STORM WATER MANAGEMENT SEMINAR
PART 1

BY
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IN DEDICATION

To those in the public service striving for a better quality of life in our cities, and who fairly administer the public's business with equity to all segments of the community.

THE AUTHOR

David L. Daugherty is a 1959 BCE graduate of the University of Louisville Speed Scientific School. His 16 years of specialized interest and training in professional hydraulics includes six years of intensive experience with the United States Army Engineers, Civil Works Division and thereafter in private practice as an engineering consultant. Since 1971 the author has served the Jefferson Fiscal Court (Jefferson County, Kentucky) as Water Management Engineer. Concurrently, since 1973, he has acted as Water Management Consultant to the Lexington-Fayette Urban County Council, Lexington, Kentucky. Mr. Daugherty is a member of the American Public Works Association and assists several universities or college faculties in the promulgation of academic programs relating to urban storm water management planning and design.

The policies, theory and practical applications in this publication are predicated on the author's design or regulatory involvement with an estimated 4,000 separate urban area projects of diverse description.

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DEFINITIONS

1. modified - conditions after new construction

2. natural - conditions before new construction

3. off-site - external to the boundary of a new project

4. on-site - internal to the boundary of a new project

5. point discharge - release of storm water at a specific location

6. retention - (sometimes termed detention) restraining the rate of storm run-off with some natural or man-made device

7. revetment - bank protection on natural or man-made channel

8. rill - specific eroded area, usually on a slope or stream bank, but which can develop in moderately flat areas

9. run-off - rainfall excess after natural losses from infiltration, evaporation, transpiration or incidental pondage

10. sheet drainage - overland run-off prior to reaching a drainage system and usually considered as being moderately uniform flow through natural growth, paved area, etc.

11. stage - depth of flow

12. swale - a surface-type conveyance for storm water, usually informally designed to convey incidental, localized run-off

13. through-drainageway - any ditch, stream, channel or creek which definably originates upstream of a tract in question and passes through that tract

14. velocity dissipator - any chute block, impact basin, stilling basin, roller bucket, flip bucket, or other device which effectively diminishes the energy content of discharge so as to abate the downstream damage potential

CHAPTER 1

INTRODUCTION

Anyone familiar with the quality of life in most American cities, large or small, is aware of the impact of storm water characteristics on the total community. To be sure, there are a few cities with reasonably satisfactory storm water facilities, but the vast majority suffer with tragically inadequate systems. Where such inadequacies exist, the population lives under constant threat of private and public flood damages, and during non-flood periods unhealthy environments are generated by poor neighborhood drainage characteristics. Since few who can afford to do so will live in unsatisfactory environments, poorly drained of floodprone neighborhoods tend towards economic decline. If enough neighborhoods of this type exist, the economic decline of the total community is likely to follow.

Most local governments strive to upgrade their problem areas where funds are available. Unfortunately, many of these same well-intentioned governments continue to permit developmental expansion under the same system of controls which created the old problems. It is usually a losing race with new problems occurring faster than older ones can be corrected. This is not always apparent in view of the time lag between problem creation and the time enough people are adversely effected to focus corrective interest by officials. A continuous cycle of publicly financed remedial construction can ultimately bankrupt a city of metropolitan area government.

Experience has proven that adverse storm water effects of new development can be effectively curtailed or eliminated altogether, thus allowing elected officials to focus their resources on correction of the old problems. This same experience has shown that a fresh, vigorous approach to storm water controls on new development can not only be fruitful for the local government and its' constituency, but can be beneficial to a development industry freed from endless litigation and other citizen opposition.

Modern storm water control techniques are not radically new to the American scene. And yet while some communities have practiced water management at various levels of effectiveness, techniques are constantly evolving on the basis of experience and new technology. Unfortunately, very few engineers are proficient in this technology, and even fewer have had the opportunity to practice water management intensively enough to address the over-all mechanics of an effective program. This publication will attempt to do so, hopefully including information beneficial to engineers, planners, public officials, industrialists and, above all, the general public to whom the professionals have an obligation to serve.

The author does not represent the methods in this publication as being infallible for all communities with storm water problems, but they seem to have worked reasonably well for several years where implemented under the auspices of elected officials who are determined to succeed. The significance of a methodology rooted in sound engineering principles and proven experience should be of interest to those seeking alternatives in abating the urban storm water problem.

CHAPTER 2

THE EVOLUTION OF STORM WATER MANAGEMENT

Storm water facilities design and construction has generally been the least understood and most neglected aspect of city planning. From the perspective of community founders, everyday needs such as roads, water supply systems, housing and other facilities took precedence in their order of priorities. In the early city struggle for survival, the immediate demand for these other municipal needs often excluded even modest consideration for drainage facilities. Unfortunately this attitude prevailed long past the time when other basic survival issues were no longer in question. In many instances even predating the twentieth century, cities had become complacent.

During the post World War II period, population centers formerly recognized as compact communities rapidly evolved into large metropolitan areas. Roads, housing facilities, and commercial areas rapidly radiated from the urban core, employing a drainage system practice reflecting essentially the same level of complacency evident during the communities' formative years. Convenient, yet often inaccurate "rules of thumb" were used in the design and placement of drainage facilities and inadequate thought was given to future maintenance and replacement costs. The old problems remained in existence and were compounded by developmental expansion. Not only were flood damages to individual citizens escalating at an alarming rate, but local governments were confronted with collective neighborhood damages which had to be solved, but without sufficient income from traditional sources.

In the absence of effective governmental controls, citizen groups began resorting to private legal actions against the development industry where it could be proved that new construction would add to area flooding. In a number of states where tested, the courts have found for the citizen actions thereby either curtailing development or forcing local government to exert positive controls. Some local governments have gone even further and enacted ordinances which have the ultimate effect of stopping developmental expansion. Extreme measures of this sort, however, can adversely effect a community in other ways, particularly where a minimum of expansion is necessary to meet ever-growing community needs in housing, the job market, schools and similar facilities. Clearly a new, even-handed approach to developmental control has become necessary for harmonious growth.

Various communities throughout the United States have evolved their own contemporary approaches to drainage-related controls on new development, and up to the present, most have developed their programs independent of other cities using parallel methods.

While the extent to which each program is successful varies considerably, the fact remains that each passing year heralds a large increase in the number of local governments adopting some type of new drainage control systems.

Not infrequently, local governments have been supported in these efforts by industry leaders who perceive the need for responsible controls, either for genuine civic-minded reasons or for the motivation that, otherwise, controls would be superimposed indirectly in the form of court judgements. Then to, it has become increasingly obvious that the Federal Government tends to step into regulatory vacuums where local governments are not responding to environmental problems. By and large the development industry prefers local to Federal regulatory procedures.

Notwithstanding the evident problem needs and available corrective technology, many cities have yet to enact some form of storm water management controls. For whatever reasons particular governments desire not to implement an effective program, the lack of information should not be one of them.

CHAPTER 3

WATER MANAGEMENT PRINCIPLES

Principle is defined formally as a "general truth or law, basic to other truths." Another definition states "moral standards collectively."

Water Management Principles are founded on the assumption that collective moral standards dictate the unacceptability of one entity developing land in such fashion as to induce water-related damages to a passive land owner in the vicinity. It is an equally basic truth that an active owner should be allowed to labor in the development marketplace without undue hindrance by either government or other land owners insofar as buyers or off-site owners are not adversely effected. These two principles equate to a fair balance of land owner rights.

The principles need not be defined with statutory specificity. Regulatory experience in the field of hydraulic design has proven that specific guidelines are a severe handicap to both the developer and the regulatory agency. Far more effective is a program where there is adherence to the two basic principles, yet yielding latitude for the designer and government to flex in coping with a particular problem at hand. Obviously a flexible program placing reliance on two principles must be administered by a technologically proficient agency, and in an equitable fashion well tempered by good judgement. This should not alarm those concerned with capricious requirements by the regulator because data unfolding in later sections of this publication will make evident physical phenomena tending to induce damages unless properly checked.

