The writer was long since impressed with the fact that during heavy showers the volume of water discharged at the mouths of several large outlet sewers in the city of Rochester, N. Y., appeared to increase and diminish directly with the intensity of the rain at different stages, but that a certain length of time was required in each case after the termination of a brief and heavy down-pour before the corresponding flood showed itself at the outfall; these floods, moreover, seemed to last about as long as the said showers themselves, and the conclusion was therefore reached that there must be some definite relation between these fluctuations of discharge and the intensity of the rain, also between the magnitude of the drainage area and the time required for the floods to appear and subside. The observations referred to were made casually, during showers giving about one-half inch depth of fall in from twenty to thirty minutes, and the flood discharge would begin about ten minutes after the commencement of the rain or of the component showers; in like manner the maximum flow would quickly diminish, leaving only a very moderate after-flow to continue for some time after the rain had ceased. Now, since the total proportion of the rainfall which is carried off by the sewers is of little consequence, whereas the absolute maximum flow during any short period of time is manifestly the controlling factor in estimating the efficiency of a sewer; and since this maximum appears to be governed by the rate of the rainfall during such short period, therefore the error made in the assumptions mentioned above must be due to the use of average rates of rainfall for an unduly long period of time, and the neglect of the rates for such short periods within which the surface drainage waters from all portions of the area may be concentrated at the outfall. This error becomes still more apparent, when we reflect that for a uniform rate of precipitation, the concentrated discharge from a given surface will become a maximum for the condition
that the duration of such rate is equal to the time required for the water which falls upon the most distant points to reach the place of observation; or, in other words, that the entire area is contributing to said discharge; also from the fact that the heavy rainfalls in our latitudes rarely ever have uniform intensities for so long a period as one hour, and that few densely populated drainage areas contained within the municipal limits are so large as to require more than from fifteen to forty minutes for the full concentration of the storm-waters at some point of discharge. The conclusion is accordingly irresistible that the rates of rainfall adopted in computing the dimensions of a main sewer must correspond to the time required for the concentration of the drainage waters from the whole tributary area when small, or from so much thereof as will produce an absolute maximum discharge when the area is very large.

In justification of the prevalent practice, however, it is fair to say that the customary methods of observing the rainfall have hitherto prevented the development of the process just indicated, and have forced engineers to the use of average rates of precipitation deduced from periods of time scarcely ever less than one hour in duration. Thus the extensively and frequently quoted collection of rainfall statistics relating to Providence, R. I., which were made during forty years by Dr. A. Caswell, of that city, and published in the excellent report on the Sewerage of Providence, by J. Herbert Shedd, M. Am. Soc. C. E., in 1874, contains in the tables giving the record of three hundred and twenty-four storms, whose duration was carefully observed, only three cases where the rain lasted less than one hour, and in those instances the intensity of the rain did not exceed 1 inch per hour. The records were alleged to cover every appreciable rainfall which had occurred in said city during the interval between 1834 and 1874, and were regarded as perfectly reliable. A few of the storms observed in this long period were, however, so violent as to have induced Dr. Caswell to make special memoranda concerning them, and of this list only three entries specified the maximum rate of fall, as follows: September 28th, 1862, "nearly 2 inches of rain fell in the course of one hour;" August 16th, 1863, "very heavy thunder shower, 1.42 inches of rain falling in about twenty minutes;" June 17th, 1870, "storm lasting five hours, and giving 3.15 inches, nearly all of which fell between 12 M. and 1 P.M." Now, from these data the following condensed tables showing the relative frequency of rainfalls of different intensity per hour at Providence were con-
structed in 1874, and have since been widely copied by many distinguished writers in proof that great intensities of rainfall are exceedingly rare occurrences in our climate.

**TABLE A.**

<table>
<thead>
<tr>
<th>Rate of rainfall in inches, per hour</th>
<th>I. Number of storms during twenty-six years, from 1844 to 1870</th>
<th>II. Number of storms during fourteen years, from 1850 to 1864</th>
<th>III. Number of storms during forty years, from 1834 to 1874</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.23, or a little less</td>
<td>131</td>
<td>28</td>
<td>220</td>
</tr>
<tr>
<td>0.33, or a little less</td>
<td>14</td>
<td>9</td>
<td>27</td>
</tr>
<tr>
<td>0.40, or a little less</td>
<td>7</td>
<td>2</td>
<td>11</td>
</tr>
<tr>
<td>0.50, or a little less</td>
<td>1</td>
<td>10</td>
<td>17</td>
</tr>
<tr>
<td>0.60 to 0.62</td>
<td>8</td>
<td>5</td>
<td>13</td>
</tr>
<tr>
<td>0.67 to 0.75</td>
<td>3</td>
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<td>0</td>
</tr>
<tr>
<td>0.75 to 0.80</td>
<td>3</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>0.87 to 0.90</td>
<td>4</td>
<td>1</td>
<td>3</td>
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<tr>
<td>1.00 to 1.19</td>
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<td>3</td>
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<tr>
<td>1.20 to 1.40</td>
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<td>2</td>
<td>2</td>
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<td>1.50 to 1.69</td>
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<td>3</td>
<td>3</td>
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<tr>
<td>1.80 to 2.15</td>
<td>0</td>
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<td>2</td>
</tr>
<tr>
<td>Totals</td>
<td>185</td>
<td>130</td>
<td>224</td>
</tr>
</tbody>
</table>

The inferences that have been drawn from the foregoing table, even long after it was discovered that some of the main sewers in Providence and other cities had repeatedly proved to be incapable of removing the storm drainage, are as follows: 1. That out of three hundred and twenty-four large rainfalls in forty consecutive years, only three gave rates of 2 inches or more per hour; six gave rates of more than 1.5 inches per hour, and eleven gave rates of 1.0 inch or more per hour; and 2. that therefore a rainfall of 2.0 inches or more per hour can reasonably be expected to occur only once in thirteen years, while one of 1.5 inches per hour may happen once in about seven years, and one of 1.0 inch or more per hour about once every four years. Such inferences are, however, certainly unwarranted by the records derived from automatic rain gauges, which indicate that heavy rates for short periods of time are very common occurrences; and it is needless to say that even casual observation of the rainfall in the principal cities of the New England and Middle States prevents the application of the aforesaid inferences to intelligent sewerage work. To give reasonable assurance that special storm sewers will not be required in a district within a few years after the development of the suburban area has progressed, accordingly involves the adoption of a much higher rate of precipitation than one inch per hour when the
time required for the concentration of the surface discharge is much less than one hour, as is generally the case.

In order to guard against engorgements of the sewers, the exact duration and depth of water of both the entire storm and its variable showers must be known, as it frequently happens that during a rainfall which lasts one or more hours without intermission, the intensity of the precipitation will change greatly from time to time, thus causing the entire storm to resemble a series of hard showers connected together by intervals of mere drizzle. The average intensity of such a storm is of comparatively little consequence in dealing with the important problem of maximum flow in sewers, since it may be only one-third or one-fourth of the rate of fall which has occurred during some one of the component hard showers, and which would properly govern an engineer in fixing the dimensions of a sewer. Hence when the records of a certain rainfall do not show that the rate of precipitation has been practically uniform throughout the entire specified duration, the results obtained by gaugings of the corresponding sewer discharge will be utterly misleading, and their indiscriminate use may give rise to errors of design which may entail serious consequences.

The usual rainfall records merely give the total depth of water precipitated during certain regular intervals, such as 24, 12 or 6 hours; another very limited class of records indicate the approximate duration of the showers and the depth, the time in such cases being occasionally estimated when the attention of the observer at the beginning or end of the rain has been diverted by other matters; while a third class of records, which are, however, rarely kept outside of the most important meteorological stations, give the rates of fall at all times during the entire continuance of the rain. For sewer-discharge computations, the first of these three classes is almost entirely worthless, and their collection may accordingly be considered as a sheer waste of time; the second has much value, but must be used cautiously, especially when the rainfall happens to be of longer duration than about thirty minutes; the third class is by far the most valuable, and should be kept in every growing city where sewerage works exist. The best method of securing such data is from a number of self-recording rain gauges located in different parts of the municipal area, since a heavy shower may pass over a city in such manner as to deliver great quantities of water in one section, while another portion may receive only a light sprinkling. Cases of this kind
have frequently been observed in Rochester, N. Y., particularly during the past year, when opportunity for comparison was afforded by the records of the two gauges maintained by the writer, and those of the United States signal service and the city water works department, the four similar instruments being located at a distance of about one mile apart. It is greatly regretted that none of these gauges were of the self-recording type; but the great expense connected with the purchase and maintenance of automatic gauges precluded the writer from availing himself of such instruments, and as a consequence, it was found too late that only a few of the many records thus obtained could be used with reasonable certainty.

The numerous observations and measurements of the rainfall and contemporaneous flood discharge of a number of large sewers which are given in full detail below, will undoubtedly suffice to point out the necessity of securing the rainfall data needed in the consideration of the discharge from surfaces by means of automatic and self-registering devices, instead of trusting to the judgment of even the most careful and well-trained observers. Many other similar instances could be selected from the mass of material thus obtained in the course of the past year; and wherever such experiments are repeated, it is now urgently advised to make use of a large number of automatic rain gauges scattered about on the various separate drainage basins and located not more than about 1 500 feet apart, if only a single season be allowed for the collection of statistics. Most of the showers, moreover, occur during the night-time, and with the usual appliances the records for such cannot be secured unless two sets of observers are employed. Measures should also be taken to cause every Signal Service station in the cities of our country to be equipped with the best automatic gauges, and to have the results thus obtained carefully tabulated, in order that communities may be spared the great outlay of soon reconstructing sewers whose dimensions were based upon the deceptive records of rainfall as heretofore kept.

That the subject is already beginning to receive some recognition is shown by the recent publication of the data derived from the self-registering rain gauge at Washington, D. C., in response to a request from the superintendent of sewers of that city; and, as the information thus elicited serves to indicate the striking differences in the rate or intensity of the rainfall during the progress of a heavy storm, the following quotations from the circular of the Chief Signal Officer, issued
have frequently been observed in Rochester, N. Y., particularly during the past year, when opportunity for comparison was afforded by the records of the two gauges maintained by the writer, and those of the United States signal service and the city water works department, the four similar instruments being located at a distance of about one mile apart. It is greatly regretted that none of these gauges were of the self-recording type; but the great expense connected with the purchase and maintenance of automatic gauges precluded the writer from availing himself of such instruments, and as a consequence, it was found too late that only a few of the many records thus obtained could be used with reasonable certainty.

The numerous observations and measurements of the rainfall and contemporaneous flood discharge of a number of large sewers which are given in full detail below, will doubtless suffice to point out the necessity of securing the rainfall data needed in the consideration of the discharge from surfaces by means of automatic and self-registering devices, instead of trusting to the judgment of even the most careful and well-trained observers. Many other similar instances could be selected from the mass of material thus obtained in the course of the past year; and wherever such experiments are repeated, it is now urgently advised to make use of a large number of automatic rain gauges scattered about on the various separate drainage basins and located not more than about 1,500 feet apart, if only a single season be allowed for the collection of statistics. Most of the showers, moreover, occur during the night-time, and with the usual appliances the records for such cannot be secured unless two sets of observers are employed. Measures should also be taken to cause every Signal Service station in the cities of our country to be equipped with the best automatic gauges, and to have the results thus obtained carefully tabulated, in order that communities may be spared the great outlay of soon reconstructing sewers whose dimensions were based upon the deceptive records of rainfall as heretofore kept.

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about one year ago, may be of interest: "The observations (referring only to Washington) cover a period of seventeen years, from January, 1871, to December, 1887, in which time rain or melted snow has been recorded 1,543 times. The rainfalls exceeding 1 inch in depth number 192, of which 37 yielded 2 inches and upwards in depth for the entire duration of the storm. Precipitation exceeding 2 inches occurred four times in August and six times each in June, July, September and October, while they were rare in other months, and never happened in February. The heaviest fall from a single storm was 5.80 inches in nineteen hours on July 29th and 30th, 1878. Of equal and perhaps greater importance than the amount recorded during a single storm is the rate which falls in any single hour. Assuming that a less rate than 1 inch per hour is not especially important, examination was confined to those cases in which the rate was greater. Sixteen such cases have occurred, as shown in the following table:

**TABLE B.**

<table>
<thead>
<tr>
<th>Date</th>
<th>Amount in Inches</th>
<th>Time (Hours:Minutes)</th>
<th>Average Rate of Fall per hour in Inches</th>
<th>Maximum Rate of Fall per hour at any time in Inches</th>
</tr>
</thead>
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<tr>
<td>July 3rd, 1871</td>
<td>1.13</td>
<td>0:30</td>
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<td>2.26</td>
</tr>
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<td>0.84</td>
<td>0:30</td>
<td>2.49</td>
<td>2.49</td>
</tr>
<tr>
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<td>1:00</td>
<td>1.80</td>
<td>1.80</td>
</tr>
<tr>
<td>August 14th, 1875</td>
<td>1.29</td>
<td>1:00</td>
<td>1.30</td>
<td>1.30</td>
</tr>
<tr>
<td>August 28th, 1875</td>
<td>1.30</td>
<td>1:00</td>
<td>1.30</td>
<td>1.30</td>
</tr>
<tr>
<td>October 23rd, 1875</td>
<td>1.40</td>
<td>1:00</td>
<td>1.40</td>
<td>1.40</td>
</tr>
<tr>
<td>June 22d, 1877</td>
<td>1.08</td>
<td>1:00</td>
<td>1.08</td>
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</tr>
<tr>
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</tr>
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<td>1.12</td>
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</tr>
<tr>
<td>July 22d, 1885</td>
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<td>0:00</td>
<td>0.56</td>
<td>0.56</td>
</tr>
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<td>1.20</td>
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</tr>
<tr>
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</tr>
<tr>
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<td>1.80</td>
<td>1:00</td>
<td>1.80</td>
<td>1.80</td>
</tr>
</tbody>
</table>

Thus it appears that on two occasions 1.50 inches fell in one hour, and on one occasion 1.80 inches fell in the same time; but it will be seen that in all of these three cases, the maximum rate was far in excess of the average rate, the table showing that for short periods of time the rain then fell at rates of 8 inches and 6 inches per hour. The greatest
rate recorded, however, was the extraordinary one of 9.60 inches per hour, which fell on July 26th, 1885, but lasted only six minutes."

It may be remarked that for the purpose of determining the percentage of rainfall discharged from urban surfaces, the maximum rates of precipitation for values less than 1 inch are also of much importance, as the difference in such percentages for various rates of fall may thereby be ascertained. The exact duration of the maximum rates, moreover, should likewise be carefully tabulated, since in small districts the surface drainage may perhaps be fully concentrated before the heaviest portion of the shower has ended, in which event the corresponding rate must be used in computing the said percentage. Concerning the latter, it is reasonable to suppose that the same should vary in some degree both with the intensity of the rainfall and its duration, in all cases where this time is equal to or exceeds the period required for the drainage from the most distant points of the area to reach the point of observation in the sewer. Thus the percentage of discharge from a given district due to a uniform rate of 1.0 inch should be somewhat greater than that derived from a similar rate of 0.5 inch in the same length of time, and it should also increase with the duration of the rain when falling uniformly. So far as can be learned, no successful attempt has yet been made to compute these differences in rates of discharge, which are of so much importance in works of municipal sewerage or drainage, and it is a source of much regret to the writer that the lack of the said data prevented an examination of this nature in connection with his work during the past year.

In a paper on the maximum rates of rainfall by Desmond FitzGer­ald, M. Am. Soc. C. E., of the Boston water works, printed in Engineer­ing News of May 31st, 1884, the author refers to some of the heaviest rain­falls which occurred in the vicinity of Boston for a series of years. The most noteworthy of these is the storm of August 16th, 17th, 18th and 19th, 1879, which yielded a total depth of 6.23 inches, and of this amount 3.43 inches fell in 10+ hours on the 18th, the maximum being at the rate of about 1.0 inch per hour. The greatest intensity of rainfall observed was recorded July 20th, 1880, when 0.75 inch fell in thirteen minutes and 1.17 inches in thirty minutes, thus giving rates of 3.46 and 2.34 inches per hour respectively. Maximum intensities ranging from 1.5 to 2.0 inches per hour and lasting from eight to forty-five minutes seem to be usual attendants upon nearly all the storms whose diagrams are exhibited;
12 KUICHLING ON RAINFALL AND DISCHARGE OF SEWERS.

and as such durations will generally admit of the concentration of the drainage from small urban districts, the importance of providing for such falls will at once be recognized.

While the imperfect rain records for Rochester and the surrounding territory do not indicate that intensities of more than 1.5 inches per hour are of frequent occurrence, yet it is barely possible that greater rates would be discovered if self-registering gauges were used. It may, therefore, be of interest to note a few more of the heavy rains which have been observed at other places. At St. Louis, the following were recorded in 1884: 5.05 inches in 1½ hours, 5.22 inches in three hours, 6.17 inches in five hours, and 7.55 inches in twenty-nine hours; at Providence, R. I., 4.49 inches fell in one hour on August 6th, 1878, of which amount 3.50 inches came in thirty-six minutes, thus giving a rate of 5.85 inches per hour; and on August 13th, 1888, a storm occurred which gave 3.10 inches in eight hours, the maximum intensity being 3.75 inches per hour; at Leland, Miss., a rainfall of 11.5 inches occurred on August 13th, 1888, and was followed on the next day by one of 9 inches; at Waltham, Mass., 5.03 inches in three hours was recorded on August 21st, 1890; during a heavy storm over eastern Connecticut in August, 1874, a depth of 12 inches of water fell in forty-eight hours, and 5 inches in four hours; another great storm in Connecticut, and which extended over the New England States generally, occurred on October 3d and 4th, 1869, when the following depths were recorded at different localities within about thirty hours: 8.43 inches at Hartford, 8.44 inches at Colebrook, 9.37 inches at Middletown, and 12.35 inches at Canton, the greatest rate then recorded being 4 inches in two hours. The continuous rain in New England on February 10th to 14th, 1886, however, surpassed most of the previous ones in both intensity and duration, as well as disastrous consequences. From the accounts of this storm given by Professor W. Upton, in Volume VII of Science, and by the commission of engineers consisting of James B. Francis, Eliot C. Clarke and Clement Herschel, in their report to the city of Boston on the prevention of floods in the Valley of Stony Brook, it seems that the greatest fall occurred on the 12th, when 6.66 inches fell in twenty-four hours at New London and a total of 8.33 inches in fifty-eight and one-half hours; the same storm also yielded the following depths at other cities: 3.41 inches in forty-nine and one-half hours, with 2.99 inches in twenty-four hours at New York; 8.13 inches in seventy and one-half hours, with 5.65 in
twenty-four hours at Providence; 5.62 inches in fifty-five hours, with 4.45 in twenty-four hours at Boston, and 4.78 inches in seventy-seven hours, with 3.30 in twenty-four hours at Newburyport.