It is not the intent of this publication to attempt advice on legal instruments necessary for the community to achieve and effective Water Management program. Some local governments are convinced that they have the power to regulate for the protection of their citizens through policies, ordinances or resolutions, and are willing to accept any and all challenges to the reasonableness of a prudently administered regulation. Others desire general statutory provisions which yield broad water-related authority to the City Engineer, County Engineer, or even the Planning Commission. Whatever course is adopted, it seems apparent that local government has not only the right but the responsibility to prevent damaging flood aggravation to the community.

CHAPTER 4

WATER MANAGEMENT OBJECTIVES

As a design and regulatory device, Water Management has two primary objectives.

(1) To insure that new construction of all descriptions shall not impose measurable off-site water-related damages.

(2) To insure that new construction shall be free from on-site water-related damages.

These two objectives pose a host of considerations, or secondary objectives, which must be weighed with each project. The number and importance of the secondary objectives vary from job to job, and are a function of the type and size of project, topography, off-site drainage characteristics, soil types, and other elements germane to the entire drainage perspective. The following is a partial listing of secondary objectives which should be considered for each project.

Off-site Considerations

(1a) During a 100-year frequency storm event, the new project will usually generate an increase in run-off rates owing to re-grading, new impervious areas, and possibly a more efficient on-site drainage system. If the run-off increases computationally aggravate downstream flood damages, then it becomes apparent that the developer must employ off-setting measures. Storm water retention basins either on-site or off-site may be the most desirable means of preventing flood aggravation. An off-site channel improvement may provide the same ultimate effect by enabling a reduction in flood stage height. Diversion of storm water, carefully performed with a basin in-line on the watershed receiving diverted flow is another alternative. There are many options to preclude an increase in off-site flooding.

(1b) Not uncommonly, new projects are inland from a defined drainage-way. Natural run-off from the undeveloped pasture or woodland sheets across property lines, sometimes crossing more than one tract before reaching a defined stream or pipe system. In cases of this sort, even on-site retention will not prevent point discharge from gouging a rill across the neighboring tract. While there is accepted responsibility by the neighboring owner to receive up-hill run-off, it behooves the developer to work out a reasonable arrangement with the neighbor whereby erosive damages are avoided.

(1c) Despite the benefits of on-site retention and the utilization of an acceptable receiving stream, point discharge of storm run-off tends to alter natural characteristics at the development boundary. If the point discharge is high enough to produce erosion, a velocity dissipator, revetment, or other counter measures may be necessary.

(1d) Multiple locations around a project perimeter where the transfer of sheet flow naturally occurs often become problem locations because modified sheeting characteristics are not properly placed under control. In those instances where storm water formerly sheeted onto a vacant field from, say, an adjoining residential yard and the field is developed into lots, a perimeter swale often becomes necessary to avert flowage obstructions caused by contractor re-grades. Conversely, where natural sheeting from undeveloped land onto the adjoining builder, the perimeter swale effectively intercepts flow.

(1e) Through-streams are usually altered by a new development. Sometimes the stream is piped, channeled, or even left in the natural state for scenic or economic reasons. Piping and channeling can alter upstream flow characteristics, and even when the natural state is honored, culverts, and over-lot filling effects flood profiles. Any developmental treatment of a through-stream should be computationally checked to insure that there is no increase in height for a 100-year flood profile at the upstream development boundary or upstream point of damage.

(1f) A common water-related damage is that caused by the transfer of soil across the development boundary by storm run-off. Soil transfer can clog downstream pipes, channels and streams, yards and buildings can be silted, and aquatic life is threatened. In areas where sinkholes prevail, siltation can impede or totally stop the sink functionality. There are many ways to curtail the transfer of soil during low to moderate rainfall events during project construction, and these alternatives should be considered as an integral part of development design.

All of the foregoing are usually major considerations in proposed developments, but there can be others. All reasonable questions of measurable off-site damages should be addressed and satisfied. A following section will deal with the question of what constitutes damages.

On-site Considerations

(2a) Whether or not a community is signatory to the federally sponsored Flood Plain Insurance program is immaterial to the local government's responsibility to its' constituency to prevent flood-prone new construction from being sold to an unwitting public. However, there is no question but that the federal program greatly strengthens a local flood plain program. The 100-year storm is considered to be a reasonable minimum protective level and is discussed under the section entitled "Rainfall Characteristics." All on-site residential, commercial and industrial buildings should be checked to preclude a damage level (first floor or basement) at least on foot above the 100-year flood profile. It should be noted that this check applies not only to those tracts contiguous to main streams, but also any tract subject to flooding from roadside ditches or drainage easements along side or rear property lines. Many communities consider that new public rights-of-way should be free from flooding and should be subject to the same type of check.

(2b) On-site drainage facilities should be checked for efficient, durable functionality. There is little logic in allowing new roads and drainage facilities to be placed in the public system (by record plat) at an unrealistically low construction cost when the public at large will have to meet the cost of reconstruction in a few years. New open channels should be erosion resistant and reasonably self-cleaning, and there should be access provisions for routine maintenance. Pipe systems should be properly jointed to prevent cratering, aligned to preclude opposing flow at catch basins, and otherwise designed for durable functionality.

(2c) The residential subdivision designer should take into account regrade possibilities by builders when laying out the lot pattern, or when considering cuts and fills in the subdivision overlot plan. There may be instances where it is desirable to insert regrade controls on the subdivision record plat.

(2d) Where the residential subdivision employs individual driveway culverts, there should be controls to prevent errors in placement which serves to obstruct flow to others who take the trouble to install culverts properly.

(2e) Drainage easements, particularly where inscribing open channels, should be designed to preclude abuse from reasonable property owners. While there is never any assurance that lot buyers will be reasonable, ill-planned open drainage easements invite intrusion by fencing, filling and undesirable plantings. Effective design can give considerable assistance to the developer and government in policing the integrity of easements as the years go by.

(2f) As a part of on-site design, there should be a clear understanding between the developer, government, and public as to responsibility for future maintenance of drainage and retention easements. A non-functioning drainage system is no drainage system at all.

CHAPTER 5

WATER RELATED DAMAGES

Rivers, creeks, streams and man-made drainage-ways are an unavoidable consequence of natural rain-fall, and it should be recognized that owners of contiguous lands (riparian owners) accepted the inherent disadvantages of riparian ownership as well as the advantages when they purchased the property. Periodic flooding, bank attack and natural accretion are several examples of undesirable consequences the riparian owner must expect. And yet, the riparian owner has a right to governmental protection from certain damages or aggravations of the natural undesirable events.

Just as beauty lies in the eyes of the beholder, damages can be what lies in the perception of the damaged party. Owing to the diversity in citizen attitudes and multiplicity in definitions of damages, government can not expect to regulate storm water to the total satisfaction of all. For the most part, however, reasonable people have very little trouble discerning what constitutes significant damages most of the time. The following are several guides which have proven workable.

Damages caused by measurable increases in flood stage

(1-a) Just as increases of in-bank stages pose little problems, slight increases in some overbank flows on an infrequent basis may cause no discernable damages where the overbank consists of woodland or relatively unimproved rear yards in residential districts. But yards improved to a high state, garages, houses or commercial buildings are ordinarily not subjected to flooding increases without some form of monetary or traumatic damages to the owner. Eroded yards, destroyed fencing or other yard appurtenances, building losses, and furniture losses are several examples of direct flood damages. Indirect damages can be equally important. The depreciated value of a flood-prone building, denial of an individual's driveway access during floods, or depreciated re-sale value of a house are indirect damages. One of the most difficult indirect damages to assess is the personal trauma accruing to an owner as a result of flash flooding over which the owner has no control.

(1-b) An increase in urban run-off peaking often leads to an increase in roadway flooding at some point downstream. While right of way flooding can be an inconvenience to some, it can be a matter of life and death to others when emergency vehicles are denied access. Fire trucks, ambulances and police vehicles should have unrestricted movement on public ways 100% of the time.

(1-c) Peaking increases usually bring velocity increases downstream which tends to damage public culverts, channels and roads if left un-checked.

Damages caused by sediment transport

(2-a) This is often a difficult type of damage to assess, yet is perhaps the most common variety in urban or suburban areas. Mud deposits in channels or pipe systems must ultimately be cleared at a distinct cost, and yet the obstructing effects and consequent higher flood stages accruing therefrom until clearing takes place are difficult to measure. Blanket mud deposition to downstream owner's yards are unmeasurable, yet usually insufficient to make the cost of litigation feasible. Unrecoverable damages are just as unfair as recoverable damages.

(2-b) On-site erosion of topsoil is an indirect damage to a future buyer in the sense that both the buyer and the community are deprived of rich vegetative growth in future years, or must go to the expense of bringing in a topsoil substitute.

CHAPTER 6

GOVERNMENTAL RESPONSIBILITY

Governmental agencies act on a number of issues every day in response to citizen or industry requests. Whether each decision is major or minor from the agencies' view, almost all are considered as major to the applicant for an approval, permit, or some other administrative action. It is important that the Water Management regulator not only exercises responsible judgement, but is perceived by those being regulated as being fair and competent. Government has a responsibility to strive for such a favorable perception, and several of the pivotal factors are listed as follows.

(1) <u>Technical Competency</u>
Since each development plan incorporates on-site considerations unique to all other plans, and since off-site considerations vary from project to project, it is not feasible for an effective regulatory program to rest on a detailed procedural manual. Even if it were possible to document every possible variable on- and off-site, the size of total documentation would be ponderous and probably unusable by industry and citizens. Then to, detailed instruction manuals on technical subjects tend to have the effect of curtailing professionalism. A well-administered Water Management program attempts to instill professionalism. In order to accomplish this end, the regulator must be technically competent in applying hydraulic engineering techniques to the program Principles and Objectives.

(2) <u>Integrity</u>
The development industry and the public at large will expect that such a broad based program shall be administered on an ethical level free of bias on issues not pertinent to storm water. The regulator should not be influenced by zoning issues not related to storm water, satellite political issues, applicant or opponent financial wealth, personal compatibility with an applicant or opponent, or any one of the myriad of issues upon which others base their alignment or opposition to a project. There are sufficient outlets elsewhere for project proponents or opponents.

(3) <u>Flexibility</u>
There are generally a number of views on how best to cope with any drainage situation. So-called established engineering methods may not always be the most effective and economic approach. It behooves the regulator to objectively consider the applicant's proposed methodology whatever it may be, albeit the method finally selected in any given situation will be subject to proof of workability. While the Water Management regulator must accomplish a definite task in plan review, it is to the advantage of all concerned to maintain a flexible approach at minimizing construction costs.

(4) Practical Knowledge

Not only is a practical knowledge of construction techniques and costs necessary, the regulator must have a reasonable knowledge of publicly-financed maintenance costs. It is a certainty that the developer will wish to minimize development costs, but this should not be allowed when the public will have to carry an undue burden of future costs as a result of improper economies during construction.

CHAPTER 7

MINIMIZING DEVELOPMENT PROBLEMS

Far from being and adversary proceeding, Water Management review is intended to regulate a development in a fashion acceptable to the community, and to minimize problems to the developer from litigation and builder-citizen complaints. There are problem aspects which the developer can avert through implementation of the following:

(a) Utilize a competent professional
When a developer retains a competent, experienced engineer, most water-related problems should not occur. Particularly during the construction process, there is no substitute for thoroughly detailed planning.

(b) Thorough project cost projections
Most experienced developers recognize the importance of compiling accurate job cost estimates during the feasibility study stage. Those who do not are faced with the temptation to under-cut expenditures on drainage items. A carefully estimated, profitable job for the developer can be an attractive job from the perspective of both government and the public, but a project on which the developer is losing money is rarely desirable to the community at large.

(c) Thorough supervision
Some developers tend to "broker" all aspects of a project from engineering through construction and lot sales. This can be effectively performed, but not often. If siltation and drainage problems are to be avoided, someone must coordinate all phases of the project with not only the developer's agents, but house builders or lot buyers as well. Subdivision plans can be carefully followed by the development contractor only to have subsequent disruptions to the drainage patterns by builders who have not been instructed as to lot grading requirements. It is not realistic to expect government supervision of the various elements which comprise a successful private project. Only the developer can perform this.

(d) Cooperation
There should always be a sound reason behind governmental controls and Water Management is no exception. A cooperative spirit by all concerned is not only helpful during construction drawing preparation, but in overcoming the numerous small problems which surface during construction.

However mundane these elements may appear, they are the most commonly violated which then produces water problems.

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CONTENTS

DEFINITIONS

NOTE: Refer to definitions listed in Part 1 of this series.

<u>bee-hive grates</u>- grates in the form of a truncated cone, truncated pyramid, or semi-sphere.

<u>critical flow</u>- discharge at minimum energy.

<u>discharge hydrography</u>- discharge from a watershed at time increments throughout the run-off period.

<u>energy dissipator</u>- any device used to reduce high velocity flow to an acceptably low velocity to prevent downstream damage.

<u>freeboard</u>- the vertical distance between design water surface and top of dam (as used in this publication).

<u>gabon</u>- rock-filled wire basket used for hydraulic or structural purposes.

<u>groyne</u>- a man-made impediment to flow placed near one bank of an open channel in such fashion as to deflect flow from that point, thereby inducing accretion.

<u>high water mark</u>- field determination of flood peaks at specific locations.

<u>impervious surfaces</u>- asphalt, concrete or any other surface which does not admit measurable infiltration.

<u>pervious surfaces</u>- earth or other porous materials which admit measurable infiltration.

<u>reach</u>- a length of open channel or pipe system.

<u>relief</u>- change in elevation.

<u>routing</u>- the computation of inflow, change in storage, and outflow through a storm water retention facility.

<u>run-off factor</u>- that percentage of precipitation which reaches the drainage system of interest.

<u>swellhead</u>- the vertical difference between headwater and tailwater through a flowage obstruction.

<u>thalweg</u>- the low point in any stream cross-section.

<u>unitgraph</u>- a discharge hydrography which accrues from one inch of run-off in a specified time frame.

CHAPTER 1

INTRODUCTION

This booklet attempts to provide technical data of use to those performing storm water management design in urban areas and to furnish generalized data for officials and land developers who share an interest in the subject. Hydrology and hydraulics can be a complex topic involving far more sophistication than that described herein, but experience has shown the need for an elementary reference for engineers and laymen who must pursue other aspects of their professions on a daily basis and of necessity must forego intensive study. This is difficult to accomplish because of the diverse backgrounds of all those who have an engineering, economic, or social interest in the ramifications of storm water management. Some readers will have had no engineering background in hydraulics while others may be accustomed to little more than applications of the "Rational" formula. Still others will be well-versed in both theoretical and applied hydraulics. In order to reach a middle ground of understanding, certain approximations will be reasonable to apply and will not introduce serious errors in the majority of urban problems. Usually the larger the land mass of interest becomes, the greater the complexity of problem solution. While the basic data described herein continues to apply, the reader is encouraged to consult more detailed publications.

The hydrology and hydraulics of urban areas is a rapidly evolving field of engineering which is distinct in itself and there are a number of people currently conducting detailed investigations which will constitute important contributions to the profession. The author takes note of several of these efforts and recognizes that there are undoubtedly others unknown to this writer which may have considerable merit.

In addition to theoretical study, the reader is urged to complement this data by diligent study of field characteristics of storm water behavior, both in overland flow movement and hydraulic behavior in open channels. The designer of excellence possesses the dexterity of judgement deriving from visual experiences just as much as from theoretical knowledge. The designer of worth must also have a practical knowledge of construction and land development methods in the vicinity of any particular project design.

CHAPTER 2

GENERAL ELEMENTS OF URBAN-AREA HYDROLOGY AND HYDRAULICS

(A) RAIN-FALL CHARACTERISTICS

Hydrology includes all aspects of precipitation prior to entry into a defined drainage system. This includes the study of rain-fall, infiltration, evaporation, transpiration, and the overland movement of water prior to reaching a defined system. Hydrology also concerns the study of snow-melt. In most urban areas, evaporation, transpiration and snow-melt characteristics have very little impact on the majority of flooding events. Accordingly, this chapter will dwell only upon the elements of rain-fall, infiltration and overland movement of run-off.