With regard to the extent of territory that may be covered by a heavy shower at any instant of time, observations show exceedingly wide variations. The storm area in our latitude is commonly several hundred miles in diameter, and occasionally exceeds two thousand miles in diameter, but the rain area is usually very much less, especially in the case of sharp thunder storms, where sometimes only a few square miles of the earth's surface are covered by the rain cloud. The writer has frequently noticed both the passage and the approach of such discharging clouds, and estimates that for a precipitation lasting about 15 minutes, the least area covered by the densest portion of the rain, as viewed from a distance, is about 4 miles in length and 1½ miles in width, thus giving an area of about 6 square miles. For shorter durations the area may be correspondingly smaller; but in general, the clouds which furnish rainfalls that greatly concern the capacity of sewers are considerably larger in extent than any single drainage area within ordinary municipal limits. On the other hand, to point out how great an area may be covered by a single storm, the following data given by James B. Francis, M. Am. Soc. C. E., and Professor W. Upton may be cited: For the storm of October 3d and 4th, 1869, the area on which 8 inches or more fell had a length of 65 miles, with an average width of 28 miles; the area on which 9 inches or more fell was about 50 miles long by 21 miles wide; the area on which 10 inches or more fell was about thirty-four miles long by 15 miles wide; and the area on which 11 inches or more fell had a length of about 20 miles, with an average width of 9 miles; similarly, for the storm of February 10th to 14th, 1886, the area on which over 8 inches of water fell was 750 square miles; that on which from 6 inches to 7 inches fell was 1,500 square miles; and that on which from 4 inches to 5 inches fell was 2,750 square miles. These areas were computed by plotting the observed depths of rain-fall at different places on the same day upon a map, and then drawing the contours showing the lines of equal depth; from the map thus prepared the required areas may then be easily determined.

Numerous other instances of severe rainfall might be cited; but the foregoing will doubtless suffice to exhibit not only the exceedingly variable character of the rate and duration of heavy storms, but also the necessity of carefully studying the data afforded by each particular
locality. As already stated, to secure a reliable estimate of the amount and rate of precipitation in Rochester, simultaneously with gaugings of the maximum flood-discharge of all the principal outlet sewers in the eastern portion of the city, two ordinary rain gauges of special design and carefully rated were procured and maintained for the past year by the writer, and placed in charge of thoroughly competent observers. Between the sites of these gauges at two points on the west and east sides of the city, the United States Signal Service station is located, while the rain gauge maintained by the Water Department of the city is situated about 1½ miles to the south. Four observations of the same rain at as many different points thus afforded the means of comparison with respect to intensity and quantity, while a careful timing and general observation of the rainfall, as viewed from the writer's office, secured many checks upon the records of those in charge of the gauges, and furnished clues to some of the numerous anomalies of discharge which were subsequently found. To obtain the depth, the water collected by three of these gauges was carefully weighed on delicate balances, and from this weight the corresponding volume was derived; hence, as the area of the 9-inch opening presented to the rain was accurately known, the resulting depth was easily found. At the Signal Service station, on the other hand, the depth is obtained by direct measurement with a graduated rod in a vessel considerably narrower than the gauge orifice. In this manner of gauging, the rate at all times during the progress of a storm cannot be secured, and with only one set of observers, many showers which happen during the night-time must necessarily be passed without specific record. The data thus derived can accordingly not be regarded as giving a perfectly correct view of the rainfall, but it is the only one which is here available.

In order to ascertain the frequency and intensity of heavy showers in this portion of the State, the writer was kindly supplied by the Chief Signal Officer with the statistics relating to rainfalls of more than 0.25 inch per hour collected at the stations in Rochester, Buffalo and Oswego from 1870 to 1888 inclusive; by the authorities of Cornell University with similar statistics relating to Ithaca from 1879 to 1888 inclusive; and by the chief engineer of the Rochester water works, with the records kept at Mount Hope Reservoir and Hemlock Lake from 1878 to 1888 inclusive. All of this information was arranged by months in tabular form; but to exhibit the essential facts in more con
venient shape, they have been summarized in the following table, which gives the recorded number of rains of different intensities during the cold and warm seasons, together with the deduced average number of times per year that a rain of the specified intensity has been observed at the several localities mentioned.

### TABLE C.

<table>
<thead>
<tr>
<th>Rate of Rainfall, inches per hour</th>
<th>Number</th>
<th>Total Number</th>
<th>Number of years' observation</th>
<th>Locality</th>
<th>Average Number per year</th>
</tr>
</thead>
<tbody>
<tr>
<td>From November 1 to April 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>From April 1 to November 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>0.20 to 0.50</td>
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<td>21</td>
<td>83</td>
<td>Rochester</td>
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<td>0.50 to 0.80</td>
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<td>43</td>
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<td>0.07</td>
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</table>

It should be remarked that the figures relating to the number of rains given in the above table are not worthy of much credit, owing to the absence of self-registering devices. For example, three of the six rains at Rochester, with rates of from two inches to three inches per

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* March, 1881, and November, 1885, lasting one hour twenty minutes and fifteen minutes respectively.
+ March, 1890, lasting only five minutes.
+ November, 1876, lasting twenty minutes.
hour, occurred in 1888 and were all obtained from the gauges main-
tained by the writer; hence, instead of such a rain—which caused seri-
ous overflows in nearly every sewer wherein observations were taken—
occurring only once in three years, as per table, it has in fact occurred
three times in one year, and twice on the same day! Furthermore, it is
the writer's firm conviction that at least two rains of similar intensity
occurred in 1887 in said city, no record of which is contained in said
table; and he has an equally strong conviction that if proper automatic
gauges had been in use at the two stations in said city, the list of heavy
showers would have been enormously increased during the course of
the past eighteen years. Doubtless the same might be said of the other
stations named, and hence the necessity for the use of better apparatus
in obtaining these important statistics. Wherever much reliance has
been placed upon such data as exhibited in Tables A and C, sewer en-
gorgements and cellar floodings have, curiously enough, soon followed
in the wake of the sewerage system, and large outlays are demanded for
the construction of relief conduits when direct storm overflows cannot
be provided.

The important question is with reference to the duration of the heavy
rainfalls which cause the capacity of the sewers to be exceeded and thus
give rise to damage in the adjacent cellars. An attempt to solve this
problem for the locality of Rochester with the data afforded by the
aforesaid local rainfall tables was made in the following manner; and
while the said data are doubtless deficient, yet they constitute the only
available means of reaching any reasonable conclusion in the premises:
The intensities of all these rainfalls, or the rates of precipitation in
inches per hour, were first computed from the given total depth and
time, and were then plotted as ordinates with the corresponding actual
durations of the rain as abscissae; a multitude of points on the diagram
was thus obtained, each one representing by its location a different rate
of fall for definite periods of time; and since the rains of relatively light
intensity were far more numerous than the heavy ones, these points
were much closer together in the lower portion of the diagram than in
the upper part. Now by connecting the successive highest points by
straight lines an irregular envelope, or line enclosing all of the re-
main ing points, will be obtained, and the ordinates of such envelope
will represent the probable maximum intensities of the rainfall in this
locality for the corresponding abscissae, or periods of time. A closer
examination of the diagram, however, shows that by omitting only a
very few of the highest points, the said envelope may conveniently be
represented by two straight lines which form an abrupt angle with each
other at a point corresponding to an intensity of about 0.87 inch and a
duration of about 1.0 hour. The line to the left of the point of inter-
section is quite sharply inclined, while that to the right is more nearly
horizontal, thus showing very clearly that the maximum uniform inten-
sity of the rainfall diminishes rapidly as its duration increases from a
few minutes to one hour, and that for rains of uniform intensity lasting
more than one hour the rate of diminution is comparatively slow. The
diagram is shown on Plate I.

To express the relations of the probable maximum intensity of the
local rainfall to its duration in mathematical terms, let \( t \) denote the
duration in minutes, and \( y \) denote the maximum intensity in inches
per hour; for periods less than one hour we will then have:

1) \[ y' = 3.73 - 0.0506t \]

while for periods longer than one hour and less than five
hours we have:

2) \[ y'' = 0.99 - 0.0021 \]

There is, however, more or less doubt about the accuracy of the data
in the said tables, and it may be argued with much force that under
the circumstances it would be preferable to consider averages rather
than extremes; also, that for application in Rochester, the data relating
to that city alone should be taken into account, especially since the same
appear to be more numerous and in much greater detail than the others.
Proceeding on this assumption, and grouping together such of the
twenty rainfalls of greatest intensity (ranging from 0.86 inches to 3.17
inches per hour) as have equal durations, we find that the averages for
periods less than one hour will be much less than the values obtained
from equation (1) above; and that the values derived from the following
substitutes for said equation will agree quite closely with such averages
for periods ranging from fifteen minutes to one hour:

3) \[ y' = 2.10 - 0.0205t \]

The results of the computations of \( y' \) from equations (1) and (3) for
different times \( t \), as well as the aforesaid averages of the observed
intensities, are arranged in the table on the next page.

The results thus found are generally much larger than those given
by the empirical formulas in common use, which predicate a rainfall of
not more than one inch per hour for drainage areas of all magnitudes.
KUICHLING ON RAINFALL AND DISCHARGE OF SEWERS.

TABLE D.

<table>
<thead>
<tr>
<th>Duration of Rain in Minutes (t)</th>
<th>Computed Intensity (y) from Eq. 1.</th>
<th>Computed Intensity (y) from Eq. 3.</th>
<th>Average of Observed Intensities</th>
<th>Number of such Observations</th>
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It should be distinctly stated that no great accuracy or general applicability is claimed for the above-mentioned Equation 3, which is of the most importance in the computation of dimensions for large sewers, nor for the data upon which said equation is based; it is merely an effort to utilize the only available records of the local rainfall in a rational manner, and to remove the subject of urban sewerage somewhat further from the realm of vague conjecture.

The foregoing method of ascertaining the probable maximum intensity of the rainfall of any particular locality was subsequently found to be practically identical with that adopted by Professor F. E. Nipher, of St. Louis, in the consideration of the rainfall of that city, a brief account of which is contained in Vol. IX of The American Engineer, Chicago, 1885. On plotting the records of the heaviest rains observed at St. Louis during a period of forty-seven years in the same manner as described above, the envelope enclosing all the points was found to be an equilateral hyperbola whose equation Professor Nipher considers to be: \( y = \frac{6}{t} \), the duration \( t \) being taken here in hours instead of minutes. This formula represents the statement that six inches of rain may fall in one hour, or that it may be spread over a greater number of hours; so that if the heaviest rain lasted uniformly for two hours, the maximum rate would then be \( y = 3 \) inches per hour, and 1.5 inches per hour for four hours, etc. An examination of the rainfall diagram for Rochester indicates that the envelope might possibly be an hyperbola, but not an equilateral one.

The famous hydrological studies of the basin of the River Seine, by Belgrand, caused the Parisian engineers to adopt a maximum rainfall of
1.77 inches in one hour as a governing factor in proportioning the modern sewerage works of the French capital. For the outlet sewers of large districts, it is assumed that about one-third of this depth is collected or concentrated during the progress of the rain and flows off rapidly, while the remainder is retarded in its flow and is lost by evaporation and absorption. To compare the foregoing maximum of 1.77 inches with a similar maximum that might be selected for some other city, such as Rochester, on the basis of the average annual rainfall, it may be mentioned that the average yearly rainfall of Paris is only about 23 inches, while that of Rochester is about 88.5 inches, so that the proportionate maximum rate for the latter city would be 2.60 inches per hour. In anticipation of comment, however, the writer would remark that the available observations of the local rainfall do not give so high an average, and that a rate of such magnitude would apply only to an extremely small district; or, in other words, would prevail for only a very short time, as fully pointed out above.

In referring to the Parisian practice in an excellent article in the *Annales des Ponts et Chaussées* for February, 1888, D. E. Mayer, of the French engineer corps, advises the adoption of Belgrand’s estimate, as given above, for like conditions of rainfall, climate and proportional amount of impervious surface, in other large cities, “deducting, however, in the computation of the tributary drainage area, all gardens and other cultivated or vacant land.” In smaller cities, “where the density of population is less, the proportion of such garden and other open land is greater than in the larger towns, and hence a much larger proportion of the rainfall would be lost before reaching the sewers, while the ordinary discharge per acre also diminishes with said density.” It is therefore probable that the proportional discharge adopted in Paris may likewise be applicable in minor municipalities when it becomes necessary to remove the surface drainage.

In German cities the recent practice appears to be similar in general theory to that just outlined for French towns, only that the maximum rainfall is usually taken at a much lower figure. Thus, for Berlin such rainfall is assumed at only seven-eighths of an inch per hour, one-third thereof reaching the sewers while the rain is falling. The mean annual rainfall is, however, only 22.8 inches, and it is claimed that a greater intensity than about 1.0 inch per hour was never observed there. Under such conditions a liberal provision appears to have been made.
For Vienna, the maximum rainfall was assumed at about 1.10 inches per hour, and three-eighths thereof is considered to reach the sewers in the same time. For Frankfort, Munich, Stuttgart and a number of other cities, variable allowances were made by the designers of the sewerage systems, but in all cases relief was assured by a sufficient number of storm overflows into the rivers. It is, however, quite noticeable, from the more recent reports and works on sewerage, that higher intensities of rainfall than heretofore considered admissible must be adopted for urban districts in which storm outlets are impracticable.

For example, when the sewerage of Stuttgart was designed in 1874, it was assumed that not more than 27.5 per cent. of any rainfall would reach the main intercepting sewer during the continuance of the storm; but the city engineer, in a work published in 1886, states that such an assumption is justifiable only when the drainage area includes large tracts of vacant land, and that his experience indicates a proportion ranging from 50 to 70 per cent. of the rainfall, according to the density of population and the condition of the street pavements. In his report on the sewerage of Königsberg, in 1883, the distinguished engineer, Wiebe, recommends that provision be made for admitting 50 per cent. of a maximum observed rainfall of 2.89 inches per hour into all sewers in the high level district of the city, where no storm overflows could be obtained; he also considers that very little of the water runs off from moderately inclined gardens, lawns and vacant land into the sewers during the first hour of the storm, and hence that only the area actually covered with buildings and pavements need be considered; for the city mentioned he estimates this relatively impervious area at from 42.3 to 54 per cent. of the total drainage area, according to the particular districts considered. To compensate for any contributions from the garden and land surface thus omitted, the roof and pavement surface is regarded as fully impervious, and taking this latter on an average at 50 per cent. of the whole, with one-half of the rainfall running off, it will be seen that by this procedure provision for about one-fourth of the maximum rate of precipitation is made in the sewers. For Mayence, the elaborate report of City Engineer Kreysig, published in 1879, contained similar statements to the effect that all sewers not provided with storm-outlets should be capable of removing the accumulated surface drainage due to the heaviest observed storms without becoming surcharged; and as the rainfall records of that city indicated that the depth
yielded by an extraordinary rain, such as occurs only once every few years, is about 1.60 inches per hour, Kreysig considered this limit as the lowest which could reasonably be adopted for said locality, and that at least 50 per cent. of such a fall would reach the sewers within one hour from the old and more densely populated districts. With respect to the general character of the surface in European cities, it may be remarked here that the local density of population in the older districts is often very great, the average being about 291 people per acre in Stuttgart and 162 in Mayence. Furthermore, that every street is well paved, and that very little of the surface is occupied by gardens or lawns, so that an estimate of only 50 per cent. of the rainfall is by no means large.

On the other hand, in support of the common theory of English engineers that heavy rainfalls of comparatively short duration do not yield such large percentages of discharge from urban surfaces, City Engineer Mank, of Dresden, published in the Deutsche Bauzeitung for 1884 the following observations: During a rain which lasted twenty-five minutes and fell at the rate of 1.96 inches per hour, the outlet sewer of a certain district of 326.7 acres in Dresden was noticed to be running completely full; the said district contained 49.1 acres of surface in the old portion of the city, which was almost entirely covered with roofs and pavements, 164.2 acres of closely built up territory in the new portion, and 118.4 acres of semi-suburban surface; considering the first-named component area as fully impervious, the second as giving 67 per cent. of impervious surface, and the third as giving only 34 per cent. of such surface, we would have an aggregate of 197.7 acres, or 60 per cent. of the whole, as practically impervious, and from which all of the rainfall should be delivered rapidly into the sewers; at the said rate of 1.96 inches per hour the water fell on said 197.7 acres at the rate of 387.5 cubic feet per second, while the outlet sewer was alleged to be discharging only 83.9 cubic feet per second; and hence it was inferred that a rain of the great intensity mentioned could yield to the sewers only about 21.7 per cent. of the precipitation on the estimated impervious surface, or only 18.1 per cent. of that on the total area. Exception to the aforesaid estimate of impervious surface might readily be taken as being excessive, and the percentage of discharge might thus easily be increased; the rain may also have been of much less intensity on the particular drainage area than it was at the location of the gauge, since in storms of such great violence the observations of the writer prove
KUICHLING ON RAINFALL AND DISCHARGE OF SEWERS.

conclusively that a difference of one-half or even one-fourth of a mile may make an enormous reduction of average intensity during so short a period as twenty-five minutes. The description given, furthermore, does not state specifically that the said discharge was the absolute maximum during the progress of the shower, and that the water did not rise higher than stated either before or after the time of the observation; but the principal exception that the writer makes to the foregoing is that the measurements of maximum flood-discharge by automatic gauges in a number of sewers in this city during the past year, firmly establish the fact that the percentage of discharge for such a shower is very much greater than as computed above; and in proof of the validity of this assertion, the following instance may here be cited:

Between 7:25 P.M. and 8 P.M. on May 9th, 1888, a violent thunderstorm, giving 0.767 inches of rain in thirty-five minutes, or a rate of 1.315 inches per hour, passed over the city from southwest to northeast. The writer had an opportunity to see the approach of the cloud from an elevated position, and to notice that one of his rain gauges lay directly in the track of the densest rain, while the other was near the edge of the shower. The latter yielded a depth of only 0.203 inches, and hence a rate of 0.848 inches per hour, its distance from the former being about two miles. Now the maximum discharge of the Clifford Street and Avenue B outlet sewer, whose drainage area was soon to be traversed by the heaviest portion of this rain, was found to be 73.3 cubic feet per second from a tributary total area of 356.9 acres, in which the average density of population does not exceed 20 per acre, and in which there are only a very few macadamized roadways, all the rest of the streets having simply natural earth roadways, with graded gutters and plank sidewalks; the vacant land, moreover, is largely of a gravelly character, so that little contributed therefrom to the sewers. About three-fourths of the said territory is well drained, nearly every street therein being provided with a sewer, while the remainder is to a great extent undeveloped agricultural land. The dwellings are principally small cottages, many of which are not yet connected with the sewers. According to the results obtained from estimates of the proportion of impervious surface on different classes of urban territory, the aggregate impervious surface should here be about 20 per cent. of the total area of 356.9 acres; but in view of the fact that such estimates predicted much better conditions of surface than are actually presented on this territory, the percentage
of impervious surface should be reduced to not more than 15, whence we would have about 53.6 acres of such surface from which all of the water would reach the sewers.

On this basis, and with a rate of rainfall of 1.315 inches per hour, the maximum sewer-discharge should be 70.5 cubic feet per second, which is a close agreement with the observed discharge of 73.3 cubic feet. To complete the data, it may be further remarked that the time required for the concentration of the surface drainage from the most distant points of the area to the point where the said maximum flow was registered is about thirty-four minutes, the average velocity of flow in the sewers being about 4.4 feet per second when nearly full, and their grades ranging from 1 in 47 to 1 in 910, with an average of 1 in 150. The said storm thus lasted long enough to cause the whole area to contribute to the flood discharge, which yielded a maximum of 15.6 per cent. of the rainfall on a territory which may fairly be classed as rural in comparison with the above described district of nearly equal magnitude in Dresden.

Under the circumstances, therefore, the writer is convinced that there must be some serious error in the aforesaid data relating to Dresden, from which a maximum discharge was deduced of only 13.1 per cent. of a storm having an intensity of 1.96 inches per hour, lasting twenty-five minutes, and falling upon a well-sewered urban area, of which 60 per cent. is regarded as impervious.

On the strength of these latter data Mr. Mauk built up a series of sewerage tables for the use of municipal engineers, which have been extensively copied into recent reports, notably those relating to Berlin and Wiesbaden. The professional eminence of the authors of these two reports is such as to have added greatly to the value and reliability of these tables, and it is only after abundant proof from the results of his own carefully conducted gaugings was afforded that the writer now ventured to call them in question. Their manifest error is attested by a number of other experiments similar to the one just described, and which will be given in detail below, also by the observations made in the past by English and American engineers. In view of these facts it is hardly worth while to consider further this method of computing the dimensions of outlets for districts of ordinary size.