Hydrologists study natural rain-fall phenomena and are able to conclude certain characteristics based on weather records. As a general rule, the accuracy of their observations are directly related to the length of time detailed records have been maintained. Although many universities, businesses and local governments have installed rain gages and maintain records of varying quality, the United States Weather Bureau is by far the leading authority in the collection and analysis of rain-fall data. In some climatological regions the Weather Bureau has compiled over one hundred years of record, but in other regions their records are substantially less. An analysis of records for each region has enabled compilation of the Weather Bureau's "Rain-fall Intensity-Duration-Frequency Curves" for public use. This data is commonly termed frequency curves. An example of these curves for the three regions of Louisville, Ky., Lexington, Ky., and Nashville, Tenn. are portrayed on Plate A-1, Appendix A in Part 3 of this series. The following is an example of one use for these valuable curves in interpreting local rain-fall events.

The 100-Year frequency curve for Nashville indicates that 60 minutes after the start of rain-fall 2.95 inches of rain will accumulate. In formal terms, this means that based on the period of record, a rain-fall accumulation of 2.95 inches 60 minutes after the start of the storm may be expected to be equaled or exceeded at least once every one hundred years. By way of further explanation, let us say that only 2.46 inches of rain had accumulated during the same storm, but at time 50 minutes. This would be equivalent to the same rate of 2.95 inches per hour (50/60 times 2.95 equals 2.46) but at time 50 minutes instead of 60 minutes. Perusal of the Nashville curves indicates that a 2.95 inches per hour rate at 50 minutes falls on the 50-Year curve. Thus, in this example rain-fall event, we observe a 50-Year frequency at 50 minutes and a 100-Year frequency at 60 minutes but with an identical rain-fall rate at both times.

Another rain-fall the following month could produce an accumulation of 2.20 inches at 30 minutes, or a rate of 4.40 inches per hour (30/60 times 4.40 equals 2.20). Once again referring to the Nashville curves, this would be a 100-Year frequency at 30 minutes. Thus, in these two example storms, two 100-Year events took place within one month. It is within the context of these curves for a 100-Year event to take place several times a year, but with the 100-Year intensity taking place at different times with each rain-fall period. That this actually happens can be verified by the installation of multiple rain gages in an urban area and recording data at five minute increments. So-called 100 Year intensities are not as infrequent as the un-informed layman commonly believes. and should not be considered as unreasonable criteria.

Weather cycles in Kentucky have been observed to commonly produce short duration. high intensity storms in the summer months during the winter months. Of course there are exceptions, but the summer-type storms are those which usually produce the serious urban floods along small channels and pipe systems. A storm lasting only ten minutes but with a very high intensity often produces far more damages than a 24-hour storm of lower intensity, even though the longer storm may even have greater accumulations of rain. It is always of paramount importance to note the length of time during which the rain occurs in addition to rain-fall amount. Without a time-quantity relationship, it is not possible to relate any historical or design storm to that storm's severity.

Recent research by notable authorities has indicated a change in weather cycles over metropolitan areas as those areas expand. In particular, the summer-type of high intensity storm tends towards higher intensities. One explanation for this phenomena focuses on increases in impervious areas which. in turn, accelerates rising currents of air during hot periods. Further research in this area of interest is needed for amplification of data regarding any such trend nation wide.

The author suggests that urban storm water management designers utilize either one of two hydrologic bases for any particular project. (1) A theoretical Weather Bureau curve of the local government's selection. or (2) a historical storm of high frequency. Even though theoretical curves may be adjusted in the years to come because of new data, they are well-founded in fact. Historical storms of record also have considerable value in view of community resident tendencies to relate the impact of any new project to historical events. Some researchers, designers and agencies perceive value in re-distributing the rain-fall in a theoretical storm. yet this author has observed that such an approach greatly confuses the non-expert and is viewed by many as a "play" on numbers. On smaller urban projects. there is more to be lost than gained by re-distribution of a theoretical rain-fall.

(B) DETERMINATION OF RUN-OFF

Analytical conversion of rain-fall to run-off can be difficult owing to the many variables in watershed topography. A number of methods have been devised to determine the shape of run-off curves but, in this writer's opinion, no one method is applicable to all watersheds with any degree of reasonable accuracy. Where accuracy is vital, the designer should always select at least two methods which experience has proven applicable to the watershed's characteristics and compare results. Yet one method of run-off derivation is sufficient where a somewhat lesser extent of accuracy is needed, but even then it is crucial that a proper methodology be used.

What is run-off? Designers and public agencies customarily consider it sufficient to determine only the peak amount of flow in a channel or pipe system with a given design rain-fall. Their reasoning is founded on the assumption that, if a culvert or bridge opening is sized to pass the storm's peak rate of discharge, there is not need to determine the shape of the discharge curve before and after the instant of peaking. There is nothing improper with this line of reasoning for those who have no responsibility to protect downstream areas from flood aggravation and whose interests are limited to the particular waterway opening in question. However, as shall be noted in a subsequent chapter "STORM WATER RETENTION", a knowledge of the shape of the entire run-off curve is vital to water management design. Accordingly, this chapter will not dwell on the commonly used "Rational" formula for determining the run-off peak aside from making one observation. The "Rational" method has been used and Miss-used to the point of flagrant abuse in urban areas merely because it is a simple method. More frequently than not, "Rational" solutions err well beyond the bounds of reasonable inaccuracies.

Run-off is actually the entire amount of rain-fall (or snow-melt) which reaches a defined drainage system after subtractions attributable to infiltration, evaporation, localized pondage and other less important factors. Even should rain-fall remain constant, run-off at any stated point on a watercourse would vary with time because of a constant shift in infiltration, evaporation and pondage characteristics during the life of a storm. But the variation in run-off at one point becomes even more pronounced when rain-fall rates themselves vary with time as they usually do behave. In small urban watersheds (up to about 2,000 acres), the amount of run-off is strongly influenced during the first hour of rain-fall by such factors as steepness of topography, the amount of vegetation, the extent and location of impervious areas, the extent of upstream obstructions or depressions in the terrain which impedes flow, and the general efficiency of the watershed's internal drainage conveyance system. Studies in some urban areas have indicated that run-off has tripled in instances where un-developed watersheds have become saturated with developments of varying descriptions. Even quadrupling of run-off is not un-common where the upstream area is totally developed with commercial areas with almost complete impervious surfaces.

The foregoing is not meant as a substantive and thorough commentary on developmental impact on run-off and of the many factors which should be considered in projecting run-off, but rather to indicate that run-off derivation can be a complex matter to even the best of experts in the field of hydraulics. There have been notable researchers in this area of interest.

(B) DETERMINATION OF RUN-OFF (continued)

Some methods, while providing important contributions to the academics of hydraulics, are unfortunately too complex for use by the overwhelming majority of engineers. Other methods are reasonable accurate for certain watershed applications (because they were derived primarily from test data in certain types of watershed) but have little value when applied to other watersheds of a different character. On the other hand, some methods are so grossly simplistic as to have little value in portraying a run-off expectation accurately. By far the most accurate method would be the installation of stream gaging devices whereby run-off at the point of interest is monitored and then theoretically translated to another rainfall or changed watershed characteristic, but this method obviously requires an expenditure for test data and personnel, and requires a considerable amount of time to procure the necessary data. Another method which yields highly accurate results is that of the physical model (as opposed to a computer model), but this is not only time consuming but very expensive.

This author expects the problem of expensive and time-consuming run-off methodology, inaccurate methodology, or non-applicable methodology to be overcome for the most part within the next several years as more and more qualified researchers embank on realistic derivation methodology which can be applied by nonexpert and still yield reasonable results. This, however, does not help the reader for the time being. As an aid to those who do not care to enter into this area of interest exhaustively, the following sub-paragraphs describe a few methods which the author believes encompasses most urban run-off problems. The methods described are not intended to imply that other methods are not also applicable.

DISCHARGE HYDROGRAPH - INSTANTANEOUS METHOD

In the absence of test data, this author has developed an easily applied theoretical method for determining the run-off hydrograph for small urban watersheds. Since this method assumes instantaneous run-off and as a consequence probably produces peaks somewhat greater than naturally occurs, it is considered inadvisable to use this approach for watersheds greater than about 50 acres in size. It should also be noted that this method is best applied to essentially developed watersheds which have primarily impervious areas and/or an internal drainage system which promotes rapid run-off.

By way of explanation, let it be assumed that the following characteristics exist for a small commercial area and it is desired to determine the shape of a 100-Year discharge hydrograph for a one hour period.

Size of watershed area equals project are..20 acres
Computed "C_m"factor..0.90
100-Year rain-fall at 5-minute increments-

TIME (min)	INTENSITY (in/hr)	ACCUMULATED RAIN (inches)	INCREMENTAL RAIN (inches)
5	9.60	0.80	0.80
10	7.56	1.26	0.46
15	6.36	1.59	0.33
20	5.52	1.84	0.25
25	4.92	2.05	0.21
30	4.40	2.20	0.15
35	4.01	2.34	0.14
40	3.70	2.47	0.13
45	3.47	2.60	0.12
50	3.26	2.72	0.12
55	3.10	2.84	0.12
60	2.95	2.95	0.11

Five-minute incremental rain may be converted to a mid-increment instantaneous run-off with the following relation:

$Q_{\text{instantaneous}}$ = C_m x incremental rain-fall accumulation x drainage area = C_m RA

= C_m x (1/12 x 1/300 x ave. incre. rain) x area

= constant x average incremental rain

For the example problem the constant becomes:

Constant = 0.90 x 1/300
x 1/12 x 20 acres x 43,560 sq. ft/acre = 217.8

A compilation of mid-increment instantaneous discharges yields the following hydrograph for the example problem.

TIME (min)	INCREMENTAL RAIN (inches)	AVE.INCREMENTAL RAIN (inches)	CONSTANT	INSTANTANEOUS DISCHARGE (cfs)
0	-	-	217.8	-
2.5	-	0.4	"	87.1
5.0	0.8	-	"	-
7.5	-	0.63	"	137.3
10.0	0.46	-	"	-
12.5	-	0.395	"	86.0
15.0	0.33	-	"	-
17.5	-	0.29	"	63.2
20.0	0.25	-	"	-
22.5	-	0.23	"	50.1
25.0	0.21	-	"	-
27.5	-	0.18	"	39.2
30.0	0.15	-	"	-
32.5	-	0.145	"	31.6
35.0	0.14	-	"	-
37.5	-	0.135	"	29.4
40.0	0.13	-	"	-
42.5	-	0.125	"	27.3
45.0	0.12	-	"	-
47.5	-	0.12	"	26.1
50.0	0.12	-	"	-
52.5	-	0.12	"	26.1
55.0	0.12	-	"	-
57.5	-	0.115	"	25.0
60.0	0.11	-	"	-

The absence of a straight line relation between time periods and the fact that run-off is never instantaneous are the most obvious approximations in this method. The computed run-off on the front end of the storm is higher than would occur while those near the end of the storm would be lower. Yet when applying the retention principle, these differences can have little impact on total problem solution as long as the drainage area to which the procedure is applied in not large.

This procedure is simple to apply and suffices in most small urban area problems where the watershed is below the 50 acre size. Undoubtedly greater accuracy can be achieved through future field research of the many factors which affect small watershed run-off, but greater accuracy will also increase problem solving complexities to those not adept in this realm of interest.

INSTANTANEOUS RUN-OFF FACTOR, "C_m"

The factor "C_m" in the foregoing INSTANTANEOUS HYDROGRAPH METHOD is intended for use in small urban watersheds which experience high intensity-short duration storms and where evaporation and localized pondage/valley storage have very little influence on run-off. the value of "c " is determined from the following relationship:

$$C_m = \frac{(C_n \ x \ A_p) + (C_i \ x \ A_i)}{A} + \frac{(A_p)}{(A)}$$

where; C_m = instantaneous run-off factor as modified by proposed construction

C_n = natural instantaneous run-off factor determined from natural topography. Values of 0.35 for rolling terrain down to 0.20 for flat terrain may be used in the absence of test data.

A_p = pervious areas in project.

C_i = instantaneous run-off factor for impervious areas. A value of 0.95 may be used in the absence of test data.

A_i = impervious areas in project.

A = total area in project.

k = coefficient reflecting the surface drainage efficiency, or re-grade, of pervious areas in the project. In the absence of test data, a value of 0.20 may be used for most re-grades of moderate to flat slopes.

This procedure is a logical progression of the commonly-used value "C_n" and has been observed through practical applications as a reasonable basis for modified run-off determination. Yet test data has not yet been compiled to firmly verified the suggested coefficient values. An example problem for a residential area, "Lake Ayre Estates", is depicted in Appendix B, Part 3. Plate B-1 portrays the subdivision plan and Plate B-2 portrays derivation of C_m.

COLORADO URBAN HYDROGRAPH PROCEDURE

A storm run-off derivation was advanced in 1969 by the firm of Wright-McLaughlin Engineers for the Denver Regional Council of Governments and was financially aided by the U.S. Department of Housing and Urban Development. This procedure is based on a unitgraph derivation which should be readily understandable by the majority of designers. This method, termed CUHP, will no doubt be modified somewhat in different geographic regions as more test data becomes available, but the basic effort constitutes an important contribution as a hydrograph methodology which is reasonable accurate under certain conditions and is fairly easy to apply. For information relative to the entire two volume study, Wright=McLaughlin Engineers should be contacted at 2059 Bryant Street, Denver, Colorado 80211.

An extract from that study describing CUHP is included herewithin Appendix "C", Part 3. This author suggests its' use for watersheds in the range of 50 to about 1,000 acres.

CLARK METHOD

The Corps of Engineers frequently use the Clark method of run-off derivation (after C.O. Clark, ASCE Transaction, 1945, Volume 110) for moderately large watersheds. This method is not complex but should take some time for the reader to develop proper familiarity. It involves a logical analytical approach to unitgraph derivation, but for any particular project the attenuation constant "R" should be determined from a known hydrograph or should be selected on the basis of considerable experience with comparable watersheds. A commentary on this method is found in Appendix "D", Part 3, Plates D-1 through D-9. This method has value for urban watersheds over about 1,000 acres in size. An example application is shown in Appendix "E", Plates E-4 through E-10.

DRAINAGE AREA PROPORTION METHOD

This procedure is applicable to any size watershed and is basically a relationship between a gaged location and an un-gaged location. If a designer is fortunate enough to have rain-fall/run-off records for a watershed of similar topography, size and developmental characteristics in relation to the watershed of interest, then it becomes a relatively simple matter to proportion hydrograph peaks and work backwards into a unitgraph.

DISCHARGE MEASUREMENT METHOD

This method is probably seldom used because of the expense and time required. Of essence to this method is the installation of one or more rain gages in the watershed of interest and field capability to measure stream flow during high intensity storms. With this data for several rain-fall events, a skilled engineer can work backwards into a unitgraph which may then be used to project a design run-off of interest. This method is the most accurate known to the author.

(C) OPEN CHANNELS AND PIPE SYSTEMS

One only needs to closely view drainage systems in most communities to understand that channel and pipe system design has frequently been of poor quality. Often poor system design is the result of ill-founded economies by either the local government or the developer, but many errors in judgement are the result of the designer's inexperience or lack of knowledge regarding all factors which should be considered in a proper design. This section is not intended to undertake this subject in depth, but rather to take note of several major points of interest influencing a total water management design.

PIPE SYSTEMS

Curiously enough, pipe systems are frequently designed so that their capacities may never be realized because of restrictive or insufficient inlets. Designers should always check the throat capacities of inlets to insure that design flows can actually enter the system.

Points of entry into pipe systems, even though of adequate size, frequently do not pass design flows because of the propensity for clogging of drop inlets or at headwalls. One can never insure that clogging will not occur, but the designer can greatly minimize this problem by the installation of bee-hive type grates where feasible and by the installation of properly designed trash racks. This author cautions against the use of trash racks where debris is not a major clogging factor, however, or where clogging may result in hazardous surcharge of the inlet condition and produce undesirable overtopping of facilities above grade. Unless the designer uses good judgement, a trash rack can create more problems than the one the rack is designed to prevent.