The relation between the magnitude of a drainage area, the surface discharge, and the time required for the concentration of such discharge, has long been recognized in a general way, but does not appear to have
been very definitely expressed. In a paper by General O'Connell on "The Flood Discharge of Rivers," published in Volume 27 of Proc. Inst. C. E., the principle is stated as follows: "When water falls in the shape of rain on any solid surface, whence it afterward flows off, it forms its own drainage vehicle. It produces over that solid surface a certain depth of water with a certain superficial fall or slope toward an outlet; these two conditions, depth and surface slope, being necessary to secure flow. Should the solid surface be at all absorbent, the rain has to furnish the quantity of water necessary to saturate it. While the drainage vehicle is forming and having its capacity increased, the water is flowing off the surface less rapidly than it falls upon it, and, should the rain cease before it has completed its own drainage vehicle, the rate of discharge from the surface upon which it falls will never equal the rate at which the rain has fallen upon it. It is only when the time necessary for this preliminary operation of forming its own drainage vehicle has elapsed, that the water flows off from a surface as rapidly as it falls upon it. The time required increases with the linear distance between the upper and lower ends of the surface drained, and with the gentleness of its fall." When the drainage area is small, and has a comparatively impervious surface, the time necessary to establish equilibrium between precipitation and discharge—or to form what is termed "the drainage vehicle" in the foregoing—is relatively short; and as the distance that the rainfall has to travel over the surface before reaching some pipe or channel directly connected with the sewers is generally quite short in populous districts, it will be seen that the maximum rates of rainfall corresponding to such short times must be considered in estimating the volume of storm-water instead of average rates deduced from relatively long periods of time; also that the time will diminish in same proportion as the amount of impervious surface on the area increases. The latter, however, may be regarded in the case of cities as directly proportional to the density of the population up to a certain limit, after which it remains substantially constant, and hence the necessity of ascertaining the probable relation between these two elements before undertaking to compute dimensions for sewers in cities which are not yet fully developed.

Several attempts have been made to express the general principles above set forth in mathematical terms, but without much success from a scientific standpoint. The eminent English engineer, Hawksley, endeav-
ored to find a relation between the diameter of a circular conduit or sewer, the magnitude of the drainage area, the general slope of the surface, which was assumed to be parallel to the inclination of the sewer, and a rainfall of one inch per hour, on the assumption that half of the water would be discharged by the sewer within one hour. After many trials he finally invented the famous empirical formula which bears his name, and from which a few others have since been deduced. Foremost among these derivatives stands the expression proposed in 1880 by the distinguished Swiss engineer, Bürkli-Ziegler, but as it is merely Hawksley's formula in a somewhat different form, although admitting of a wider range of application by means of variable co-efficients, it cannot be characterized as a great improvement over the original. With reference to Hawksley's formula, Colonel J. W. Adams, Hon. M. Am. Soc. C. E., in his excellent work on Sewerage, remarks that: "While it gives ample capacity for the smaller dimensions of sewers and for limited areas, it did not prove so satisfactory in the larger," and he accordingly proposes a different empirical expression which, while "giving slightly less results in the smaller areas, give the increased dimensions in the larger that experience has pointed out as desirable in this locality."

The latest of such formulas is the one proposed by R. E. McMasth, M. Am. Soc. C. E., of St. Louis, in the Transactions of the American Society of Civil Engineers for 1887; it is modeled after that of Bürkli-Ziegler, but with a different empirical exponent, so that materially different results are obtained.

As it may be of interest to compare these four different formulas with each other, as well as with reliable observations, they have for convenience all been reduced to the same notation by the writer; and to make the first and third named applicable to other rates of rainfall than one inch per hour, this factor has been introduced in making the necessary transformation. Accordingly, with the following notation: 

- \( Q \) = maximum discharge of the outlet sewer in cubic feet per second; 
- \( v \) = maximum rate of rainfall in inches per hour, which is practically the same as if expressed in cubic feet per acre per second; 
- \( A \) = magnitude of the drainage area in acres, and 
- \( \phi \) = the sine of the general slope of the surface, or the quotient of the average fall divided by the average length, we will have the formulas given on the next page.

It may be remarked that the first and third expressions relate to ordinary urban conditions of surface, and are designed to apply best when
(1.) Hawksley ............... \( Q = 3.946 \frac{A r}{\sqrt{A r}} \)

(2.) Bürkli-Ziegler ... \( Q = (1.757 \text{ to } 1.216) \frac{A r^{11/3}}{\sqrt{A r}} \) Average 3.615

(3.) Adams ................... \( Q = 1.035 \frac{A r^{11/3}}{\sqrt{A r^{2/3}}} \)

(4.) McMath.............. \( Q = (1.234 \text{ to } 2.986) \frac{A r^{11/3}}{\sqrt{A r^{2/3}}} \) Average 2.488

\( r = 1.0; \) while in the second and fourth expressions the smaller co-efficients refer to suburban, and the larger to densely populated districts, the average referring to the same conditions assumed in the first and third. It will also be observed that in formulas 1 and 3, the ratio \( \frac{Q}{A r} \) will diminish as the intensity of the rainfall increases; but since the fundamental principles of hydraulics teach that the resistances to flow diminish rapidly with an increase of depth or volume, the writer is constrained to believe that there is a defect in these expressions which will manifest itself particularly in the case of relatively small drainage areas. For large districts, on the other hand, it may be conceded that the said ratios may perhaps not increase perceptibly within the range of usual intensities; nevertheless there is certainly no reason apparent why they should diminish when the rate of rainfall increases. The only justification for such a diminution lies in the circumstance that very heavy intensities usually last only a short time, and that consequently the whole area may not be contributing to the observed maximum discharge; but as this depends entirely upon the form, magnitude and slope of the territory, it is obvious that the said formulas must be used with great caution.

The safer method, in the writer's opinion, will be to estimate the probable future amount of impervious surface on the given area, either with reference to the density of population or in any other more reliable manner that may be devised, and to assume that all of the water which falls upon such surface will run off without loss; further, since the topography of the area is supposed to be known, the grades and length of the longest tributaries to the outlet sewer can readily be determined, as well as their approximate diameters, and thence also the velocities of flow therein; from these elements, the time required for the flood-waters to reach the outlet sewer from the most
distant points in the area can next be found, and when the relation between the probable maximum intensity of the rain and its corresponding duration are known, as exhibited in the preceding, the maximum rate of rainfall belonging to the time so found can then be deduced. By proceeding in this manner, it is believed that the least error will accrue in the results, and that the dimensions of a sewer so computed will be found adequate until the assumed amount of impervious surface or density of population has been exceeded.

It may be urged that the process indicated is nothing more than a crude approximation, and that some one of the various empirical formulas might as well have been applied; but to this it may be answered that the method is at all events intelligible and rational, besides being founded upon a somewhat better array of ascertained facts than is the case with the empirical formulas mentioned; it also has the merit of compelling the exercise of an engineer's judgment and discretion with respect to the future of particular localities of a city, or even of different portions of the same large drainage area, instead of dealing alike with all. Moreover, it rarely happens that the history and composition of such formulas become known to the majority of those who may be called upon to apply them, and hence a process in which every single component can be thoroughly scrutinized and amended to suit different circumstances will generally prove to be safer than the application of indefinite rules.

In the foregoing an attempt has been made to exhibit briefly the methods by which engineers of the widest repute and experience have been accustomed to estimate the greatest amount of rainfall, or storm-water, for which provision should be made in the sewers of populous districts, and a modification of these processes was suggested by the writer, insomuch as the data underlying such methods appear to be entirely inadequate to warrant unqualified acceptance. A careful analysis of all available records of the actual discharge of sewers in times of heavy or protracted rain, undertaken some years ago, revealed enormous incongruities or anomalies which could only be explained by erroneous premises, and which accordingly left the whole subject in a very unsettled condition from a scientific standpoint; hence, when circumstances recently enabled the writer to carry out a series of gaugings of the flood-discharge of a number of large sewers in the city of Rochester, N. Y., he made every possible effort to discover a more trustworthy
relation between the rainfall and the corresponding maximum flow from
the surface of a variety of urban districts. An account of these opera-
tions, together with the principal results of the computations involved,
is herewith submitted.

It may be remarked in the outset that while many of the difficulties
subsequently encountered were duly anticipated, yet the writer did not
appreciate fully the extreme delicacy with which the sewer-discharge
always responds to variations in the intensity of the rainfall until a
large number of observations had been collected and the tedious com-
putations completed; neither had he any reason to believe from existing
records that rainfalls of uniform intensity for considerable periods of
time were so comparatively rare. As a consequence, it was learned too
late that the most delicate and accurate self-registering rain gauges were
essential to the complete success of the experiments, and that a com-
paratively large number of such devices should be distributed over the
urban drainage area in order to detect all local variations in the rate
of precipitation. The results obtained from the gauges used in the work
are therefore susceptible of much criticism, yet it must be remembered
that not only were no precedents for the undertaking available, but also
that the appliances and methods of observation adopted were, on the
whole, much more trustworthy than in the case of similar published
experiments elsewhere.

The general plan of the work was as follows:

First.—The rainfall was observed at four different stations within the
city limits, located from three-fourths to one and one-half miles apart, as
already described. The observers were all urged to take the utmost
care in noting the exact duration of each heavy rain, and also to record
the duration and weight or depth of the water caught when the intensity
of the rain appeared to vary, thus dividing an irregular storm into its
component parts or showers. Independent records of relative intensity
and duration of such rains were also kept by the writer and his immedi-
ate assistants, who by constant practice and comparison with the meas-
ured results were soon enabled to form a tolerably accurate estimate of
the rate of precipitation from both the sound of the rain upon the roof
of the building and the appearance of the street gutters. The two
gauges maintained by the writer were placed about 35 feet above the sur-
face of the ground at the Municipal Gas Works and the Rochester
Bridge Works, where they were entirely free from the influence of sur-
rounding buildings, while the Signal Service gauge was located over 100 feet above the surface on the top of the tower of a lofty building in the center of the city, and the Water Works gauge was stationed on the bank of Mt. Hope Reservoir, which is on the crest of the range of hills in the southern districts. A comparison of the records of these four gauges afforded the means of determining whether the rainfall was evenly distributed over the whole territory.

Second.—The maximum flood-flow in the principal outlet sewers of the east side was obtained by means of self-recording gauges located both in the manholes and in the open channels or ditches which receive the discharge of such sewers in the suburbs. Owing to the nature of the liquid, delicate mechanical appliances could not be used for securing such measurements, as the solid matters in suspension would quickly obstruct the satisfactory operation of such instruments; and after much experimenting, the following simple device was found to give the best results. A thin strip of wood \( \frac{4}{4} \) inches wide and \( \frac{1}{4} \) inch thick was painted white and coated with a thin wash of whitening or pulverized chalk, and immediately sprinkled over with carefully sifted coarse sand; thus prepared, it was then inserted and fixed in a suitably grooved and narrow frame, which was securely fastened to the side walls of the manhole, or in an enclosed and locked framework secured to a post driven into the bottom of the ditch. The strip rested at its lower end upon an iron support firmly screwed to the frame, and its rise by flotation was prevented by fitting it snugly under the curbing or covering of the manhole, and by a locking device in the case of the open channels. Being exposed on both sides to the water or sewage, the thin wash of whitening was quickly softened by contact with the liquid, and the sand immediately dropped away from all places which had become immersed, but remained fixed on the surface of the strip above the flow-line. A sharply defined maximum flood-mark was thus obtained in almost every instance, and from the height of this mark, as well as from the previously ascertained relation of the iron support to the bottom of the sewer or channel, the greatest depth of the stream was definitely known.

The cross-sectional dimensions and slopes of the several sewers and channels so gauged were also carefully measured, so that all of the elements required for the computation of the maximum discharge were available. These gauges were examined immediately after every hard rainfall or shower, and the surfaces of the strips newly prepared as above.
described. It had been noticed previously by the writer that the discharge of certain large sewers in this city varied with the intensity of the rainfall, as well as with its duration, and numerous subsequent observations during the past year abundantly corroborated the view that comparatively slight variations in the rate of precipitation are quickly felt in the sewers, thus establishing the fact that the flood-marks must be attributed to the maximum intensities of the rain during relatively short periods of time, and not to the average intensity for the entire duration of the storm unless it be proven that the same was uniform throughout. The greatest care is therefore necessary in the observation of the rainfall and its variations, otherwise the deductions from the sewer-gaugings become highly deceptive. It may also be mentioned that the period of maximum discharge is usually equal to the duration of the corresponding maximum intensity of the rain, and that the flow is not in the form of a short, sharp-crested wave of momentary duration like that produced from a flushing tank. Thus, in a heavy shower lasting twenty minutes at uniform intensity, the maximum sewer discharge will continue for about the same length of time; and as this period is amply sufficient to flood any cellar or basement when a sewer is overcharged, the damage is then as great as if the duration had been longer. The importance of dealing with shorter periods of time than heretofore customary will accordingly be apparent.

Third.—To obtain the hydraulic slope in case of great depths of flow, or when the sewers are overcharged, the gauges were arranged in pairs, one being placed in each of two consecutive manholes in the same sewer; in the open channels they were likewise arranged in pairs, from 90 to 300 feet apart. In nearly every instance the manholes were located at the junctions of tributary sewers, and the gauges were placed in such manner as to be least affected by the inflow of water from such lateral pipes or conduits. It was also noticed that where the bottom of the main sewer was on a continuous grade, the depth of flow at the lower manhole or junction was always somewhat greater than at the upper one, a circumstance necessarily due to the inflow of the contributions from the intervening territory and the lower lateral sewers. Unfortunately, there were generally no intermediate manholes between such junctions, and hence the true hydraulic slope could not be definitely ascertained. It was, therefore, assumed that when the difference in depth of flow at each pair of manholes was not great, the surface slope would be par-
allel to the bottom slope, as determined by careful leveling, and that the back-water curve or "remoua" caused by accretions at the lower manhole would practically not extend up to the upper manhole, since the distance between the two always amounted to several hundred feet. On the other hand, when the sewers were overcharged, the true hydraulic slope was given directly by the gauge records, except in cases where the engorgements were so great as to cause the storm-water to rise above the tops of both manholes and to flood the adjacent low grounds; in such event no definite estimate of the flood-discharge is possible, and the nearest approximation is the maximum capacity of the sewer under various conditions of flow. Again, when the difference in depth of flow was so great as to warrant the inference that the back-water curve extended beyond the upper manhole, the hydraulic slope for computing the discharge at said manhole was estimated as being somewhat less than the bottom slope, and somewhat greater than the slope given by the gauge readings, in order to compensate for the inflow at the lower manhole.

Fourth.—The tributary drainage area behind each gauge was carefully computed from a topographical map of the eastern half of the city, upon which all existing sewers had been indicated, together with their direction of flow and relative size. The division lines between the various separate areas are necessarily approximations, as it would be impracticable to define exactly the actual limits in the great majority of cases, owing to the fact that few of the dividing ridges are distinctly marked or noticeable, and that in many instances the natural course of the drainage has been reversed in order to obtain an outfall into existing sewers. Much time and study were devoted to this matter, and it is believed that the errors so made are insignificant. Having thus learned the form and magnitude of each main area tributary to the sewers at the said gauges, computations of the time required for the storm-waters from the most distant points on the surface to reach each gauge by flowing through the tributary and main sewers were next made, and for this purpose it became necessary to make use of the records of the grades and dimensions of such sewers filed in the office of the City Surveyor. In a few cases the time actually required for the flood to make its appearance, or rather to attain its maximum height, at the gauge, was also observed and compared with the computed time, the result being that the former always exceeded the latter by several minutes, thus indicating that the water requires a certain length of time to reach
the sewers from the roofs, and more especially from the street surfaces. With the information thus obtained, the increased discharge at the lower gauges could readily be compared with the volume that would probably be delivered by the tributaries which entered the sewer at that point, and where the discrepancy between these two quantities was found to be large, the observations were rejected.

Fifth.—Thirty-one of the above described automatic flood gauges were located in eleven different main sewers and five open ditches or channels within the city limits, whereby an opportunity was afforded to observe the maximum discharge from twenty-four drainage areas, varying in magnitude from 25 acres to 600 acres, and presenting a great diversity of character with reference to quality of soil, density of population, class of roadways and extent of sewerage. Few of these districts, however, were similar in general character, and hence the final results cannot well be compared. For the sake of brevity the essential data relating to said districts have been arranged in Tables Nos. 1 and 2, hereto appended. It should be stated that in many cases the observations of the flood-flow were useless, as no reliable record of the rainfall was at hand; also that of the remaining observations only those which related to rainfall intensities of about 0.25 inch and upward per hour have thus far been worked out in detail for some of the most populous and best sewered districts. To a number of these computations, moreover, considerable uncertainty is attached, by reason of the irregularities in grade of the sewers; and hence in the following the results from only a few of the entire number of districts above mentioned can be submitted with confidence.

Sixth.—The discharge was computed with variable co-efficients deduced from Kutter's formula, and in a few instances where experiments were made, the computed velocities were found to agree closely with the observed average velocities between the gauges. Weir measurements would doubtless have been far more satisfactory; but as a matter of fact, none of the sewers were large enough to admit of using this method of gauging during heavy showers. In the distant suburbs it was possible to find a few places in the natural water-courses where a weir could have been applied without damaging the adjacent low lands by overflow or back-water; yet as the particular object of the gaugings was to obtain the maximum proportion of rainfall which finds its way into the sewers of the urban districts, the purpose would have been utterly defeated if large
tracts of agricultural land had been thus included in the tributary areas. No other process of gauging than by computing the discharge from some approved formula for the flow in a regular channel was therefore practicable.

Seventh.—The percentage of the rainfall which is discharged by the sewers during the period of maximum flow depends, as previously stated, chiefly upon the greatest intensity of the precipitation during the continuance of the storm; and if the average intensity were used in computing such percentages, an assortment of results would be obtained which are manifestly absurd from their enormous magnitude. For example, let us consider the case of the rain on June 2d, 1888, when a heavy shower lasting fifteen minutes was followed by thirty-five minutes of light rain of probably less than one-fourth of the previous intensity, and by a light drizzle lasting fifteen minutes more, after which the rain ended. The rain had thus a continuous duration of sixty-five minutes, and yielded a total depth of 0.152 inches in all parts of the city, its average intensity being 0.14 inches per hour. On this day, however, the Court and William Streets outlet sewer showed at the intersection of Alexander Street and University Avenue a maximum flow of 19.97 cubic feet per second from a drainage area of 132.96 acres; but at the said average intensity, the amount of water falling upon the entire area is only 18.6 cubic feet per second, and hence on this assumption the sewer would have discharged over 100 per cent. of the average rainfall, which is a palpable absurdity. On the other hand, it was estimated that during the fifteen minutes of the heavy shower at least two-thirds of the whole amount of water fell, which would give a maximum intensity of about 0.60 inch per hour, and hence also a precipitation of 53.2 cubic feet per second upon the whole drainage area; thereby reducing the percentage of the rainfall discharged by the outlet sewer to 37.5, which is doubtless not far from the truth, since about 38 per cent. of the surface may fairly be regarded as impervious. From the data collected by the writer, many other similar instances might be adduced, but it is believed that the foregoing one will suffice as an illustration of the necessity of the most minute and careful observation of the rainfall.