Engineers accustomed to sanitary sewer design work frequently make the mistake of opposing entry pipes into a manhole. During high velocity design rain-fall/run-off conditions, opposing flow has been known to negate discharge from both entry pipes. This is not a major factor in sanitary sewer design, but becomes a vital concern in dealing with the higher velocity storm water.

As will be noted in the following paragraphs, an open channel properly designed can be more economical and aesthetical pleasing than a pipe system.

OPEN CHANNELS

Rolling topography and gently meandering streams appear to have substantial sales appeal in residential areas. It is good practice to leave desirable streams in their natural state, or with perhaps slight modification, and afford the developer with an economy. However, a natural channel incorporated into final project design should have the characteristics of sufficient slope to preclude either accretion or erosion. Meanders often should be protected against bank attack. Either one of the factors of accretion, invert erosion, or bank attack can induce substantial maintenance costs to local government in future years. In addition, an easement should be impressed on all contiguous lands at least tot he 100-Year flood level to prevent un-authorized encroachments in the flood plain. The topography within the easement should be such to permit governmental access for future maintenance.

When an open channel is used in rolling terrain, residential record plats should not only reflect the 100-Year easement lines but should also indicate the flooding elevation on each riparian lot. This precludes the possibility that a builder might excavate a natural slope and construct walk-out basements or recreation rooms below the control elevation.

Excavated open channels should not have earthen side slopes any steeper than 2:1. Steeper slopes prevent reasonable attempts at maintenance operations. Channel bottom slopes in the longitudinal direction should not be flatter than 0.5% if at all possible, and even then the bottom should have concrete with finish grades staked at least every 25 feet to avert localized pondage areas. Concrete bottoms should have concrete returns on each side to prevent out-flanking of the bottom section. Earthen bottoms can be acceptable with slopes steeper than 1.0% up to a slope which will computationally generate erosive velocities, and then the bottom should b e of concrete or some other material which will withstand erosive effects. Plate G-2, Appendix "G", Part 3 portrays a design curve based on test data which will indicate stone size necessary to prevent dislodgement by specific stream velocities.

The use of open channels sometimes pivots on a matter of the developer's taste in esthetics versus economy. One factor which should always be considered is a particular area's probability of attracting litter or debris. Open channels tend to become depositories for the type of litter that a pipe system grate often prevents.

The point of emphasis in this section concerns the need for the hydraulic designer to carefully consider maintenance considerations in addition to pure hydraulic design of the channel. A poorly maintained channel is but one element in a total water management design, and if it malfunctions, total project functionality can be impaired.

(D) <u>FLOOD PROFILES</u>
The only precise methods to determine a flood profile for any stated discharge are the following:

(1) Field measurement of the flood discharge and setting high water marks.
(2) Hydraulic modeling.

Since both of the foregoing are generally impractical for the great majority of small urban design problems, the designer must seek practical methods which have a lesser degree of accuracy. The method one utilizes should be a function of the desired accuracy. Hydraulic engineers usually employ the following method for projects of moderate size.

(3) Computational backwater analysis

A backwater study can become extensive in field cross-sectioning, design time and total analysis expenditure, and unless the primary effort for the design rests on a hydraulically oriented project, (for instance a channel improvement) one is not likely to encounter those who are willing to fund extensive investigation of a "secondary" type of design feature. Where the accuracy tolerance is not severe, there is another procedure which is easy to understand and use by non-specialists. This is portrayed on Plates 11 and 12, Appendix "E", Part 3, and is descriptive of a channel rating at steady flow with negligible backwater effects.

(4) Channel rating at steady flow-negligible backwater.

This method is quite simple to use and involves the following:

(4-a) Determine a cross-section at a representative (or typical) cross-sectional location.

(4-b) Determine the thalweg elevation at some up- or downstream location, measuring the distance to the point of cross-section so that a slope is obtained for the channel.

(4-c) Obtain the desired design discharge (Q) from one of the run-off methods and assume that this discharge flows at a depth of your selection at the cross-section.

(4-d) The selected depth will yield a corresponding Hydraulic Radius "R" and Area "A" which may then be substituted in the Mannings's equation shown along with the previously computed Slope "S" to arrive at a corresponding "Q". If the corresponding "Q" differs from the known "Q", the depth selection was obviously in error. A comparison of the two "Q's" will enable a more accurate second trial.

(4-e) Generally after two or three quick trials, the designer obtains an assumed depth which is compatible with the discharge. This procedure may be repeated with additional cross-sections as necessary, but for short reaches in urban areas, it is usually sufficient to strike a profile through the computed point parallel to the thalweg.

In the Appendix "E" example shown, the author performed a slight variation from the foregoing sequence and merely assumed various depths, computed the "R", "S" and "A", and arrived at the corresponding "Q". If this is performed with different depths, a Rating Curve for the cross-section may be drawn. The Rating may then be entered with the "Q" of interest and the corresponding depth determined graphically.

The author cautions that this procedure is approximate and is predicated on the following:

<u>There is no backwater effect.</u>
<u>Steady flow exists.</u>
<u>The cross-section and slope are representative of the reach.</u>

For the reader who may inquire regarding the foregoing use of the channel slope as being equal to the energy gradient slope "S" in Manning"s equation, be advised that the energy slope attempts to attain "critical" value as a limit. Equal values are a reasonable approximation for steady flow conditions which exist at time of peak discharge.

Of course, where a downstream bridge, culvert or fill obviously controls flowage depth, the foregoing will not apply. Backwater methods should then be employed.

(E) DETERMINATION OF SWELLHEAD THROUGH AN OBSTRUCTION

Where the designer desires the rise in discharge profile (or swellhead) through a bridge or similar obstruction, several methods of computation are available. The Bureau of Public Roads and others have advanced noteworthy procedures which vary in complexity. However, for a very simple method within the grasp of most non-specialists an energy balance is usually of sufficient accuracy. The derivation for energy balance is shown on Plate E-13, Appendix "E", Part 3, and an example problem is portrayed on the following Plate E-14.

This method is approximate and does not involve an approach velocity (which invariably exists in channel flow) nor does it involve peculiarities of turbulence unique to every structure. However, it may be noted that approach velocity tends to diminish the swellhead computed by this method while turbulence tends to increase the computed value. It is a simplification to assume that these effects cancel, but once again this is an acceptable approximation for most small urban problems.

CHAPTER 3

STORM WATER RETENTION

(A) THE CASE FOR RETENTION

If all urban drainage systems were designed and constructed to convey the 100-Year floods generated by fully developed watersheds, there would not be much of a flood problem. But most communities have constructed their systems to pass very low frequency storms under developmental conditions that prevailed at the time the systems were built. Some may argue that local governments have been short-sighted, yet it may also be argued that historical governments performed to the best of their financial capabilities each time a piped or open channel element was added to the total system. It is not the intent of this chapter to argue the merits of each point of view, but rather to take note of the fact that many urban drainage systems are not deficient.

Now comes the developer of a school, shopping complex, residential area or some other facility of which the community has need. Impervious areas and project re-grade greatly accelerates storm run-off and, if unchecked, adds to downstream flood stages. Local governments and the developer of any particular project are confronted with three options. (1) The run-off may be allowed to increase and aggravate damages to passive land owners, but this exposes both government and the developer to needless litigation by damaged parties. Irrespective of potential litigation, this course of inaction speaks little of a government on whom its' constituency relies for basic protection. (2) The entire downstream drainage system can be improved to accommodate increased flows. This procedure is encouraged on a long-term basis, but it is hardly practical to improve an element of size each time a building permit is issued. (3) Storm water retention may be employed to temporarily restrain run-off leaking generated by the new project. This alternative is by far the most economically feasible and legitimate of the three options in the majority of problem areas.

While complete storm water management involves drainage system improvement, diversion of watersheds where properly performed, and many other items, storm water retention is a major element which should be considered. The advantages of retention far out-weigh the disadvantages in most instances, provided retention facilities are properly designed. Several major advantages will be described in the following paragraphs. For the reader who has studied this topic elsewhere and may take issue with the term "retention" in lieu of the word "detention", be advised that the author prefers the former to avert confusion in communities where the penal institution is referred to as a detention facility. Since both words have essentially the same meaning in hydraulics, it has proven preferable to use "retention".

Retention of storm water implies that run-off in excess of natural, or even greater amounts, is being temporarily restrained to prevent either flooding or flooding aggravation. From a study of the preceding chapter on run-off, it may be seen that the top portion of a run-off hydrograph generally involves relatively little water volume, and of course it is this top portion of the hydrograph which imposes downstream flood damages. Usually it is far more economical to restrain this relatively small volume than to improve lengthy sections of the downstream drainage system to accommodate the same amount of flow. There will of course be exceptions where the downstream system is short and potential damages nominal, and where this occurs retention may have lesser benefits.

A later section of this chapter will comment on retention criteria which suggests that a new project be held only accountable for restraining its' own run-off increases as opposed to being held responsible for natural run-off as well. Yet there will always be circumstances favorable to specific developers for retaining additional amounts of storm run-off and thus voluntarily reducing natural downstream flooding. Where such favorable circumstances develop, storm water retention has no economic peer and becomes an invaluable community resource.

Another section of this chapter will dwell upon variations in retention design which net both the developer and the community with dual usages. Recreational areas, "green belts" and space buffers are but a few of dual usages of storm water retention areas. Whereas channel improvements are usually devoted to hydraulic purposes exclusively, retention areas which accomplish the same purpose can provide other enhancements.

Off-site siltation of storm piping and open channels, yards and other elements of the public and private community is a continuing problem to most communities, and retention facilities frequently serve as a temporary silt trap during project construction. Construction mud control is not only good for the local government and downstream citizens, but frequently serves to minimize the developer's exposure to litigation or off-site expenditures relating to mud cleaning operations. It has been observed that retention areas often serve an even more important silt control function than that of flood control, and this advantage should not be minimized.

The total case for retention is good provided the designer and the approving authority recognize sound design methods. Versatility and the application of basic techniques are essential.

(B) RETENTION CRITERIA

Each community adopting storm water management must develop its' own criteria which reflects community sensitivity to flooding, the adequacy of the existing drainage system, and the character of topography. The following criteria has been developed by the author on the basis of experience with metropolitan areas which have acute citizen attitudes towards flooding and whose residents do not wish to sustain flooding aggravation from new construction. For the most part this criteria has proven acceptable to the great majority of citizens and has, as of this date, precluded any rational attempts by community residents to obtain financial damages from developers complying with this criteria. To the best of this author's knowledge, this methodology has been perceived by the great majority of the development industry as being equitable. It has been utilized in one metropolitan area for a period of five years and another for about four years and has appeared to stand the test of acceptability by reasonable people and has proven functional. Yet it is recognized that community attitudes differ and community facilities differ. For these reasons the following criteria may be viewed with a flexible attitude.

RETENTION VOLUME-

Downstream areas shall not be subject to any flood aggravation as a result of new construction during a 100-Year frequency rain-fall event. Where improvement of the downstream storm water system is not feasible, retention volumes should be equal to the change in run-off generated by the 100-Year storm for a time period equal to whichever of the two following conditions apply:

(1) Where the drainage outlet for the new construction is, in its' entirety, a surface-gravity system, the change in run-off during the first one hour of the 100-Year storm will apply. (The one hour standard is applicable to most areas because urban storm water movement will usually pass most points of damage within 60 minutes after the start of the storm. A time less than one hour might be applicable, but would involve computational proof of travel time and can introduce confusion to those non-specialists working with urban development.)

(2) Where the drainage outlet for new construction is either a pump and force main or a sink-hole situation, the change in run-off for three hours will apply. (The three hour time frame has as its' basis the authors observation and experience that the first three hours of a 100-Year storm are generally the most critical in non-surface, gravity systems. Thereafter rain-fall rapidly diminishes and most pump-storage and sinkhole situations are able to recover. This is not always the case, but is sufficiently so to enable the three hour time as a general standard).

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RETENTION OUTLET SIZE AND LOCATION

The retention device may restrain on-site storm water from entering a drainage arterial or may be located and designed to allow arterial storm water to drain into the basin, but in any event the retention facility should prevent flood aggravation during 100-Year storm peaking at downstream points of damage. There may be a design exception to the foregoing noted in the following paragraph.

While retention facility volume is predicated on the change in run-off for the 100-Year storm, it may be to the advantage of local government and downstream owners to use the fixed volume for reductions of lesser frequency storms. Where this preference prevails, the outlet may be sized for any low flow release rates desired but with a corresponding increase in spillway capability for a total low flow/spillway release rate equivalent to the 100-Year storm.

The combination low flow and spillway release capability should equal at least the 100-Year storm with a minimum spillway freeboard of one foot. Large structures which fall within the purview of KGS 151.250 (Commonwealth of Kentucky) must satisfy the regulatory procedures of Kentucky's Division of Water Resources pertaining to dams.

RETENTION FACILITY MAINTENANCE

The retention facility must be designed for ease of maintenance and there must be a designated entity for periodic maintenance. There should be a recognized procedure for governmental inspection to insure compliance with maintenance requirements and, if the maintenance entity is non-governmental, there should be a penalty provision for failure to properly maintain the facility.

LEGAL DESIGNATION
The retention facility should be circumscribed by a metes and bounds for inclusion in either a retention easement or a nonbuildable lot. If a separate lot or specific easement is impractical, as would be the case with a retention vault, the facility purpose and maintenance requirements may be described in the deed.

The intent of appropriate legal designation is to insure that successors in title will have reasonable notice of the retention facilities purpose and maintenance provisions.

(C) VARIOUS RETENTION DEVICES

There are a number of storm water retention devices which may be employed in different circumstances, but all available options should meet the following needs.

(1) The measure must function properly to hydraulically prevent off-site flood aggravation.
(2) The measure must be legally identifiable to prevent usurpation of its' intended function and to establish maintenance responsibility.
(3) The measure must be easily maintained.
(4) The measure should be reasonably safe.

The design of any specific retention measure is a function of out-fall characteristics, on-site topography, dual usage potential and compatibility with other on-site features, ease of maintenance, and a host of other considerations which may vary from job to job. It is not practical to attempt a description herein of all the possibilities or combinations of possibilities, but the following will indicate several basic retention facilities.

"NATURAL OPEN AREA-DRY BASIN"
This type utilizes natural terrain for economic or esthetic reasons and consists of a designated storage area with a throttled outlet for effective retention. The hydraulic throttle may be a restricted culvert under an elevated roadway, a man-made dam warped and blended with the terrain for a pleasing appearance, or some other restriction which, when surcharged, will not backflood any on-site feature subject to damage.

"MODIFIED OPEN AREA-DRY BASIN"
This consists of an excavated area which is readily recognized as a retention facility to the casual observer, but otherwise is similar to the foregoing type.

"NATURAL OPEN AREA-WET BASIN"
The "Wet Basin" designation implies the use of a formal permanent pool. this type of basin is usually employed for esthetic reasons and generally drains a sufficient sized watershed to preclude substantial evaporation effects during the summer months. However, this is not always

the case when low flow augmentation is used to off-set evaporation. Aside from esthetics, this type is often beneficial for elongated projects and flat slopes wherein a rise in storm piping or channel elevations are critical and the flat permanent pool saves several vital feet in relief.

"MODIFIED OPEN AREA-WET BASIN"

This type consists of a man-made excavation with a permanent pool and is usually employed in an urban project where fountains or other permanent pool appurtenances are desirable.

"LATERAL BASIN"

This type of retention facility is basically an over-sized drainage ditch or channel with appropriate checks to insure temporary pooling of storm water. This measure is particularly appealing in broad, flat areas. and primarily in industrial park sites where run-off is intense and substantial lengths of pooled areas present a minimal of safety hazard.

"RECREATIONAL BASIN"

This is usually similar to the general category of either "Natural Open Area" or "Modified Open Area", but chiefly alludes to the dual usage of tennis courts, ball fields, or some other recreational facility which is not likely to be used during intense rain-fall periods. There should always be at least one percent slopes in these areas to promote rapid drying.

"SUB-GRADE BASINS"

Either a sub-grade vault or over-sized storm piping with throttled outlets would be examples of this category. Where open areas are at a financial premium, the developer may elect to use fairly costly sub-grade facilities. This usually applies to commercial areas.