During the past year seventeen noteworthy rainfalls, ranging in maximum intensity from 0.24 inch to 3.20 inches per hour, have been more or less accurately observed in this city at the several stations mentioned, and for nearly all of these the corresponding maximum flow in the
sewers was obtained in the manner described. The complete analysis and description of these rains, as deduced from the various records by the writer, are given in full detail in Table No. 3; and a summary thereof exhibiting only the maximum intensities and durations of the component showers, along with some explanatory remarks, is given in Table No. 4, hereto appended. It should be mentioned that no great accuracy for the figures thus exhibited is claimed, as the rates of the component showers are generally estimates based upon numerous actual measurements; the total precipitation and the timings, however, can be accepted with much confidence in the majority of cases, as the errors are relatively small. The light rains and drizzles are always small fractional parts of the total fall in the class of storms here considered, and a close knowledge of their intensities can soon be acquired by practice; hence, when the duration of these estimated lighter rains is known, together with the total fall, the intensities of the heavy component showers can be reached with a fair degree of certainty. In this manner the rates for the heavy rains were estimated, as shown in Table No. 3; and in the absence of data from the most delicate self-registering rain gauges, they are submitted as the best which the writer can offer. No revision of them has been attempted in order to secure more harmonious correspondence with the sewer gaugings, and they are accordingly open to amendment, along with the computed percentages of discharge.

The storm-discharge from five different districts in this city may now be considered, the general description of these districts being as follows:

District I.—Discharge measured at gage No. 2 in the Clifford Street and Avenue B outlet sewer at the intersection of Avenue B and Harris Avenue. About one-half of the total irregular tributary drainage area of 356.94 acres has a dense population, averaging about 55 per acre, while the remainder is thinly settled and presents much agricultural land. The soil is generally a clayey loam, with gravel and muck in some places; its surface is slightly undulating, with no sharply defined natural watercourses. Nearly all of the existing streets are sewer'd and have graded earthen roadways and gutters, and only a very small proportion of the aggregate length of roadway is macadamized. The average grade of the sewer'd streets is about 1 in 150, and the sewer grades range from 1 in 47 to 1 in 910. Few large buildings or factories are found in the district. The outlet sewer appears to be in excellent order, and affords good facility for gauging at all depths of flow.

District IV.—Discharge measured at gage No. 8, in the North Avenue outlet sewer, near Syracuse Street. The tributary drainage area of 128.67 acres is generally well developed, and is in the form of an irregu-
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A lar strip 4800 feet long by about 1200 feet average width, beginning in the central part of the city and extending northerly. The average density of population may be estimated at about 92 per acre. The area contains many large business blocks and other buildings along North Avenue, but the rest of the territory is occupied chiefly by residences of medium size standing on moderately large lots. The soil is mainly a clayey loam, with muck in the lower districts, and the surface slopes gently to the north as far as the New York Central and Hudson River Railroad, after which it becomes very flat. All of the streets are sewered and graded, and about one-third of the aggregate length of roadway has been paved with asphalt, stone blocks, macadam and gravel, the macadam, however, predominating in extent; the remainder of the roadways are of common earth. The average grade of the streets is 1 in 180, and the sewer grades range from 1 in 50 to 1 in 630. The outlet sewer is of good rubble masonry, with a flat bottom, excavated in a horizontally stratified limestone rock; for small depths of flow no great accuracy can be expected from the gaugings.

District X.—Discharge measured at gauge No. 18, in the East Main Street sewer at the intersection of North Union Street. The tributary drainage area of 25.12 acres is well developed, and is in the form of a long and comparatively narrow strip traversed by East Main Street, along which many large business blocks and apartment houses have been built. There are, however, still many detached small residences on the area, with considerable garden space. The average density of population may be estimated at about forty per acre. The soil is a clayey loam, and its surface inclines moderately to the east. Almost every street is sewered and provided with a macadamized or gravel roadway, the average surface grade being 1 in 172, while the sewer grades vary from 1 in 70 to 1 in 330. The outlet sewer is of ordinary rubble masonry, with a flat bottom of rock, hard-pan or plank. For small depths of flow the gaugings are probably unreliable. From this district the greatest percentage of discharge may be expected, as it contains the largest proportion of impervious surface of any in the whole list.

District IX.—Discharge measured by gauge No. 19, in the Court and William Streets outlet sewer at the intersection of Alexander Street and University Avenue. The tributary drainage area of 132.96 acres is chiefly a well developed residential district, with a few large buildings and apartment houses. Most of the dwellings are large and stand rather close together on lots of medium size. The average density of population is about thirty-six per acre. Every street is sewered and graded, and the roadways are nearly all improved with macadam or gravel, but there is not a single first-class pavement in the whole district. The soil is generally of a loamy character, with some clay, gravel and muck in different portions; its surface is somewhat undulating, the prevailing slope, however, being towards the north and east; the average grade of the streets...
is about 1 in 151, and the sewer grades range from 1 in 54 to 1 in 400. The outlet sewer is of ordinary rubble masonry, with a flat bottom excavated in the limestone rock and trimmed to the slope; hence it is not well adapted to the gauging except when running at considerable depths. From the general character of this district, a relatively large percentage of discharge may be expected.

District XVII.—Discharge measured by gauge No. 30 in the Griffith Street sewer at the intersection of Broadway. The tributary drainage area of 92.27 acres is well sewered and developed, and the average density of population may be taken at about 35 per acre. Almost every street has been improved, about one-fifth of the aggregate length being paved with asphalt, one-fourth with stone blocks, and the remainder with macadam and gravel. The territory contains a number of large business blocks and apartment houses, but the greater portion is occupied by detached residences standing on lots of medium width. In one district of about twenty-five acres the lots are very deep and afford opportunity for additional streets. The soil is mainly a clayey loam; its surface slopes generally to the south, but in the aggregate one-half of the whole area is quite flat. The average grade of the streets is 1 in 240, and the sewer grades vary from 1 in 100 to 1 in 300. The sewerage is not of the best description, and the outlet has frequently been overcharged. It is reasonable to infer that the proportion of rainfall reaching the sewers is less than in the preceding district.

The foregoing five districts have been selected from the entire number available because they represent not only the best developed and most populous localities on the east side, but also the largest and most accessible outlet sewers. It is greatly regretted that much doubt with respect to the bottom slope of some of the other sewers, or of the same sewers at other gauges, prevents the utilization of the records obtained until such time as the nominal grades may be tested by numerous excavations; but the computations for several of these other districts show such wide differences and improbabilities as to greatly impair their value; furthermore, most of these districts include large tracts of agricultural or unimproved territory, and are therefore of minor interest in connection with the discharge from populous areas. The combined flow from a series of contiguous districts was measured a number of times by gauges 9 and 10, 11, 12 (a) and 13, and 12 (b); but as it was impracticable to determine each component separately, the gaugings must necessarily relate to large territories, and can serve only to check the computations made for the smaller areas in which the essential elements were known with reasonable certainty. For these reasons only a
part of all the records secured are now of use, while the remainder must be laid aside until the sewer grades can be properly verified hereafter.

Some of the details of the discharge computations for the aforesaid five districts are given in the appended tables, Nos. 5, 6, 7, 8 and 9, while Table No. 10 shows the computed percentages of the heaviest rainfall so discharged during the period of maximum flow. To exhibit these important results in more convenient and compact form, however, they are herewith submitted in the table on the following page.

It will be noticed that there are numerous discordances in this table, most of which can fairly be ascribed to imperfect estimates of the maximum intensity of the rainfall, while the remainder have doubtless arisen from errors made in observing the flood-marks left on the sewer gauges. Fortunately, however, the data are sufficiently numerous to admit of comparison; and by averaging the results obtained for similar durations of heavy rain it is probable that the majority of the discrepancies will be equalized, and that the mean values of the percentages of the rainfall so removed by the sewers will afford a clue to the general laws which govern such discharge. For facilitating the study of the problem, these average values for each of the districts were plotted as ordinates with the corresponding durations of the maximum rainfall as abscissas, thus obtaining a series of five somewhat irregular curves shown in Plate No. II.; and upon carefully examining these diagrams in conjunction with the explanatory remarks relating to both the rainfall and the sewer gaugings, the irregularities were corrected or equated by suitably fitting regular lines or curves to the various points obtained as stated. These new lines or curves accordingly represent a more or less close approximation to the actual relation of the rainfall to the flood flow in the sewers of populous districts; and while the numerical results thus reached may not be absolutely correct, the diagrams nevertheless point unmistakably to the following general conclusions:

First.—The percentage of the rainfall discharged from any given drainage area is nearly constant for rains of all considerable intensities and lasting equal periods of time. This circumstance can be attributed only to the fact that the amount of impervious surface on a definite drainage area was also practically constant during the time occupied by the experiments.
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### TABLE E.

Showing the computed percentages of the heaviest rainfall discharged from five different city districts by the respective outlet sewers during the period of maximum flow, also the average values of such percentages. Arranged with reference to duration of heaviest rainfall.

<table>
<thead>
<tr>
<th>Date</th>
<th>Maximum intensity of rainfall, inches per hour</th>
<th>Duration of rainfall at maximum intensity, minutes</th>
<th>Percentages of rainfall discharged</th>
</tr>
</thead>
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<tr>
<td></td>
<td></td>
<td>Dated * Min. acre</td>
<td>Dated 1 in. min. acre</td>
</tr>
<tr>
<td>December 10th, 1887</td>
<td>0.31*</td>
<td>60</td>
<td>13.3</td>
</tr>
<tr>
<td>September 10th, 1889</td>
<td>0.471</td>
<td>59</td>
<td>19.8</td>
</tr>
<tr>
<td>Averages</td>
<td></td>
<td>55</td>
<td>16.8</td>
</tr>
<tr>
<td>May 9th, 1889</td>
<td>1.31B1 (to 0.93)</td>
<td>38</td>
<td>10.4</td>
</tr>
<tr>
<td>April 26th, 1889</td>
<td>0.24*</td>
<td>30</td>
<td>10.4</td>
</tr>
<tr>
<td>May 12th, 1889</td>
<td>0.20*</td>
<td>20</td>
<td>11.0</td>
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<td></td>
<td>30</td>
<td>10.7</td>
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<tr>
<td>June 24th, 1888</td>
<td>0.22*</td>
<td>20</td>
<td>6.3</td>
</tr>
<tr>
<td>June 28th, 1888</td>
<td>0.80*</td>
<td>20</td>
<td>14.3</td>
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<tr>
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<td></td>
<td>20</td>
<td>10.3</td>
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<tr>
<td>June 30, 1888</td>
<td>0.40*</td>
<td>15</td>
<td>5.5</td>
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<tr>
<td>July 11th, 1888</td>
<td>0.70†</td>
<td>15</td>
<td>7.9</td>
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<tr>
<td>August 15th, 1888</td>
<td>1.61†</td>
<td>15</td>
<td>13.6</td>
</tr>
<tr>
<td>Averages</td>
<td></td>
<td>15</td>
<td>6.9</td>
</tr>
<tr>
<td>May 4th, 1889</td>
<td>0.3*</td>
<td>13</td>
<td>6.8</td>
</tr>
<tr>
<td>May 22nd, 1889</td>
<td>1.0*</td>
<td>13</td>
<td>10.6</td>
</tr>
<tr>
<td>August 4th, 1888</td>
<td>1.00†</td>
<td>12</td>
<td>4.8</td>
</tr>
<tr>
<td>August 26th, 1888</td>
<td>2.00†</td>
<td>14</td>
<td>4.0</td>
</tr>
<tr>
<td>Averages</td>
<td></td>
<td>13</td>
<td>6.9</td>
</tr>
<tr>
<td>July 18th, 1888</td>
<td>0.25†</td>
<td>10</td>
<td>7.6</td>
</tr>
<tr>
<td>August 15th, 1888</td>
<td>1.30†</td>
<td>10</td>
<td>5.5</td>
</tr>
<tr>
<td>Averages</td>
<td></td>
<td>10</td>
<td>6.5</td>
</tr>
</tbody>
</table>

| Probable time required for concentration of flow at gauges, minutes | 44 | 60 | 18 | 23 | 24 |

* Proceeded and followed by lighter rain.
† Light shower, followed by lighter rain.
‡ Heavy shower, proceeded by lighter rain.
§ Shower here run under beam: percentage is computed from maximum discharge without head previous to surcharge.
†† Figures obviously too high or low and rejected in deriving averages.
Second.—The said percentage varies directly with the degree of urban development of the district; or, in other words, with the amount of impervious surface therein. This fact is clearly shown by the large percentages derived from the relatively best developed District X, in contrast with the smaller percentages obtained from the relatively less improved Districts IX, IV and XVII, and to the still smaller results yielded by the least improved District I; and it also serves to account for the constancy of the percentage discharged from any particular district for rainfalls of the same duration.

Third.—The said percentage increases rapidly, and directly or uniformly, with the duration of the maximum intensity of the rainfall, until a period is reached which is equal to the time required for the concentration of the drainage waters from the entire tributary area at the point of observation; but if the rainfall continues at the same intensity for a longer period, the said percentage will continue to increase for the additional interval of time at a much smaller rate than previously. This circumstance is manifestly attributable to the fact that the permeable surface is gradually becoming saturated and is beginning to shed some of the water falling upon it; or, in other words, the proportion of impervious surface slowly increases with the duration of the rainfall.

Fourth.—The said percentage becomes larger when a moderate rain has immediately preceded a heavy shower, thereby partially saturating the permeable territory and correspondingly increasing the extent of impervious surface.

Fifth.—The sewer-discharge varies promptly with all appreciable fluctuations in the intensity of the rainfall, and thus constitutes an exceedingly sensitive index of the rain and its variations of intensity.

Sixth.—The diagrams also show that the time when the rate of increase in the said percentages of discharge changes abruptly from a high to a low figure, agrees closely with the computed lengths of time required for the concentration of the storm-waters from the whole tributary area; and hence the said percentages at such times may be taken as the proportion of impervious surface upon the respective areas. For example, the percentage curves for District IV and XVII are seen to be practically coincident, whence it might be inferred that the proportions of impervious surface are alike in both areas; as a fact, this conclusion is fully warranted by an examination of the two territories, which are separated by a large intermediate area.
The relation between the maximum sewer-discharge and the rainfall has thus been approximately established for five different districts in this city, and it has been seen that the flood-volume stands in direct proportion to the magnitude of the impervious surface on the drainage area, and to the intensity and duration of the rain; also that such flood-volume reaches practically a maximum when the precipitation continues uniformly for a sufficient length of time to secure the concentration of the storm-waters from all portions of the area. The element of time, therefore, enters twice into the determination of the flood-volume, and from the relation between duration and maximum intensity of the rainfall in this locality heretofore established, we may accordingly find the duration of that particular rainfall for which the sewer-discharge will become an absolute maximum. With the following notation: \( r \) = maximum intensity of the rainfall in inches per hour; \( t \) = duration in minutes of such intensity; \( Q \) = sewer discharge in cubic feet per second; \( A \) = magnitude of the entire drainage area in acres; \( m \) = proportion of impervious surface on said area, which is also substantially the same as the proportion of the rainfall discharged during the period of greatest flow; and with \( a, b \) and \( c \) = certain empirical constants, we will have:

1. \( Q = m A r \)
2. \( m = a t \)
3. \( r = b - c t \)
4. \( Q = A a t (b - c t) \)

and for the usual condition under which \( Q \) will become a maximum, we obtain:

5. \( A a (b - 2 c t) = 0 \), whence \( t = \frac{b}{2c} \)

But in the foregoing it was shown that the values of the empirical constants \( b \) and \( c \) were, for rainfalls lasting less than one hour in the locality of Rochester, \( b = 2.10 \) and \( c = 0.0205 \); hence the duration \( t \) of the heaviest rain which will cause \( Q \) to become an absolute maximum is: \( t = 51 \) minutes. This solution, however, is to be regarded simply as a crude approximation and valid only under certain circumstances; but it suffices to show that in drainage areas of moderate size, the heaviest discharge always occurs when the rain lasts long enough at its maximum intensity to enable all portions of the area to contribute to the flow. For large areas, on the other hand, a more elaborate analysis becomes necessary in order to find under what conditions the absolute
maximum discharge will occur, although the method of procedure above indicated will remain the same.

The present percentages of the rainfall discharged from the above-mentioned urban districts cannot, however, be regarded as permanent, since improvements are constantly being made by the construction of new buildings, pavements and sewers; hence not only is the proportion of impervious surface on these districts steadily growing, but the time required for the concentration of the storm-water in the outlet-sewers is also becoming materially reduced. In planning new sewers, therefore, it will be necessary to provide for the drainage from districts which, sooner or later, will be much better developed than any of those described above; and in the absence of more trustworthy data, we may be justified in concluding that the greatest percentages of discharge from such improved districts will continue to be practically equal to the percentages of impervious surface thereon, as was found to be the case with the five districts described.

The results of the flood gaugings are thus seen to be in general accord with the above described process, suggested by the writer for computing the necessary capacity of sewers on the "combined" system, and hence the method may be considered as reasonably accurate. To indicate what figures the writer adopted in computing the maximum flow in the several sections of the proposed east side trunk sewer, it may be stated that four different classes of territory were taken into account, as follows: Class I, with 50 persons per acre and 55 per cent. of impervious surface; Class II, with 40 persons per acre and 46 per cent. of impervious surface; Class III, with 25 persons per acre and 27 per cent. of impervious surface, and Class IV, with 15 persons per acre and 14 per cent. of impervious surface. It should also be stated that the central districts of the city, which will in the future undoubtedly afford a considerably higher percentage of impervious surface than has been assigned to Class I, are not embraced in the drainage area of said trunk sewer; furthermore, that in the estimate of these percentages a much better condition of the roadways and pavements has been assumed than now prevails.

In conclusion, it may be of interest to make an application of the above method and compare the result with the results given by the four formulas mentioned. For this purpose, let us consider District I, already described, with an area of 360 acres, which may, in the future,
be constituted as follows: 60 acres of Class I, 90 acres of Class II, 120 acres of Class III, and 90 acres of Class IV, thus giving 119.4 acres, or 33 per cent., of impervious surface, and a population of 10,900, or an average density of about thirty persons per acre; these conditions will doubtless be recognized as representing medium urban territory, to which any of the four formulas are directly applicable; furthermore, let it be assumed that the time required for the concentration of the storm-waters at the lower end of this district is: \( t = 44 \) minutes, and that the average surface slope of the streets is \( s = \frac{1}{10} \), with sewer grades ranging from 1 in 50 to 1 in 900, the main collector, however, having an average grade of 1 in 500. For the probable maximum intensity of the rainfall continuously during forty-four minutes we will have from Equation 3:
\[
r = 2.10 - 0.0205 t = 1.20 \text{ inches per hour (or cubic feet per acre per second),}
\]
and hence the volume of storm-water running off into the sewers from the 119.4 acres of impervious surface will at first be \( 119.4 \times 1.2 = 143.3 \) cubic feet per second; but as the rain lasts uniformly for so long a time, it may be considered that the permeable area has become partially saturated, and will toward the close of the rain be contributing about 15 per cent. of the precipitation thereon to the sewers, thus giving an additional quantity of \( 240 \times 1.2 \times 0.15 = 43.2 \) cubic feet per second, or a total storm-flow of 186.5 cubic feet per second. With a water-supply of 100 gallons per head per day, and one-half of this amount flowing off as sewage uniformly in six hours, the volume of sewage will be about 3.5 cubic feet per second; and hence the required capacity of the sewer at the lower end of said district should be, according to the writer's method: \( Q = 190 \) cubic feet per second.

On the other hand, we will obtain from Hawksley's formula, which predicates that \( r = 1.0 \), and that \( (s) \) is the sine of the slope of the outlet-surface, or in this case \( s = \frac{1}{10} \):

\[
(1) \quad Q = 3.946 \sqrt{A} = 68.97 \text{ cubic feet per second; or, if the formula be taken as above transcribed, with } r = 1.2 \text{ and } (s) \text{ denoting the sine of the average surface slope, or } s = \frac{1}{10}:
\]

\[
(1^*) \quad Q = 3.946 Ar \sqrt{\frac{A}{s}} = 106.66 \text{ cubic feet per second.}
\]

From Bürkli-Ziegler's transcribed formula, with the average value of the co-efficient \( = 3.515 \), \( r = 1.2 \) and \( s = \frac{1}{10} \), we find:

\[
(2) \quad Q = 3.515 Ar \sqrt{\frac{A}{s}} = 99.44 \text{ cubic feet per second.}
\]
The difficulty here is to determine what value shall be given to \( r \). Since Bürkli-Ziegler distinctly states that it should be the maximum which obtains during the continuance of the storm, and assigns to it for central Europe values ranging from 1.75 to 2.75. If an irregular rain lasting forty-four minutes be assumed, it is easy to see that a maximum intensity of 2.40 inches per hour might prevail for a few minutes, with lesser rates for the remainder of the time, and giving an average of 1.2 inches, as above computed; and with \( r = 2.40 \) we would have double the discharge just computed, or practically the same as the volume calculated by the writer's method.

From Colonel Adams' formula, as transcribed, and which originally predicates \( r = 1.0 \) and \( (\tau) \) as denoting the sine of the slope of the sewer, or in this case, \( s = \frac{\tau}{2} \), we have:

\begin{align*}
Q &= 1.035 \ A \ \frac{s^{\left(\frac{1}{2}\right)}}{A^{\frac{1}{2}}} = 83.23 \ \text{cubic feet per second}; \text{ whereas, if we use the values } r = 1.2 \text{ and } s = \frac{\tau}{2}, \text{ as before, we will obtain:} \\

Q &= 1.035 \ A \ \frac{s^{\left(\frac{1}{2}\right)}}{A^{\frac{1}{2}}} = 107.05 \ \text{cubic feet per second.}
\end{align*}

In like manner we will find from the transcribed MacMath formula, for the average co-efficient = 2.488 and \( r = 1.2 \), with \( s = \frac{\tau}{2} \):

\begin{align*}
(4) \quad Q &= 2.488 \ A \ \frac{s^{\left(\frac{1}{2}\right)}}{A^{\frac{1}{2}}} = 121.41 \ \text{cubic feet per second}; \text{ but if } (r) \text{ were taken at the value adopted for St. Louis, i. e., } r = 2.75, \text{ the discharge would be increased to about 278 cubic feet per second, or nearly fifty per cent. more than the volume computed by the writer's method.}
\end{align*}

In the choice of the several processes of estimating the required capacity of a combined sewer for a populous district, it must be remembered that with the heavy rains of frequent occurrence in this country, the proportioning of sewers by Hawksley's formula has usually resulted in floodings, and that an extensive experience with the other formulas has not yet been gained. The above investigations, moreover, show that larger quantities of storm-water run off from urban surfaces than is commonly supposed, and hence it is obvious that a more rational method of sewer computation is urgently demanded. Much room for improvement in this direction is still left, and it is sincerely hoped that the efforts of the writer will be amply supplemented by many valuable suggestions and experimental data which other members of the Society may generously contribute.
### TABLE No. 1.
**Showing Tributary Drainage Areas at the Different Flood-Gauges in Outlet Sewers.**

<table>
<thead>
<tr>
<th>Number of District</th>
<th>Designation of Sewer</th>
<th>Number of Lower Gauge</th>
<th>Tributary Area</th>
<th>Number of Upper Gauge</th>
<th>Tributary Area</th>
<th>Distance Between Gauges</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Acros.</td>
<td></td>
<td>Acros.</td>
<td>Feet</td>
<td></td>
</tr>
<tr>
<td>I.</td>
<td>Avenue B Outlet Sewer</td>
<td>1</td>
<td>393.71</td>
<td>2</td>
<td>256.04</td>
<td>397.5</td>
<td></td>
</tr>
<tr>
<td>II.</td>
<td>Lowell Street Outlet Sewer</td>
<td>3</td>
<td>43.90</td>
<td>4</td>
<td>37.52</td>
<td>338.0</td>
<td></td>
</tr>
<tr>
<td>III.</td>
<td>St. Joseph Street Outlet Sewer</td>
<td>5</td>
<td>108.54</td>
<td>6</td>
<td>102.78</td>
<td>473.1</td>
<td></td>
</tr>
<tr>
<td>IV.</td>
<td>North Avenue Outlet Sewer</td>
<td>7</td>
<td>216.56</td>
<td>8</td>
<td>130.07</td>
<td>776.9</td>
<td></td>
</tr>
<tr>
<td>V.</td>
<td>North Avenue Outlet Sewer Ditch</td>
<td>9</td>
<td>243.51</td>
<td>10</td>
<td>654.31</td>
<td>112.3</td>
<td></td>
</tr>
<tr>
<td>VI.</td>
<td>Goodman Street Outlet Sewer Ditch</td>
<td>11</td>
<td>690.62</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>VII (a)</td>
<td>Court and William Streets Outlet Sewer Ditch</td>
<td>13</td>
<td>341.31</td>
<td>120</td>
<td>341.31</td>
<td>1025</td>
<td>No. 12a changed to 12b, June 29, 1888.</td>
</tr>
<tr>
<td>VII (b)</td>
<td>Court and William Streets Outlet Sewer</td>
<td>15</td>
<td>177.35</td>
<td>17</td>
<td>167.00</td>
<td>553.6</td>
<td></td>
</tr>
<tr>
<td>VIII</td>
<td>Court and William Streets Outlet Sewer</td>
<td>17</td>
<td>179.15</td>
<td>19</td>
<td>179.15</td>
<td>816.6</td>
<td></td>
</tr>
<tr>
<td>IX.</td>
<td>Court and William Streets Outlet Sewer Ditch</td>
<td>17</td>
<td>25.78</td>
<td>18</td>
<td>25.78</td>
<td>749.2</td>
<td></td>
</tr>
<tr>
<td>X.</td>
<td>East Main Street Tributary Sewer</td>
<td>17</td>
<td>157.68</td>
<td>14</td>
<td>157.68</td>
<td>91.3</td>
<td>No. 15a changed May 14, 1888.</td>
</tr>
<tr>
<td>X (a)</td>
<td>Upton Park Outlet Sewer Ditch</td>
<td>15a</td>
<td>187.68</td>
<td>14</td>
<td>187.68</td>
<td>115.0</td>
<td></td>
</tr>
<tr>
<td>X (b)</td>
<td>Upton Park Outlet Sewer</td>
<td>15b</td>
<td>110.39</td>
<td>10</td>
<td>110.11</td>
<td>176.1</td>
<td></td>
</tr>
<tr>
<td>XII.</td>
<td>Upton Park Outlet Sewer</td>
<td>21</td>
<td>157.68</td>
<td>14</td>
<td>157.68</td>
<td>91.3</td>
<td></td>
</tr>
<tr>
<td>XIII.</td>
<td>East Avenue Outlet Sewer Ditch</td>
<td>23</td>
<td>211.90</td>
<td>22</td>
<td>211.90</td>
<td>1125</td>
<td>No. 23 was reset on May 14, 1888.</td>
</tr>
<tr>
<td>XIV (a)</td>
<td>Monroe Avenue and Nichols Park Outlet Sewer Ditch</td>
<td>25a</td>
<td>125.36</td>
<td>24</td>
<td>125.36</td>
<td>121.1</td>
<td></td>
</tr>
<tr>
<td>XIV (b)</td>
<td>Monroe Avenue and Nichols Park Outlet Sewer</td>
<td>25b</td>
<td>110.49</td>
<td>24</td>
<td>110.49</td>
<td>77.4</td>
<td></td>
</tr>
<tr>
<td>XV (a)</td>
<td>Pinnacle Avenue Outlet Sewer</td>
<td>27a</td>
<td>171.17</td>
<td>25</td>
<td>171.17</td>
<td>104.4</td>
<td></td>
</tr>
<tr>
<td>XV (b)</td>
<td>Pinnacle Avenue Outlet Sewer</td>
<td>27b</td>
<td>181.44</td>
<td>27</td>
<td>181.44</td>
<td>304.8</td>
<td></td>
</tr>
<tr>
<td>XVI.</td>
<td>Mt. Hope Avenue Outlet Sewer</td>
<td>29</td>
<td>215.77</td>
<td>28</td>
<td>215.77</td>
<td>590.5</td>
<td></td>
</tr>
<tr>
<td>XVII.</td>
<td>Griffith Street Outlet Sewer</td>
<td>31</td>
<td>183.58</td>
<td>30</td>
<td>183.58</td>
<td>627.8</td>
<td></td>
</tr>
</tbody>
</table>
TABLE No. 2.
SHOWING Characteristics of the Several Sewers at the Flood-Gauges.

<table>
<thead>
<tr>
<th>Drainage District</th>
<th>Flood Gauges</th>
<th>Actual Bottom at gauges</th>
<th>Width of sewer</th>
<th>Vertical size of Invert.</th>
<th>Shape of Roof</th>
<th>Total Height</th>
<th>Co-efficient of Roughness</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>I.</td>
<td>1 and 2</td>
<td>1 in 344</td>
<td>3.10</td>
<td>0.50</td>
<td>semicircle</td>
<td>5.50</td>
<td>0.015</td>
<td>Brick invert; remainder is rubble masonry in good condition. Grade appears to be regular.</td>
</tr>
<tr>
<td>II.</td>
<td>3 and 4</td>
<td>1/1 241</td>
<td>0.92</td>
<td>0.90</td>
<td>flat</td>
<td>2.00</td>
<td>0.020</td>
<td>Flat earth or plank bottom; rubble masonry in poor condition. Grade probably irregular.</td>
</tr>
<tr>
<td>III.</td>
<td>5 and 6</td>
<td>1 in 473</td>
<td>2.10</td>
<td>0.50</td>
<td>semicircle</td>
<td>5.50</td>
<td>0.017</td>
<td>Flat plank bottom; rubble masonry in average condition. Grade is probably somewhat irregular.</td>
</tr>
<tr>
<td>IV.</td>
<td>7 and 8</td>
<td>1/2 314</td>
<td>4.00</td>
<td>0.90</td>
<td>&quot;</td>
<td>6.00</td>
<td>0.017</td>
<td>Flat rocky bottom; rubble masonry in average condition. Grade probably somewhat irregular.</td>
</tr>
<tr>
<td>VII. (6)</td>
<td>15 (6)</td>
<td>1/2 000</td>
<td>3.25</td>
<td>1.00</td>
<td>&quot;</td>
<td>5.05</td>
<td>0.014</td>
<td>Brick invert; rubble masonry in good condition (new). Grade appears to be regular.</td>
</tr>
<tr>
<td>VIII.</td>
<td>16 and 17</td>
<td>1/1 85</td>
<td>2.44</td>
<td>0.90</td>
<td>&quot;</td>
<td>3.50</td>
<td>0.017</td>
<td>Flat rocky bottom; rubble masonry in average condition. Grade is somewhat irregular.</td>
</tr>
<tr>
<td>IX.</td>
<td>17 and 19</td>
<td>1/1 85</td>
<td>2.44</td>
<td>0.90</td>
<td>&quot;</td>
<td>3.50</td>
<td>0.017</td>
<td>Flat rocky bottom; rubble masonry in average condition. Grade is somewhat irregular.</td>
</tr>
<tr>
<td>X.</td>
<td>17 and 18</td>
<td>1/1 78</td>
<td>1.50</td>
<td>0.90</td>
<td>flat</td>
<td>2.20</td>
<td>0.017</td>
<td>Flat rock or hard earth bottom; rubble masonry in average condition. Grade probably irregular.</td>
</tr>
<tr>
<td>XII.</td>
<td>19 and 21</td>
<td>1/2 55</td>
<td>2.00</td>
<td>0.90</td>
<td>&quot;</td>
<td>2.50</td>
<td>0.017</td>
<td>Flat rock bottom; rubble masonry in average condition. Grade is somewhat irregular.</td>
</tr>
<tr>
<td>XIII.</td>
<td>22 and 23</td>
<td>1/248</td>
<td>1.83</td>
<td>&quot;</td>
<td>egg-shaped</td>
<td>2.50</td>
<td>0.015</td>
<td>Cement pipe, cracked in many places. Grade may be somewhat irregular.</td>
</tr>
<tr>
<td>XIV. (6)</td>
<td>24 and 25</td>
<td>1/187</td>
<td>2.00</td>
<td>0.90</td>
<td>flat</td>
<td>2.00</td>
<td>0.017</td>
<td>Flat rock bottom; rubble masonry in average condition. Grade is somewhat irregular.</td>
</tr>
<tr>
<td>XV. (6)</td>
<td>26 and 27</td>
<td>1/145</td>
<td>2.00</td>
<td>0.90</td>
<td>&quot;</td>
<td>2.00</td>
<td>0.017</td>
<td>Flat rock bottom; rubble masonry in average condition. Grade is somewhat irregular.</td>
</tr>
<tr>
<td>XVI.</td>
<td>28 and 29</td>
<td>1/102</td>
<td>2.10</td>
<td>0.90</td>
<td>&quot;</td>
<td>3.00</td>
<td>0.017</td>
<td>Flat rocky bottom; rubble masonry in average condition. Grade is somewhat irregular.</td>
</tr>
<tr>
<td>XVII.</td>
<td>20 and 21</td>
<td>1/208</td>
<td>2.00</td>
<td>0.90</td>
<td>&quot;</td>
<td>2.00</td>
<td>0.017</td>
<td>Flat rock bottom; rubble masonry in average condition. Grade is somewhat irregular.</td>
</tr>
</tbody>
</table>
TABLE No. 3.

Showing duration and analysis of intensity of the principal rainfalls at Rochester, N. Y., from October 1st, 1887, to October 1st, 1888.

<table>
<thead>
<tr>
<th>DATE</th>
<th>Rain Begin</th>
<th>Rain Ended</th>
<th>Duration</th>
<th>Depth Fallen</th>
<th>Assumed rates of Fall, in Inches, per hour.</th>
<th>Equivalent Depth Fallen, Inches.</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Hrs. Min.</td>
<td>Inches</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>December 10th, 1887......</td>
<td>5.30 a.m.</td>
<td>7.00 a.m.</td>
<td>1-30</td>
<td>0.104</td>
<td>0.002</td>
<td>0.104</td>
<td>Moderate rain.</td>
</tr>
<tr>
<td></td>
<td>7.00</td>
<td>9.00</td>
<td>2-00</td>
<td>0.025</td>
<td>0.035</td>
<td>0.070</td>
<td>Light rain.</td>
</tr>
<tr>
<td></td>
<td>9.00</td>
<td>10.00</td>
<td>1-00</td>
<td>0.310</td>
<td>0.030</td>
<td>0.300</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>10.00</td>
<td>11.00</td>
<td>1-00</td>
<td>0.441</td>
<td>0.040</td>
<td>0.050</td>
<td>Light rain.</td>
</tr>
<tr>
<td></td>
<td>11.30</td>
<td>12.00 P.M.</td>
<td>0-30</td>
<td>0.090</td>
<td>0.105</td>
<td>0.215</td>
<td>Drizzling rain.</td>
</tr>
<tr>
<td></td>
<td>12.00 P.M.</td>
<td></td>
<td>1-40</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>7-30</td>
<td>0.145</td>
<td></td>
<td>0.311</td>
<td></td>
</tr>
<tr>
<td>April 6th, 1888..........</td>
<td>8.40 a.m.</td>
<td>10.40 a.m.</td>
<td>1-30</td>
<td>0.230</td>
<td>0.073</td>
<td>0.105</td>
<td>Light rain.</td>
</tr>
<tr>
<td></td>
<td>9.40</td>
<td>10.40</td>
<td>0-50</td>
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</tr>
<tr>
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<td></td>
<td></td>
<td>2-00</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>9.40</td>
<td>10.40</td>
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</tr>
<tr>
<td>May 4th, 1888............</td>
<td>4.05 P.M.</td>
<td>4.13 P.M.</td>
<td>0-08</td>
<td>0.160</td>
<td>0.055</td>
<td>0.215</td>
<td>Moderate rain.</td>
</tr>
<tr>
<td></td>
<td>4.13</td>
<td>4.26</td>
<td>0-13</td>
<td>0.300</td>
<td>0.065</td>
<td>0.365</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>4.26</td>
<td>4.45</td>
<td>0-20</td>
<td>0.200</td>
<td>0.045</td>
<td>0.045</td>
<td>Drizzling rain.</td>
</tr>
<tr>
<td></td>
<td>4.45</td>
<td>5.00</td>
<td>0-14</td>
<td>0.326</td>
<td>0.055</td>
<td>0.381</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>5.00</td>
<td>5.18</td>
<td>0-18</td>
<td>0.200</td>
<td>0.060</td>
<td>0.260</td>
<td>Drizzling rain.</td>
</tr>
<tr>
<td></td>
<td>5.18</td>
<td>5.30</td>
<td>0-19</td>
<td>0.300</td>
<td>0.065</td>
<td>0.365</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>5.30</td>
<td>5.47</td>
<td>0-17</td>
<td>0.200</td>
<td>0.060</td>
<td>0.260</td>
<td>Drizzling rain.</td>
</tr>
<tr>
<td></td>
<td>5.47</td>
<td>6.05</td>
<td>0-18</td>
<td>0.200</td>
<td>0.060</td>
<td>0.260</td>
<td>Light rain.</td>
</tr>
<tr>
<td></td>
<td>6.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8.40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

273
<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Rainfall (in)</th>
<th>Discharge (gpm)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>May 9th</td>
<td>7:26</td>
<td>0.00</td>
<td>0.727</td>
<td>Heavy shower at Municipal Gas Works</td>
</tr>
<tr>
<td></td>
<td>7:30</td>
<td>0.00</td>
<td>1.314</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7:30</td>
<td>0.00</td>
<td>0.727</td>
<td></td>
</tr>
<tr>
<td>May 12th</td>
<td>2:40</td>
<td>0.00</td>
<td>0.293</td>
<td>Light shower at Rochester Bridge Works</td>
</tr>
<tr>
<td></td>
<td>3:00</td>
<td>0.00</td>
<td>0.348</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3:30</td>
<td>0.00</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>3:30</td>
<td>0.00</td>
<td>0.310</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4:00</td>
<td>0.00</td>
<td>0.210</td>
<td>Moderate rain</td>
</tr>
<tr>
<td></td>
<td>4:30</td>
<td>0.00</td>
<td>0.210</td>
<td></td>
</tr>
<tr>
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<td>5:00</td>
<td>0.00</td>
<td>0.210</td>
<td></td>
</tr>
<tr>
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<td>0.00</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>6:00</td>
<td>0.00</td>
<td>0.210</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6:30</td>
<td>0.00</td>
<td>0.210</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7:00</td>
<td>0.00</td>
<td>0.210</td>
<td>Moderate rain</td>
</tr>
</tbody>
</table>

* Not uniform over territory.

Average value, on west side depth was 0.25 inches, and on east side, 0.21 inches.
TABLE No. 3.—(Continued.)

<table>
<thead>
<tr>
<th>Date</th>
<th>Rain Began</th>
<th>Rain Ended</th>
<th>Duration</th>
<th>Depth Fallen</th>
<th>Assumed Rate of Fall, in Inches per Hour</th>
<th>Equivalent Depth Fallen, Inches</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>June 28th, 1888</td>
<td>7:00 A.M.</td>
<td>9:00 A.M.</td>
<td>2-40</td>
<td></td>
<td>0.100</td>
<td>0.200</td>
<td>Light rain.</td>
</tr>
<tr>
<td></td>
<td>9:00</td>
<td>9:15</td>
<td>0-15</td>
<td></td>
<td>0.150</td>
<td>0.300</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>9:15</td>
<td>9:25</td>
<td>0-10</td>
<td></td>
<td>0.200</td>
<td>0.400</td>
<td>Light rain.</td>
</tr>
<tr>
<td></td>
<td>9:25</td>
<td>10:00</td>
<td>0-35</td>
<td>1.000</td>
<td>0.250</td>
<td>0.500</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>10:00</td>
<td>10:20</td>
<td>0-20</td>
<td></td>
<td>0.280</td>
<td>0.560</td>
<td>Moderate rain.</td>
</tr>
<tr>
<td></td>
<td>10:20</td>
<td>10:25</td>
<td>0-25</td>
<td></td>
<td>0.200</td>
<td>0.400</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>10:25</td>
<td>10:50</td>
<td>0-10</td>
<td></td>
<td>0.150</td>
<td>0.300</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>10:50</td>
<td>11:00</td>
<td>0-15</td>
<td></td>
<td>0.100</td>
<td>0.200</td>
<td>Light rain.</td>
</tr>
<tr>
<td></td>
<td>5-25</td>
<td>5:25</td>
<td>0-20</td>
<td>1.20</td>
<td>0.150</td>
<td>0.300</td>
<td>Moderate rain.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Heavy rain.</td>
</tr>
<tr>
<td>July 11th, 1888</td>
<td>4:25 P.M.</td>
<td>5:15 P.M.</td>
<td>0-20</td>
<td>0.20</td>
<td>0.250</td>
<td>0.500</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>5:25</td>
<td>5:50</td>
<td>0-15</td>
<td></td>
<td>0.150</td>
<td>0.300</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td>July 18th, 1888</td>
<td>2:50 A.M.</td>
<td>3:45 A.M.</td>
<td>0-45</td>
<td>0.25</td>
<td>0.250</td>
<td>0.500</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>11:00</td>
<td>11:25</td>
<td>0-30</td>
<td></td>
<td>0.150</td>
<td>0.300</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>11:25</td>
<td>11:35</td>
<td>0-15</td>
<td></td>
<td>0.100</td>
<td>0.200</td>
<td>Moderate rain.</td>
</tr>
<tr>
<td></td>
<td>11:35</td>
<td>12:20 P.M.</td>
<td>0-45</td>
<td>0.650*</td>
<td>0.325</td>
<td>0.650</td>
<td>Moderate rain.</td>
</tr>
<tr>
<td></td>
<td>12:25 P.M.</td>
<td>1:30 P.M.</td>
<td>1-40</td>
<td></td>
<td>0.150</td>
<td>0.300</td>
<td>Moderate rain.</td>
</tr>
<tr>
<td>August 4th, 1888</td>
<td>1:00 A.M.</td>
<td>1:12 A.M.</td>
<td>0-12</td>
<td>0.451</td>
<td>1.000</td>
<td>2.000</td>
<td>Heavy rain.</td>
</tr>
<tr>
<td></td>
<td>1:12</td>
<td>2:15</td>
<td>1-31</td>
<td></td>
<td>0.100</td>
<td>0.200</td>
<td>Moderate rain.</td>
</tr>
<tr>
<td></td>
<td>2:15</td>
<td>2:50</td>
<td>0-05</td>
<td></td>
<td>0.050</td>
<td>0.100</td>
<td>Light rain.</td>
</tr>
<tr>
<td></td>
<td>2:50</td>
<td>3:40</td>
<td>0-25</td>
<td></td>
<td>0.100</td>
<td>0.200</td>
<td>Light rain.</td>
</tr>
<tr>
<td></td>
<td>3-40</td>
<td>4-00</td>
<td>1-50</td>
<td></td>
<td>0.150</td>
<td>0.300</td>
<td>Light rain.</td>
</tr>
<tr>
<td>Date</td>
<td>Time</td>
<td>Duration</td>
<td>Rainfall</td>
<td>Description</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>-----------------</td>
<td>-------------------</td>
<td>----------</td>
<td>----------</td>
<td>---------------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>August 15th, 1888</td>
<td>1:00 A.M.</td>
<td>1-00</td>
<td>0.84</td>
<td>Heavy rain</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1:15</td>
<td>0-15</td>
<td>0.464</td>
<td>Light rain</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1-30</td>
<td>0.121</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>August 15th, 1888</td>
<td>During night</td>
<td>unknown</td>
<td>0.525</td>
<td>Heavy rain</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0-10</td>
<td>0.272</td>
<td></td>
<td>On east side</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>August 17th, 1888</td>
<td>4:00 A.M.</td>
<td>0-20</td>
<td>0.175</td>
<td>Moderate rain</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5:00</td>
<td>0-25</td>
<td>0.363</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6:00</td>
<td>0-05</td>
<td>0.363</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7:00</td>
<td></td>
<td>0.121</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>August 20th, 1888</td>
<td>10:20 P.M.</td>
<td>0-15</td>
<td>2.900</td>
<td>Very heavy shower with hail</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>10:43</td>
<td>0-06</td>
<td>0.359</td>
<td>Moderate rain</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>September 15th, 1888</td>
<td>2:00 P.M.</td>
<td>0-25</td>
<td>0.431</td>
<td>Heavy rain</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2:50</td>
<td></td>
<td>0.475</td>
<td>Light rain</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Average value; on west side depth was 0.516 inches, and on east side, 0.494 inches.
† Not uniform over territory.
‡ About one-fifth was hail, thus leaving depth of water equivalent to a rate of about 2.50 inches per hour. On the east side the depth of water precipitated was 0.711 inches when weighed on the morning of August 27th.
TABLE No. 4.

Showing duration of the maximum intensity of the principal rainfalls at Rochester, N. Y., from October 1st, 1887, to October 1st, 1889.

<table>
<thead>
<tr>
<th>Date</th>
<th>Maximum Intensity</th>
<th>Duration of Rain of Maximum Intensity</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>December 10th, 1887</td>
<td>0.310</td>
<td>R. M. 1-00</td>
<td>Preceded by two hours' light rain and followed by ten minutes' light rain. Uniformly distributed.</td>
</tr>
<tr>
<td>April 3rd, 1888</td>
<td>0.249</td>
<td>0-30</td>
<td>Preceded and followed by light rain, lasting from one hour to one and a half hours. Uniformly distributed.</td>
</tr>
<tr>
<td>May 4th, 1888</td>
<td>0.300</td>
<td>0-13</td>
<td>Preceded by eight minutes moderate rain and followed by twenty minutes' drizzling rain. Uniformly distributed.</td>
</tr>
<tr>
<td>May 9th, 1888</td>
<td>1.315</td>
<td>0-35</td>
<td>Sudden heavy shower, followed by drizzle lasting one hour. Direction from S. W. to N. E. Not uniformly distributed. In extreme eastern district maximum intensity was only 0.33 inch per hour.</td>
</tr>
<tr>
<td>May 12th, 1888</td>
<td>0.300</td>
<td>0-30</td>
<td>Preceded by twenty minutes moderate rain, and followed by ten minutes of light rain. Not very uniform.</td>
</tr>
<tr>
<td>May 26th, 1888</td>
<td>1.000</td>
<td>0-13</td>
<td>Preceded by short, heavy shower, and fifteen minutes light rain, and followed by short, light rain and drizzle. Uniformly distributed.</td>
</tr>
<tr>
<td>June 1st, 1888</td>
<td>0.400</td>
<td>0-15</td>
<td>Sudden heavy shower, followed by fifty minutes light rain and drizzle. Was a little heavier on west side.</td>
</tr>
<tr>
<td>June 16th, 1888</td>
<td>1.000</td>
<td>0-30</td>
<td>Sudden heavy shower No. 1, followed by twenty-seven minutes of light rain and drizzle. Apparently uniform.</td>
</tr>
<tr>
<td>June 26th, 1888</td>
<td>2.500</td>
<td>0-20</td>
<td>Sudden heavy shower No. 2, followed by twenty-eight minutes of light rain. Apparently uniform.</td>
</tr>
<tr>
<td>June 26th, 1888</td>
<td>0.800</td>
<td>0-20</td>
<td>Preceded by sharp shower and forty-five minutes light and moderate rain, and followed by ten minutes' light rain, and then by sharp shower. Apparently evenly distributed.</td>
</tr>
<tr>
<td>July 11th, 1888</td>
<td>0.750</td>
<td>0-15</td>
<td>Preceded by twenty minutes moderate rain. Not very uniform.</td>
</tr>
<tr>
<td>July 14th, 1888</td>
<td>0.720</td>
<td>0-10</td>
<td>Preceded by twenty-five minutes' light rain, and followed by forty-five minutes' moderate rain. Much heavier on west than east side.</td>
</tr>
<tr>
<td>August 4th, 1888</td>
<td>1.000</td>
<td>0-12</td>
<td>Sudden heavy shower, followed by one hour's moderate rain with light showers. Probably not very uniform.</td>
</tr>
<tr>
<td>August 15th, 1888</td>
<td>1.015</td>
<td>0-15</td>
<td>Sudden heavy shower, followed by fifteen minutes moderate rain; three hours previously heavy rain. Not uniform.</td>
</tr>
<tr>
<td>August 17th, 1888</td>
<td>1.233</td>
<td>0-10</td>
<td>Sudden heavy shower, followed by one hour's moderate rain. Rained some hours previously. Apparently uniform.</td>
</tr>
<tr>
<td>August 25th, 1888</td>
<td>3.200</td>
<td>0-11</td>
<td>Sudden heavy shower with half, followed by eight minutes' moderate rain. Fairly uniformly distributed. Deducting half rate would be about 2.5 inches.</td>
</tr>
<tr>
<td>September 16th, 1888</td>
<td>0.470</td>
<td>0-60</td>
<td>Sudden heavy rain, followed by twenty-five minutes' light rain. Had rained some hours previously. Apparently uniform.</td>
</tr>
</tbody>
</table>
### TABLE No. 5.

Showing the maximum flow and percentage of rainfall discharged during the period of such flow in the Clifford Street and Avenue B outlet sewer at Gauge No. 2, at intersection of Avenue B and Harris Avenue.

Tributary drainage area = 356.9 acres. Time required for passage of storm-waters through longest line of sewers above said gauge = 34 minutes, which should be increased by about ten minutes for concentration in sewers. Co-efficient of roughness: $n = 0.015$.

<table>
<thead>
<tr>
<th>Date</th>
<th>Maximum Intensity of Rainfall</th>
<th>Duration of Rain at Maximum Intensity</th>
<th>Precipitation on Drainage Area</th>
<th>Cross-sectional Area of Flow in Sewer</th>
<th>Mean Hydraulic Radius (r) of such Area</th>
<th>Coefficient for Velocity ($c$)</th>
<th>Coefficient for Discharge ($q$)</th>
<th>Maximum Sewer Discharge ($Q$)</th>
<th>Percentage of Rainfall Discharged</th>
</tr>
</thead>
<tbody>
<tr>
<td>December 10th, 1897</td>
<td>0.31</td>
<td>60</td>
<td>110.64</td>
<td>3.56</td>
<td>0.772</td>
<td>1/343</td>
<td>33.24</td>
<td>13.25</td>
<td>13.8%</td>
</tr>
<tr>
<td>April 5th, 1897</td>
<td>0.34</td>
<td>30</td>
<td>115.57</td>
<td>2.44</td>
<td>0.569</td>
<td>1/300</td>
<td>16.05</td>
<td>9.8</td>
<td>10.8%</td>
</tr>
<tr>
<td>May 4th, 1897</td>
<td>0.51</td>
<td>13</td>
<td>167.06</td>
<td>2.13</td>
<td>0.532</td>
<td>1/288</td>
<td>16.75</td>
<td>7.31</td>
<td>6.8%</td>
</tr>
<tr>
<td>12th, 1897</td>
<td>1.31</td>
<td>35</td>
<td>167.07</td>
<td>3.77</td>
<td>0.653</td>
<td>1/242</td>
<td>31.15</td>
<td>11.8</td>
<td>11.0%</td>
</tr>
<tr>
<td>26th, 1897</td>
<td>1.00</td>
<td>13</td>
<td>196.48</td>
<td>6.04</td>
<td>0.522</td>
<td>1/180</td>
<td>38.15</td>
<td>8.6</td>
<td>8.6%</td>
</tr>
<tr>
<td>June 3rd, 1898</td>
<td>0.40</td>
<td>15</td>
<td>142.78</td>
<td>2.29</td>
<td>0.547</td>
<td>1/160</td>
<td>41.75</td>
<td>7.8</td>
<td>7.8%</td>
</tr>
<tr>
<td>24th, 1898</td>
<td>1.55</td>
<td>20</td>
<td>233.19</td>
<td>7.53</td>
<td>1.005</td>
<td>1/130</td>
<td>48.62</td>
<td>7.4</td>
<td>7.4%</td>
</tr>
<tr>
<td>24th, 1898</td>
<td>2.84</td>
<td>20</td>
<td>265.52</td>
<td>2.93</td>
<td>1.005</td>
<td>1/130</td>
<td>59.46</td>
<td>13.2</td>
<td>13.2%</td>
</tr>
<tr>
<td>26th, 1898</td>
<td>0.80</td>
<td>15</td>
<td>292.20</td>
<td>7.53</td>
<td>1.005</td>
<td>1/130</td>
<td>59.46</td>
<td>13.2</td>
<td>13.2%</td>
</tr>
<tr>
<td>July 11th, 1898</td>
<td>0.75</td>
<td>20</td>
<td>271.76</td>
<td>5.33</td>
<td>0.785</td>
<td>1/120</td>
<td>50.62</td>
<td>11.3</td>
<td>11.3%</td>
</tr>
<tr>
<td>20th, 1898</td>
<td>0.75</td>
<td>15</td>
<td>267.68</td>
<td>4.63</td>
<td>0.867</td>
<td>1/110</td>
<td>50.62</td>
<td>11.3</td>
<td>11.3%</td>
</tr>
<tr>
<td>August 4th, 1898</td>
<td>1.00</td>
<td>11</td>
<td>336.94</td>
<td>3.78</td>
<td>0.765</td>
<td>1/90</td>
<td>53.95</td>
<td>16.4</td>
<td>16.4%</td>
</tr>
<tr>
<td>13th, 1898</td>
<td>1.018</td>
<td>15</td>
<td>476.75</td>
<td>5.48</td>
<td>0.887</td>
<td>1/80</td>
<td>57.85</td>
<td>17.1</td>
<td>17.1%</td>
</tr>
<tr>
<td>17th, 1898</td>
<td>1.333</td>
<td>15</td>
<td>473.87</td>
<td>6.26</td>
<td>0.873</td>
<td>1/70</td>
<td>57.85</td>
<td>17.1</td>
<td>17.1%</td>
</tr>
<tr>
<td>20th, 1898</td>
<td>2.50</td>
<td>14</td>
<td>592.35</td>
<td>6.73</td>
<td>0.964</td>
<td>1/60</td>
<td>59.85</td>
<td>18.3</td>
<td>18.3%</td>
</tr>
<tr>
<td>September 10th, 1898</td>
<td>0.47</td>
<td>50</td>
<td>117.75</td>
<td>6.41</td>
<td>0.948</td>
<td>1/50</td>
<td>59.85</td>
<td>18.3</td>
<td>18.3%</td>
</tr>
</tbody>
</table>

* Preceded and followed by lighter rain.
1 Sudden shower followed by lighter rain.
11 Heavy shower preceded by lighter rain.
111 Intensity roughly estimated, deducting half.
TABLE No. 6.

Showing the maximum flow and percentage of rainfall discharged during the period of such flow in the North Avenue sewer at Gauge No. 8, at the angle in North Avenue near Syracuse Street.

Tributary drainage area = 128.67 acres. Time required for passage of storm-waters through longest line of sewers above said gauge = 18 minutes, which should be increased by about eight minutes for concentration in sewers. Co-efficient of roughness: \( n = 0.017 \).

<table>
<thead>
<tr>
<th>Date</th>
<th>Maximum Intensity of Rainfall</th>
<th>Duration of Rain at Maximum Intensity</th>
<th>Precipitation on Drainage Area</th>
<th>Cross-sectional Area of Flow in Sewer</th>
<th>Mean Hydraulic Radius of such Area</th>
<th>Co-efficient for Velocity set</th>
<th>Maximum Sewer Discharge. (cubic feet per second)</th>
<th>Percentage of Rainfall Discharged.</th>
</tr>
</thead>
<tbody>
<tr>
<td>December 19th, 1887...</td>
<td>0.31</td>
<td>60</td>
<td>23.90</td>
<td>3.68</td>
<td>0.630</td>
<td>1.040</td>
<td>77.70</td>
<td>9.27</td>
</tr>
<tr>
<td>April 5th, 1889...</td>
<td>0.21</td>
<td>50</td>
<td>19.98</td>
<td>2.43</td>
<td>0.492</td>
<td>1.005</td>
<td>73.00</td>
<td>4.91</td>
</tr>
<tr>
<td>May 4th, 1889...</td>
<td>0.20</td>
<td>35</td>
<td>14.00</td>
<td>9.64</td>
<td>0.240</td>
<td>1.095</td>
<td>37.00</td>
<td>5.25</td>
</tr>
<tr>
<td>25th, 1890...</td>
<td>1.00</td>
<td>20</td>
<td>10.56</td>
<td>1.66</td>
<td>0.318</td>
<td>1.000</td>
<td>74.00</td>
<td>4.80</td>
</tr>
<tr>
<td>15th, 1889...</td>
<td>6.00</td>
<td>12</td>
<td>6.00</td>
<td>1.50</td>
<td>0.456</td>
<td>1.050</td>
<td>43.41</td>
<td>25.21</td>
</tr>
<tr>
<td>June 25th, 1889...</td>
<td>0.40</td>
<td>12</td>
<td>34.00</td>
<td>3.05</td>
<td>0.845</td>
<td>1.050</td>
<td>71.90</td>
<td>4.97</td>
</tr>
<tr>
<td>July 2nd, 1890...</td>
<td>2.02</td>
<td>20</td>
<td>132.67</td>
<td>12.50</td>
<td>1.008</td>
<td>1.000</td>
<td>71.45</td>
<td>4.72</td>
</tr>
<tr>
<td>29th, 1890...</td>
<td>1.00</td>
<td>20</td>
<td>102.56</td>
<td>1.83</td>
<td>0.047</td>
<td>1.000</td>
<td>65.06</td>
<td>23.24</td>
</tr>
<tr>
<td>14th, 1890...</td>
<td>0.75</td>
<td>15</td>
<td>27.41</td>
<td>0.39</td>
<td>0.768</td>
<td>1.000</td>
<td>92.05</td>
<td>12.44</td>
</tr>
<tr>
<td>August 10th, 1890...</td>
<td>1.50</td>
<td>10</td>
<td>65.50</td>
<td>3.24</td>
<td>0.701</td>
<td>1.000</td>
<td>70.80</td>
<td>11.78</td>
</tr>
<tr>
<td>17th, 1890...</td>
<td>1.00</td>
<td>12</td>
<td>171.00</td>
<td>2.38</td>
<td>0.725</td>
<td>1.000</td>
<td>80.69</td>
<td>11.63</td>
</tr>
<tr>
<td>25th, 1890...</td>
<td>2.00</td>
<td>14</td>
<td>321.78</td>
<td>11.20</td>
<td>1.107</td>
<td>1.700</td>
<td>86.90</td>
<td>14.94</td>
</tr>
<tr>
<td>September 10th, 1890...</td>
<td>0.47</td>
<td>80</td>
<td>60.49</td>
<td>6.32</td>
<td>0.937</td>
<td>1.700</td>
<td>85.00</td>
<td>22.12</td>
</tr>
</tbody>
</table>

* Preceded and followed by lighter rain.
† Sudden shower followed by lighter rain.
‡ Heavy shower preceded by lighter rain.
§ Intensity roughly estimated.
TABLE No. 7.

Showing the maximum flow and percentage of rainfall discharged during the period of such flow, in the East Main Street sewer at Gauge No. 18, at intersection of East Main and Union Streets. Tributary drainage area = 25.12 acres.

Time required for passage of storm-water through longest line of sewers above said gauge = 10 minutes, which should be increased by about six minutes for concentration in sewers. Co-efficient of roughness: \( n = 0.017 \).

<table>
<thead>
<tr>
<th>DATE</th>
<th>Maximum Intensity of Rainfall</th>
<th>Duration of Rain at Maximum Intensity</th>
<th>Precipitation on Drainage Area</th>
<th>Cross-sectional Area of Flow in Sewer</th>
<th>Mean Hydraulic Radius (( r )) of such Area</th>
<th>Adopted Slope of Water Surface (( s ))</th>
<th>Coefficient of Velocity (( c ))</th>
<th>Maximum Sewer Discharge (( Q ))</th>
<th>Percentage of Rainfall Discharged</th>
<th>Cubic Feet per Second</th>
</tr>
</thead>
<tbody>
<tr>
<td>December 16th, 1887</td>
<td>0.31</td>
<td>60</td>
<td>7.80</td>
<td>1.01</td>
<td>0.354</td>
<td>1/30</td>
<td>67.5</td>
<td>4.54</td>
<td>56.2*</td>
<td>7.5</td>
</tr>
<tr>
<td>April 3rd, 1888</td>
<td>0.26</td>
<td>30</td>
<td>0.83</td>
<td>1.05</td>
<td>0.356</td>
<td>&quot;</td>
<td>64.2</td>
<td>4.49</td>
<td>64.4*</td>
<td>6.0</td>
</tr>
<tr>
<td>May 6th, 1888</td>
<td>0.30</td>
<td>13</td>
<td>7.54</td>
<td>1.39</td>
<td>0.469</td>
<td>&quot;</td>
<td>73.3</td>
<td>6.15</td>
<td>52.1*</td>
<td>7.5</td>
</tr>
<tr>
<td>12th, 1888</td>
<td>0.30</td>
<td>30</td>
<td>18.64</td>
<td>1.79</td>
<td>0.557</td>
<td>&quot;</td>
<td>63.8</td>
<td>2.56</td>
<td>35.8*</td>
<td>9.0</td>
</tr>
<tr>
<td>26th, 1888</td>
<td>1.00</td>
<td>13</td>
<td>25.12</td>
<td>2.10</td>
<td>0.419</td>
<td>&quot;</td>
<td>79.2</td>
<td>7.04</td>
<td>41.5*</td>
<td>10.0</td>
</tr>
<tr>
<td>June 23rd, 1888</td>
<td>0.49</td>
<td>13</td>
<td>10.05</td>
<td>2.39</td>
<td>0.450</td>
<td>&quot;</td>
<td>76.2</td>
<td>21.04</td>
<td>20.0*</td>
<td>11.5</td>
</tr>
<tr>
<td>24th, 1888</td>
<td>0.64</td>
<td>20</td>
<td>7.90</td>
<td>2.63</td>
<td>0.500</td>
<td>&quot;</td>
<td>78.9</td>
<td>7.06</td>
<td>35.2*</td>
<td>13.0</td>
</tr>
<tr>
<td>25th, 1888</td>
<td>0.80</td>
<td>10</td>
<td>20.19</td>
<td>2.93</td>
<td>0.415</td>
<td>&quot;</td>
<td>79.4</td>
<td>6.01</td>
<td>41.2*</td>
<td>15.5</td>
</tr>
<tr>
<td>July 11th, 1888</td>
<td>0.76</td>
<td>15</td>
<td>19.98</td>
<td>2.73</td>
<td>0.452</td>
<td>&quot;</td>
<td>71.2</td>
<td>6.01</td>
<td>41.2*</td>
<td>15.5</td>
</tr>
<tr>
<td>18th, 1888</td>
<td>0.75</td>
<td>10</td>
<td>19.83</td>
<td>1.655</td>
<td>0.357</td>
<td>&quot;</td>
<td>67.2</td>
<td>4.70</td>
<td>25.0*</td>
<td>17.0</td>
</tr>
<tr>
<td>August 4th, 1888</td>
<td>1.00</td>
<td>12</td>
<td>25.12</td>
<td>3.05</td>
<td>0.453</td>
<td>&quot;</td>
<td>72.6</td>
<td>10.01</td>
<td>24.1*</td>
<td>19.0</td>
</tr>
<tr>
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<td>1.518</td>
<td>15</td>
<td>40.56</td>
<td>3.853</td>
<td>0.498</td>
<td>&quot;</td>
<td>69.6</td>
<td>6.17</td>
<td>18.4*</td>
<td>21.5</td>
</tr>
<tr>
<td>17th, 1888</td>
<td>2.39</td>
<td>10</td>
<td>33.47</td>
<td>1.39</td>
<td>0.299</td>
<td>&quot;</td>
<td>60.4</td>
<td>6.17</td>
<td>18.4*</td>
<td>21.5</td>
</tr>
<tr>
<td>25th, 1888</td>
<td>2.50</td>
<td>14</td>
<td>32.80</td>
<td>3.50</td>
<td>0.500</td>
<td>&quot;</td>
<td>76.2</td>
<td>21.04</td>
<td>33.5*</td>
<td>28.0</td>
</tr>
</tbody>
</table>

* Preceded and followed by lighter rain.
1 Sudden shower, followed by lighter rain.
II Heavy shower, preceded by lighter rain.

1 Intensity roughly estimated.
§ Sewer not under head; figures given are for maximum discharge without head, previous to surcharge.
TABLE No. 8.

Showing the maximum flow and percentage of rainfall discharged during the period of such flow, in the Court and William Streets outlet sewer at Gauge No. 19, at intersection of Alexander Street and University Avenue. Tributary drainage area = 132.96 acres. Time required for passage of storm-water through longest line of sewers above said gauge = 15 minutes, which should be increased by about eight minutes for concentration in sewers. Coefficient of roughness: $n = 0.017$.

<table>
<thead>
<tr>
<th>DATE</th>
<th>Maximum Intensity of Rainfall</th>
<th>Duration of Rain at Maximum Intensity</th>
<th>Precipitation on Drainage Area</th>
<th>Cross-sectional Area of Flow in Sewer</th>
<th>Mean Hydraulic Radius $r$ of such Area</th>
<th>Adopted Slope of Water Surface $i$</th>
<th>Co-efficient of Velocity $c$ for $i = 0, f$</th>
<th>Maximum Sewer Discharge, CY.</th>
<th>Percent of Rainfall Discharged</th>
</tr>
</thead>
<tbody>
<tr>
<td>December 14th, 1897</td>
<td>0.31</td>
<td>0.31</td>
<td>0.31</td>
<td>0.31</td>
<td>0.31</td>
<td>0.31</td>
<td>0.31</td>
<td>0.31</td>
<td>0.31</td>
</tr>
<tr>
<td>April 5th, 1898</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>May 4th, 1898</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
<td>0.20</td>
</tr>
<tr>
<td>June 21st, 1898</td>
<td>0.49</td>
<td>0.49</td>
<td>0.49</td>
<td>0.49</td>
<td>0.49</td>
<td>0.49</td>
<td>0.49</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>July 22nd, 1898</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
<td>0.80</td>
</tr>
<tr>
<td>August 11th, 1898</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td>September 1st, 1898</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

* Preceded and followed by lighter rain.
† Sudden shower, followed by lighter rain.
‡ Heavy shower, preceded by lighter rain.

Intensity roughly estimated.
§ Sewer ran under much head; figures given are for maximum discharge without head, previous to surcharge.
TABLE No. 9.

Showing the maximum flow and percentage of rainfall discharged during the period of such flow, in the Griffith Street sewer at Gauge No. 30, at intersection of Griffith Street and Broadway.

Tributary drainage area = 92.27 acres. Time required for passage of storm-water through longest line of sewers above said gauge = 10 minutes, which should be increased by about eight minutes for concentration in sewers.

Co-efficient of roughness: \( n = 0.017 \).

<table>
<thead>
<tr>
<th>DATE</th>
<th>Maximum Intensity of Rainfall</th>
<th>Duration of Rain at Maximum Intensity</th>
<th>Precipitation on Drainage Area</th>
<th>Cross-sectional Area of Flow in Sewer</th>
<th>Mean Hydraulic Radius (( r )) of such Area</th>
<th>Adopted Slope (( \beta )) of Water Surface</th>
<th>Co-efficient for Velocity (( v )) in ( v = cv/\beta )</th>
<th>Maximum Sewer Discharge (( q ))</th>
<th>Percentage of Rainfall Discharged</th>
</tr>
</thead>
<tbody>
<tr>
<td>December 10th, 1907</td>
<td>0.31</td>
<td>60</td>
<td>28.61</td>
<td>2.08</td>
<td>0.510</td>
<td>17/20</td>
<td>74.2</td>
<td>7.43</td>
<td>20.0%</td>
</tr>
<tr>
<td>April 4th, 1907</td>
<td>0.24</td>
<td>36</td>
<td>21.13</td>
<td>1.48</td>
<td>0.425</td>
<td>&quot;</td>
<td>70.9</td>
<td>4.61</td>
<td>20.0%</td>
</tr>
<tr>
<td>May 4th, 1907</td>
<td>0.30</td>
<td>12</td>
<td>27.49</td>
<td>2.14</td>
<td>0.319</td>
<td>&quot;</td>
<td>74.5</td>
<td>7.92</td>
<td>20.0%</td>
</tr>
<tr>
<td>9th, 1907</td>
<td>0.35</td>
<td>33</td>
<td>22.33</td>
<td>4.10</td>
<td>0.172</td>
<td>&quot;</td>
<td>72.3</td>
<td>17.37</td>
<td>10.0%</td>
</tr>
<tr>
<td>13th, 1907</td>
<td>0.30</td>
<td>30</td>
<td>27.62</td>
<td>1.50</td>
<td>0.129</td>
<td>&quot;</td>
<td>71.0</td>
<td>4.70</td>
<td>10.0%</td>
</tr>
<tr>
<td>20th, 1907</td>
<td>3.00</td>
<td>10</td>
<td>12.00</td>
<td>3.76</td>
<td>0.300</td>
<td>&quot;</td>
<td>55.5</td>
<td>10.74</td>
<td>10.0%</td>
</tr>
<tr>
<td>June 5th, 1907</td>
<td>0.40</td>
<td>15</td>
<td>15.50</td>
<td>3.18</td>
<td>0.307</td>
<td>&quot;</td>
<td>68.1</td>
<td>3.23</td>
<td>8.0%</td>
</tr>
<tr>
<td>6th, 1907</td>
<td>2.62</td>
<td>20</td>
<td>21.43</td>
<td>6.90</td>
<td>0.750</td>
<td>&quot;</td>
<td>81.2</td>
<td>28.45</td>
<td>11.0%</td>
</tr>
<tr>
<td>20th, 1907</td>
<td>0.75</td>
<td>30</td>
<td>21.44</td>
<td>3.91</td>
<td>0.745</td>
<td>&quot;</td>
<td>84.1</td>
<td>27.65</td>
<td>33.4%</td>
</tr>
<tr>
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<td>0.75</td>
<td>15</td>
<td>20.14</td>
<td>2.28</td>
<td>0.621</td>
<td>&quot;</td>
<td>77.9</td>
<td>13.37</td>
<td>19.4%</td>
</tr>
<tr>
<td>17th, 1907</td>
<td>0.75</td>
<td>10</td>
<td>23.22</td>
<td>3.03</td>
<td>0.543</td>
<td>&quot;</td>
<td>74.0</td>
<td>7.14</td>
<td>10.3%</td>
</tr>
<tr>
<td>August 4th, 1907</td>
<td>1.00</td>
<td>12</td>
<td>22.57</td>
<td>2.12</td>
<td>0.099</td>
<td>&quot;</td>
<td>77.4</td>
<td>12.71</td>
<td>13.8%</td>
</tr>
<tr>
<td>10th, 1907</td>
<td>1.516</td>
<td>15</td>
<td>16.75</td>
<td>6.00</td>
<td>0.150</td>
<td>&quot;</td>
<td>81.2</td>
<td>28.45</td>
<td>11.0%</td>
</tr>
<tr>
<td>17th, 1907</td>
<td>3.38</td>
<td>10</td>
<td>12.06</td>
<td>3.00</td>
<td>0.341</td>
<td>&quot;</td>
<td>76.6</td>
<td>19.94</td>
<td>8.3%</td>
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<tr>
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<td>14</td>
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<td>&quot;</td>
<td>81.2</td>
<td>28.45</td>
<td>12.3%</td>
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<tr>
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<td>0.17</td>
<td>10</td>
<td>43.34</td>
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<td>0.053</td>
<td>&quot;</td>
<td>78.8</td>
<td>10.14</td>
<td>10.0%</td>
</tr>
</tbody>
</table>

* Preceded and followed by lighter rain.
† Sudden shower, followed by lighter rain.
‡ Heavy shower, preceded by lighter rain.
§ Intensity roughly estimated.
$ Sewer ran under head; figures given are for maximum discharge without head, previous to surcharge.
TABLE No. 10.

Showing the computed percentages of the heaviest rainfall discharged from five different city districts by the respective outlet sewers, during the period of maximum flow.

<table>
<thead>
<tr>
<th>DATE</th>
<th>Maximum Intensity of Rainfall</th>
<th>Duration of Rain at Maximum Intensity</th>
<th>Percentages of Rainfall Discharged</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Inches per Hour.</td>
<td>Minutes</td>
<td>District I</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Area = 426.91</td>
</tr>
<tr>
<td>December 15th, 1887</td>
<td>0.34</td>
<td>90</td>
<td>13.8</td>
</tr>
<tr>
<td>April 25th, 1888</td>
<td>0.15</td>
<td>12</td>
<td>10.4</td>
</tr>
<tr>
<td>May 4th</td>
<td>0.27</td>
<td>15</td>
<td>6.8</td>
</tr>
<tr>
<td>9th</td>
<td>0.34</td>
<td>36</td>
<td>16.4</td>
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<tr>
<td>12th</td>
<td>0.24</td>
<td>15</td>
<td>11.0</td>
</tr>
<tr>
<td>20th</td>
<td>0.10</td>
<td>15</td>
<td>3.6</td>
</tr>
<tr>
<td>June 5th</td>
<td>0.44</td>
<td>15</td>
<td>5.4</td>
</tr>
<tr>
<td>24th</td>
<td>1.2</td>
<td>18</td>
<td>7.4</td>
</tr>
<tr>
<td>24th</td>
<td>2.64</td>
<td>20</td>
<td>6.3</td>
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<tr>
<td>24th</td>
<td>0.96</td>
<td>20</td>
<td>14.3</td>
</tr>
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<td>July 11th</td>
<td>0.66</td>
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<td>7.4</td>
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<td>18th</td>
<td>0.24</td>
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<td>7.4</td>
</tr>
<tr>
<td>August 4th</td>
<td>1.02</td>
<td>12</td>
<td>4.6</td>
</tr>
<tr>
<td>16th</td>
<td>1.06</td>
<td>12</td>
<td>4.7</td>
</tr>
<tr>
<td>17th</td>
<td>1.62</td>
<td>10</td>
<td>5.5</td>
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<tr>
<td>September 16th</td>
<td>2.69</td>
<td>14</td>
<td>4.0</td>
</tr>
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</table>

Probable time required for concentration of flow at gauges: minutes

* Preceded and followed by lighter rain.
† Showed shower, followed by lighter rain.
‡ Heavy shower, preceded by lighter rain.

† Intensity roughly estimated, deducting half.
§ Showed here rain under head; percentage is computed from maximum discharge without head, previous to surcharge.
DISCUSSION ON RAINFALL AND DISCHARGE OF SEWERS

By Rudolph Hering, M. Am. Soc. C. E.

There is no doubt that all engineers who are engaged on municipal works are very much interested in the subject which has been so ably treated by Mr. Kuchling. Its importance is clear from the fact that if we make our sewers too large we spend more money than necessary, and if in built up cities we make them too small they are apt to cause considerable damage by flooding cellars. Therefore we are very anxious to know as nearly as possible the maximum quantity of rain-water for which provision is necessary. I have myself been particularly interested in the subject, and have often urged engineers to make experiments regarding the amount of rainfall entering sewers. Very little positive information is known about it either in our country or in Europe. There seems to be difficulty in convincing some municipal councilmen that the automatic rain-gage, for instance, is anything but a costly scientific play-thing; that was an objection I heard made some years ago.

You will remember there was a committee appointed by the Society last year to urge upon Congress the importance of having the Signal Service Bureau use automatic rain-gages in the principal cities, and a communication was sent to Washington asking for an appropriation of $2,000 for this purpose. This sum was not granted; but General Greeley has been enabled from his other funds to have some of those gauges erected, and I believe there are some working now.

He has also recently collected and published in the "Weather Review" the results of a large number of stray observations which show what heavy rainfalls we have in a very short duration of time. I thought before that a fall at the rate of 4 inches an hour was very rare, but I have come to the conclusion that we must have many storms with at least such intensity. I have here an extract from the "Review," in which I included rainfalls of over 3 inches in an hour, 2 inches in half an hour, and 1 inch in ten minutes.

In New York, Dr. Draper, of the Observatory at the Park, has kept an automatic registration since 1880, and from this I have taken the maximum intensity of storms for a few minutes, in order to determine the probable maximum quantity of water reaching the house drains.

If we reduce these amounts to a rainfall of inches per hour, we have four storms in New York City since 1880 which exceeded the rate of 4 inches an hour, continuing, at this rate, at least five minutes, two storms exceeding the rate of 6 inches an hour for at least three minutes, and one storm exceeding the rate of 7 inches an hour for two minutes. Four times within the last eight years has there been a fall at the rate of
58 DISCUSSION ON RAINFALL AND DISCHARGE OF SEWERS.

5 inches an hour for at least five minutes, sufficiently long to allow the water from the roofs to get into the house drains.

Complaints have often been made to the Board of Health that the drains were too small and an investigation was made by the Department of Public Works to find out whether they were or not. The result indicated that 6-inch pipes draining a property having 25 feet front, laid at a grade of 1 inch to the foot, were large enough to take a rainfall having the intensity of 6 inches per hour.

The Department of Public Works has made gaugings of the flow in a large sewer, but I regret to say that as the results have not yet been reported to the Commissioner, I am not able to give any details. But the large figures that Mr. Kuichling has arrived at are fully verified by the observations made here. It is certain that we have generally underestimated the quantity of water that comes into the sewers in our most populous cities.

The gaugings I refer to were made in a district of New York which is pretty densely built up. It has been found that three times during the last year, when the maximum rainfalls were not over .6 of an inch in ten minutes, more than 1 cubic foot per second per acre reached the sewer. That is considerably more than is given by the formulas we have been in the habit of using.

In connection with this work I tried to collect all the relative data that were available, and proceed somewhat on the method adopted by Mr. McMath in a paper read before this Society December 15th, 1886, by recording the elements of sewers that were, and those that were never, overtaxed by the heaviest rains. Then, by drawing a line between the plottings of the two results, we obtain the approximate capacity which the sewers ought to have. I have collated a number of such data, and arranged them in tabular form, which I think indicate very closely the maximum flow from the heaviest rains.

These results show that some of our sewers have been built too large, and others too small.

The Chair.—May I ask Mr. Heising if it is intended to publish the results of his experiments when they are completed?

Mr. Heising.—They will be presented to Mr. D. Lowher Smith, Commissioner of Public Works and Member of the Society. In a conversation with him he said that the detailed matter would probably be a better subject for a paper to be presented to the Society than to print it in the Commissioner's Annual Report, as the public are not particularly interested in technical details of this nature.

I think there is one point in Mr. Kuichling's formula which presents a little difficulty. He introduces the element of time. Now, that is rather difficult to fix upon in practice. Recognizing this, I tried another plan, one which I thought more practicable, and that is by substituting the average slope of the ground. The diagrams are arranged
so that the ordinates represent the discharges, and the abscissæ the drainage areas. The discharges from different areas having the same slope are united by one curve. For the same area it is seen that the greater the slope, the greater will be the discharge, so that the element of time is introduced in that way.

Maximum Rainfall in short periods of ten minutes or less between January, 1880, and January, 1888, as recorded by the New York Meteorological Observatory.

<table>
<thead>
<tr>
<th>Date</th>
<th>Max. fall in inches</th>
<th>Time in minutes</th>
</tr>
</thead>
<tbody>
<tr>
<td>May 23, 1884</td>
<td>1.15</td>
<td>10</td>
</tr>
<tr>
<td>June 5th, 1881</td>
<td>0.34</td>
<td>30</td>
</tr>
<tr>
<td>6th</td>
<td>0.24</td>
<td>30</td>
</tr>
<tr>
<td>13th</td>
<td>0.22</td>
<td>30</td>
</tr>
<tr>
<td>28th</td>
<td>0.15</td>
<td>30</td>
</tr>
<tr>
<td>July 20th, 1884</td>
<td>0.10</td>
<td>30</td>
</tr>
<tr>
<td>10th, 1884</td>
<td>0.20</td>
<td>30</td>
</tr>
<tr>
<td>September 21st, 1882</td>
<td>0.25</td>
<td>30</td>
</tr>
</tbody>
</table>

Maximum Rainfall compiled from Monthly Weather Review, United States Signal Service, and published during the year 1888, having falls of:

3.0 inches in .................................. 1 hour 0 minutes.
2.0 " " ........................................ 30 " "
1.0 " " ........................................ 10 " "

and over.

<table>
<thead>
<tr>
<th>State</th>
<th>City</th>
<th>Date</th>
<th>Amount in inches</th>
<th>Temp. in Min.</th>
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<tr>
<td>New Hampshire</td>
<td>Concord</td>
<td>August 29th, 1887</td>
<td>3.11</td>
<td>30</td>
</tr>
<tr>
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<td>Boston</td>
<td>June 29th, 1879</td>
<td>2.20</td>
<td>20</td>
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<tr>
<td>Rhode Island</td>
<td>Providence</td>
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<td>2.00</td>
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<tr>
<td>Connecticut</td>
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<tr>
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<tr>
<td>Pennsylvania</td>
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<td>Galveston</td>
<td>April 19th, 1883</td>
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</table>
DIAGRAM showing the relation between the rates and durations of rainfall in the cities of ROCHESTER for the period from 1871 to 1888, of BUFFALO from 1872 to 1882 of OSWEGO from 1870 to 1887 and of ITHACA from 1878 to 1888, compiled from U.S. Signal Service and other records.

- represent rainfalls at ROCHESTER
- BUFFALO
- OSWEGO
- ITHACA
- HEMLOCK LAKE

Scale: 1 Vertical, 1 inch = 0.3 inches per hour.
APPENDIX C
CALCULATIONS AND PRINTOUTS
Project Name and Location: CRAYCROFT WASH TRIBUTARY

Drainage Concentration Point: A

Watershed Area (A) at Concentration Point: 24.3 acres

Length of Hydraulically Longest Watercourse (Lc): 2165 ft.

Length from Center of Watershed Area (Lca) along Lc: 1100 ft.

Mean Slope (Sc) = (Lc/Lca) = 0.07300 ft./ft.

Basin Factor (Nbw): 0.022

Runoff Coefficient (Cw): 0.75

Z-Factor (z) = 0.444 Nbw (Lc/Lca)^.3 / (Sc Cw)^.4 = 2.556 (future)

Time of Concentration (Tc): 5.0 minutes (future).

At Tc, 100-year Rainfall Intensity (I) = 12/ (1+0.05 Lc) = 9.6 inches/hour.

100-year Flood Peak = Cw * A = 175 cfs (future)

Degree of Urbanization: Moderately Urban

For Other Return Periods:

NOTE: Use Pima County Method (with City Rainfall) if time of Concentration is Needed for Lesser Return Periods

25-year: 114 cfs
10-year: 79 cfs
2-year: 35 cfs

Prepared By: CURTIS LUECK
Company: UNIVERSITY OF ARIZONA
Date: 20 FEB 89

Report Name: 
City Base Map: 
Planning Case Number: 
FOR CITY USE ONLY: Program ID: COTM Version 1.3 (PC) December, 1984

FOR CITY USE ONLY: Program 10: COTM Version 1.3 (PC) December, 1984
Project Name and Location: CRAYCROFT WASH TRIBUTARY

Drainage Concentration Point: A

Watershed Area (A) at Concentration Point: 24.3 acres

Length of Hydraulically Longest Watercourse (Lc): 2165 ft.

Length from Center of Watershed Area (Lca) along Lc: 1100 ft.

Mean Slope (Sc) = (Lc/Lca) .2 = 0.07300 ft./ft.

Basin Factor (Nbw): 0.032

Runoff Coefficient (Cw): 0.65

Z-Factor (z) = 0.444 Nbw (Lc/Lca) .3 / (Sc Cw) .4 = 3.596 (future)

Time of Concentration (Tc): 5.0 minutes (future).

At Tc, 100-year Rainfall Intensity (i) = 12/i(1+0.05 Tc) = 9.6 inches/hour.

100-year Flood Peak = Cw A = 152 cfs (future)

Degree of Urbanization: Suburban

For Other Return Periods:

NOTE: Use Pima County Method (with City Rainfall) if Time of Concentration is Needed for Lesser Return Periods

25-year: 91 cfs
10-year: 61 cfs
2-year: 23 cfs

Prepared By: CURTIS LUECK
Company: UNIVERSITY OF ARIZONA
Date: 20 FEB 87

FOR CITY USE ONLY:
Program ID: COTM Version 1.0 (FC)
December, 1984

Report Name:
City Base Map:
Planning Case Number:
Project Name and Location: CHOLLA WASH

Drainage Concentration Point: MISSION RUHD

Watershed Area (A) at Concentration Point: 813.0 acres

Length of Hydraulically Longest Watercourse (Lc): 17213 ft.

Length from Center of Watershed Area (Lca) along Lc: 8000 ft.

Mean Slope (Sc) = (Lc/I) 2 = 0.01500 +t./ft.

Basin Factor (Nb): 0.032

Runoff Coefficient (Cw): 0.70

Z-Factor \( z \) = 0.444 NbW Lc Lca \( 0.3 / (Sc \ Cw) \) / 4 = 24.308 (future)

Time of Concentration (Tc): 37.0 minutes (future).

At Tc, 100-year rainfall intensity \( i \) = 12/1+0.05 Tc = 4.2 inches/hour.

100-year Flood Peak = Cw \( A = 2596 \text{ cfs} \) (future)

Degree of Urbanization: Suburban

For Other Return Periods:

NOTE: Use Pima County Method (with City Rainfall) if Time of Concentration is Needed for Lesser Return Periods

25-year: 1438 cfs
10-year: 958 cfs
2-year: 359 cfs

Prepared By: CURTIS LUECK

Company: UNIVERSITY OF ARIZONA

Date: 29 FEB 84
Project Name and Location: CHULLA WASH

Drainage Concentration Point: MISSION ROAD

Watershed Area (A) at Concentration Point: 813.9 acres

Length of Hydraulically Longest Watercourse (Lc): 17213 ft.

Length from Center of Watershed Area (Lca) along Lc: 8000 ft.

Mean Slope (Sc) = (Lc/I)^2 = 0.01500 ft./ft.

Basin Factor (Nbw): 0.022

Runoff Coefficient (Cw): 0.75

Z-Factor (z) = 0.444 Nbw (Lc Lca)^.5 / (Sc Cw)^.4 = 16.257 (future)

Time of Concentration (Tc): 22.0 minutes (future).

At Tc, 100-year Rainfall Intensity (i) = 12/(1+0.15 Tc) = 5.7 inches/hour.

100-year Flood Peak = Cw i A = 3484 cfs (future)

Degree of Urbanization: Moderately Urban

For Other Return Periods:

NOTE: Use Pima County Method (with City Rainfall) if Time of Concentration is Needed for Lesser Return Periods

25-year: 2265 cfs
10-year: 1568 cfs
2-year: 697 cfs

Prepared By: CURTIS LUECK
Company: UNIVERSITY OF ARIZONA
Date: 20 FEB 89

FOR CITY USE ONLY: Program ID: CDTM Version 1.3 (PC)

Report Name:
City Base Map:
Planning Case Number:
CITY OF TUCSON FLOOD FLOW ESTIMATION METHOD

WATERSHED AREA (Acres) = 6112
Area is too large -- Use Pima County Method.
Area is too Large -- Use Pima County Method.

Alamo Wash
<table>
<thead>
<tr>
<th>WATERSHED</th>
<th>C</th>
<th>I</th>
<th>A</th>
<th>Q (CFS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CWT</td>
<td>.45</td>
<td>9.0</td>
<td>24</td>
<td>97</td>
</tr>
<tr>
<td>CHOLLA</td>
<td>.35</td>
<td>4.0</td>
<td>812</td>
<td>1136</td>
</tr>
<tr>
<td>ALAMO</td>
<td>.35</td>
<td>3.7</td>
<td>6118</td>
<td>7923</td>
</tr>
</tbody>
</table>

RATIONAL

METHO

D I S C H A R G E
C A L C U L A T I O N S

2/20/89
HYDROLOGIC DATA SHEET
FOR WATERSHEDS OF LESS THAN TEN (10) SQUARE MILES

<table>
<thead>
<tr>
<th>Project Name and Location: Alam Wash, Tucson, AZ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage Concentration Point: Fort Lowell Road</td>
</tr>
<tr>
<td>Watershed Area (Ac): 4616 acres</td>
</tr>
<tr>
<td>Watershed Type: Urban</td>
</tr>
<tr>
<td>Length of Watercourse (Lc): 32860 ft</td>
</tr>
<tr>
<td>Length to Center of Gravity (Lc): 17000 ft</td>
</tr>
<tr>
<td>Incremental Change in Length (Lc): ft</td>
</tr>
<tr>
<td>Incremental Change in Elevation (Hc): ft</td>
</tr>
<tr>
<td>32860.0</td>
</tr>
<tr>
<td>Incremental Cover: 50%</td>
</tr>
<tr>
<td>Cover Density: 25%</td>
</tr>
<tr>
<td>Soil Group: C</td>
</tr>
<tr>
<td>Cover Type: Urban Law, Desert Grass</td>
</tr>
<tr>
<td>Mean Slope Sc: 1.0039 (1/f.ft.)</td>
</tr>
<tr>
<td>Basin Factor Multiplier: .522</td>
</tr>
</tbody>
</table>

Adjusted Curve Numbers (CN)

<table>
<thead>
<tr>
<th>CN</th>
<th>(P100)</th>
<th>(P50)</th>
<th>(P25)</th>
<th>(P10)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>68.00</td>
<td>90.73</td>
<td>90.14</td>
<td>89.29</td>
</tr>
<tr>
<td></td>
<td>90.00</td>
<td>86.50</td>
<td>87.29</td>
<td>84.94</td>
</tr>
</tbody>
</table>

Storm Recurrence Interval (yr)

<table>
<thead>
<tr>
<th>100</th>
<th>50</th>
<th>25</th>
<th>10</th>
<th>5</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>P24 (24 Hour) in.</td>
<td>4.20</td>
<td>3.72</td>
<td>3.24</td>
<td>2.76</td>
<td>2.20</td>
</tr>
<tr>
<td>P6 (6 Hour) in.</td>
<td>3.40</td>
<td>3.12</td>
<td>2.84</td>
<td>2.26</td>
<td>2.08</td>
</tr>
<tr>
<td>P1 (1 Hour) in.</td>
<td>1.17</td>
<td>1.08</td>
<td>1.01</td>
<td>0.94</td>
<td>0.87</td>
</tr>
<tr>
<td>P3 (1 Hour) in.</td>
<td>1.15</td>
<td>1.04</td>
<td>1.03</td>
<td>1.01</td>
<td>0.97</td>
</tr>
<tr>
<td>P2 (2 Hour) in.</td>
<td>3.14</td>
<td>2.78</td>
<td>2.37</td>
<td>2.03</td>
<td>1.86</td>
</tr>
</tbody>
</table>
| Runoff to Rainfall Ratio (C):
| (Pervious Areas)    | 0.66 | 0.96 | 0.91 | 0.82 | 0.76 |
| (Impermeable Areas) | 0.95 | 0.94 | 0.93 | 0.92 | 0.90 |
| (Weighted, C)       | 0.80 | 0.77 | 0.74 | 0.69 | 0.65 |
| Time of Concentration (hr):
| 141.51 | 171.16 | 197.54 | 226.24 | 253.59 | 226.04 |
| Rainfall Intensity (in) at Tc (in/hr):
| 1.08  | 0.98  | 0.90  | 0.84  | 0.78  | 0.72  |
| Runoff Supply Rate (ft) at Tc (in/hr):
| 0.67  | 0.68  | 0.52  | 0.47  | 0.42  | 0.37  |
| Peak Discharge: 1,000 cfs (cfs):
| 5342.7 | 4214.3 | 3191.2 | 2285.8 | 1673.6 | 959.7 |
| PEAK DISCHARGE (cfs):
| 5343 | 4214 | 3191 | 2286 | 1694 | 897 |
HYDROLOGIC DATA SHEET
FOR WATERSHEDS OF LESS THAN TEN (10) SQUARE MILES

Job Number: none
Date: 19 Feb 87
By: Curtis L. Lane

Project Name and Location: Cholla Wash, Tucson, AZ

Drainage Concentration Points: Mission Road

Watershed Area (ac): 812 acres
Length of Watercourse (ft): 17212 ft

Watershed Types: Suburban
Length to Center of Gravity (ft): 8600 ft

Incremental Change in Length (L) = ft.

Incremental Change in Elevation (H) = ft.

17212.0
296.0 - 100.0 = 196.0

Impervious Covers: 33 %
Cover Density: 25 %
Soil Groups: C
Cover Types: desert brush
Mean Slope (S): 0.0104 ft/ft.
Basin Factor (mb): 0.022

Adjusted Curve Numbers (CNs)

<table>
<thead>
<tr>
<th>Soil</th>
<th>CN</th>
<th>R10</th>
<th>R25</th>
<th>R50</th>
<th>R100</th>
<th>R500</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>88.00</td>
<td>87.01</td>
<td>86.55</td>
<td>87.71</td>
<td>86.60</td>
<td>85.50</td>
</tr>
</tbody>
</table>

Storm Recurrence Interval (yrs)

<table>
<thead>
<tr>
<th>100</th>
<th>50</th>
<th>25</th>
<th>10</th>
<th>5</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>P24 (24 Hour) in.</td>
<td>4.20</td>
<td>3.72</td>
<td>3.24</td>
<td>2.1b</td>
<td>2.40</td>
</tr>
<tr>
<td>P6 (6 Hour) in.</td>
<td>3.40</td>
<td>3.02</td>
<td>2.64</td>
<td>1.78</td>
<td>1.78</td>
</tr>
<tr>
<td>P1 (1 Hour) in.</td>
<td>2.57</td>
<td>2.29</td>
<td>2.01</td>
<td>1.52</td>
<td>1.77</td>
</tr>
<tr>
<td>P2 (2 Hour) in.</td>
<td>2.05</td>
<td>1.84</td>
<td>1.62</td>
<td>1.47</td>
<td>1.28</td>
</tr>
<tr>
<td>P3 (3 Hour) in.</td>
<td>1.94</td>
<td>1.71</td>
<td>1.47</td>
<td>1.78</td>
<td>1.46</td>
</tr>
<tr>
<td>Runoff to Rainfall Ratio (C): (Impervious Areas)</td>
<td>0.60</td>
<td>0.54</td>
<td>0.44</td>
<td>0.39</td>
<td>0.32</td>
</tr>
<tr>
<td>(Pervious Areas)</td>
<td>0.95</td>
<td>0.93</td>
<td>0.93</td>
<td>0.92</td>
<td>0.87</td>
</tr>
<tr>
<td>(weighted, Cw)</td>
<td>0.71</td>
<td>0.68</td>
<td>0.63</td>
<td>0.57</td>
<td>0.52</td>
</tr>
<tr>
<td>Time of Concentration (min):</td>
<td>28.77</td>
<td>31.56</td>
<td>35.27</td>
<td>40.49</td>
<td>40.91</td>
</tr>
<tr>
<td>Rainfall Intensity (in/hr) at 1c:</td>
<td>4.18</td>
<td>3.33</td>
<td>2.69</td>
<td>2.27</td>
<td>1.83</td>
</tr>
<tr>
<td>Runoff Supply Rate (ft) at 1c (in/hr):</td>
<td>2.99</td>
<td>2.39</td>
<td>1.82</td>
<td>1.30</td>
<td>0.95</td>
</tr>
</tbody>
</table>

Peak Discharge: 1.000B ft³/s:

2446.3 | 1955.7 | 1489.3 | 1062.5 | 774.1 | 387.8 |

PEAK DISCHARGE (cfs):

2446
HYDROLOGIC DATA SHEET
FOR WATERSHEDS OF LESS THAN TEN (10) SQUARE MILES

Project Name and Location: Craycraft Wash Tributary, Tucson, AZ

Drainage Concentration Point: Outlet

Watershed Area (Ac): 24 acres
Length of Watercourse (Lc): 2165 ft

Incremental Change in Length (Lc) - ft. 2165.0
Incremental Change in Elevation (Hc) - ft. 490.0 - 100.0 = 390.0

Watershed Type(s): Suburban

Impervious Cover: 45 %
Cover Density: 35 %

Soil Group(s): c

Cover Type(s): urban lawn

Mean Slope (S): 0.18014 ft./ft.

Basin Factor (nb): .022

Adjusted Curve Numbers (CN):

<table>
<thead>
<tr>
<th>Soil CN</th>
<th>R1(0)</th>
<th>R50</th>
<th>R25</th>
<th>R10</th>
<th>R5</th>
<th>R2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>87.00</td>
<td>89.97</td>
<td>89.55</td>
<td>87.49</td>
<td>86.44</td>
<td>85.05</td>
</tr>
</tbody>
</table>

Storm Recurrence Interval (yr):

<table>
<thead>
<tr>
<th></th>
<th>100</th>
<th>50</th>
<th>25</th>
<th>10</th>
<th>5</th>
<th>2</th>
</tr>
</thead>
</table>

Runoff to Rainfall Ratio (CI):
(Pervious Areas)
0.62 0.57 0.50 0.42 0.34 0.19
(Impervious Areas)
0.95 0.95 0.95 0.95 0.92 0.70
(weighted, Cw)
0.77 0.74 0.70 0.65 0.61 0.51

Time of Concentration (Tc) in.
5.00 5.00 5.00 5.00 5.00 5.00

Rainfall Intensity (I) at Tc (in/hr)
9.10 8.10 7.11 6.12 5.37 4.15

Runoff Supply Rate (q) at Tc (cfs/hr)
7.01 5.99 4.97 3.95 2.92 2.12

Peak Discharge: 1.00664 cfs:
169.5 144.8 120.3 96.2 78.7 51.2

PEAK DISCHARGE (cfs):

170
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