NON-DESIRABLE TYPES

Some areas employ roof top storage of storm water, but the author finds this method objectionable for several reasons. First, owners generally have a difficult time maintaining a water-tight roof without deliberate storage, so their problems generally escalate where roof top storage is built in to the scupper system. Secondly, it is difficult for governmental inspectors to determine when the roof top retention facility has been modified without authorization. this author also considers roadway pondage undesirable as a deliberate retention measure. It was previously mentioned in this series that deliberate pondage in roadways impede the movement of emergency vehicles. Limited parking area pondage may be acceptable insofar as the depth of inundation during flash flooding would not damage packed vehicles, but extensive pondage exposes unwary automobile owners to needless damages.

(D) STORM ROUTINGS THROUGH RETENTION

Storm routings should be performed through any basin for accuracy of design, but as a matter of everyday practice it appears unnecessary for basins which must store less than about one acre-foot of flood waters. A later section will dwell upon a quick design approach for these smaller basins. But for storage greater than about one acre-foot, the following is one method for storm routings.

INFLOW-STORAGE OUTFLOW

After the retention designer has secured an inflow hydrograph from one of the preceding methods and properly sized the basin, it becomes necessary to route the storm through the basin with at least one, and generally several low flow/spillway outlet combinations to insure that total outflow does not exceed natural outflow. Appendix "F" includes Plates F-1 through F-5 which is an example of a storm routing. The procedure is simply one of balancing inflow, the change in basin storage, and outflow during each increment of time during the storm. If on the first trial selection of low flow/spillway sizes the outflow peak exceeds the natural run-off peak, the outlet combination must be revised accordingly.

It should be noted that there is a graphical solution to a storm routing problem, and there are also programs for large data processing systems which facilitate lengthy designs, but the author suggests that engineers who use the several 'desk top' type of computers currently on the market might consider programming for that purpose. The 'desk top' computer promotes a rapid design while, at the same time, allows the designer a close rapport with the design in progress without loosing accuracy.

(E) OUTLET WORKS DESIGN

A proper storm routing procedure will result in sizing of the low flow/spillway combination. It was previously mentioned however that basins smaller than about one acre-foot in temporary storage size may be designed with an approximate method without routing. The following is descriptive of that method.

SIZING OUTLET-APPROXIMATE METHOD

the entire purpose in storm water retention is to prevent off-site flooding aggravation, or in other words, to insure that the project of interest will not discharge more flood peaking than that which occurred under natural conditions. For small basins of less than one acre-foot of storage, the outlet may be sized for natural run-off from the watershed in question. The reader has noted the previous section on rain-fall characteristics and observed the small, almost negligible difference between the 100-Year storm and small frequency storms in the short time frames near the beginning of rain, say at time five, ten, and fifteen minutes. It is during these early time frames that small projects generally peak and that the total accumulation during those times are nearly the same irrespective of the design storm frequency. For this reason, it is suggested that the outlet be sized for the natural watershed and the storm frequency generally used in the community drainage system. 100-Year inflows are then impounded and surcharge the low flow will accommodate under surcharged conditions. Plate G-1, Appendix "G". Part 3 is included as an aid to those using either this method or the more detailed and accurate storm routing method.

OTHER OUTLET DESIGN CONSIDERATIONS

There are at least two other main considerations in outlet works design. First, the outlet must be durable and capable of withstanding continued erosion or vandalism. Concrete low flow pipes with anti-seep collars should be mandatory when heads over four feet are generated on the inlet, and concrete spillways are desirable when those spillways are subject to frequent use. Second, the outlet velocity under surcharge inlet conditions should be checked for erosive attack on the receiving channel. Where this possibility exists, an energy dissipator should be employed. A following chapter will dwell on this aspect.

Trash racks can be a major consideration, as can open basin fencing, but these elements are subject to specific site circumstances.

(F) RETENTION BASIN MAINTENANCE

It was previously noted that every basin should be legally defined in either a deed or on a record plat, and that the maintenance entity should be specified. It is preferable for local government to assume maintenance operations, but this can be impractical for communities in short supply of funding capability. If a private maintenance entity is specified, the requirements should be set forth in a separate maintenance agreement with a cross reference on the plat or deed.

The maintenance agreement should dictate that vegetative growth should not exceed five inches in height, that all foreign objects and debris are to be kept removed from the site, and that periodic maintenance is to be performed to insure the hydraulic and structural integrity of the project. Structural and landscaping intrusions onto the site are not allowed without the written approval of the regulating agency.

CHAPTER 4

SINK-HOLES AS DRAINAGE OUTLETS AND RETENTION AREAS

A sink-hole is a depression or cavity in the terrain caused by the movement of surface water towards a subterranean drain. A sink may have an exposed outlet or may be a highly pervious earthen depression which transmits surface water to the underground outlet. Sinks are particularly dominant in the Bluegrass and Pennyroyal regions of Kentucky where underlying strata is composed of limestone or other highly erodible material, but they also are not uncommon in other areas of the state. Historical developers and local governments have found it convenient to use sinks as formal drainage outlets, and in many cases encircled sink-hole areas to such an extent with development as to make abandonment of the sinks impractical without considerable expenditure for land acquisition and storm piping.

Sink-holes are very undesirable as formal drainage outlets for a number of reasons. They become plugged with silt or debris and they are prone to collapse in subterranean areas which are not subject to control by local government. Another undesirable aspect focuses on outlet capacity determination.

It is impossible to determine a generalized rule for sink-hole discharge capability. Each sink behaves differently from all other sinks and discharge from each is a function of the unknown subterranean streams. Underground stream-flow is irregular, varying in cross-section, and subject to the vicissitudes of subterranean erosion, channel collapse, backwater effects and varying inflow from many points. This is impossible to determine without specific inflow-outflow tests at each sink in question. Sink-holes should be used as an integral part of a storm water system only where no other outlet is feasible, and even then specific criteria should apply to their use. The following lists critical elements in sink-hole criteria.

DRAINAGE OUTLETS-
There should be as little disruption of the immediate environs of the sink as possible and the placement of mechanized equipment near the subterranean drain should be avoided. All construction work in this area should be by hand, consisting of the following:

(1) Flow exiting from new culverts or focalized points of inflow to the sink area should be controlled by concrete or rip-rap to the drain so as to preclude erosive damage to the outlet.

(2) A steel grate of adequate proportions should encase the drain to prevent stoppage from debris.

(3) The immediate environs of the sink should be fenced to minimize vandalism.

The following paragraph will dwell upon the use of a sink as a retention area, but it should be noted that computational, no outflow should be assumed from any sink unless verified by field tests during rain-fall events. It maybe apparent that a particular sink is functioning the time of project design, but the extent to which it functions must be accurately determined.

RETENTION AREAS

When using a sink-hole area as a drainage outlet and retention area, it has been previously noted that a 100-Year storm over a three hour time span is desirable. There should be sufficient retention volume around the sink to contain the entire run-off from this storm with no outflow, unless field measurements corroborate an acceptable outflow rating for the subterranean drain. This retention area should be defined as an easement or non-buildable lot, and the maintenance entity should be specified.

This author has observed enough sink-hole malfunction, either through stoppage from surface silt or debris or through underground collapse, to suggest that sinks designed as a part of new projects should have an initial emergency plan for discharge relief in the event malfunction occurs. Such a plan may be preliminary in nature, but should verify that either a surface channel, storm piping, r pump station and force main is a feasible alternative, and should specify the entity which would perform this emergency relief construction in the event of sink malfunction.

FIELD MEASUREMENT OF SINK-HOLE DISCHARGE CAPABILITY

Either a staff gage or continuous recorder can be mounted at the low point of the sink. During the run-off event, the variation in ponding level can then be recorded. If the sink environs have been mapped and inflow either recorded or determined computationally through one of the foregoing hydrograph procedures, then a storm routing process will yield the sink outflow rating.

CHAPTER 5

ENERGY DISSIPATORS

Point discharge from a culvert or retention basin often conveys high velocity flow which may tend to degrade the receiving channel or adjoining property. Generally, any velocity over about five feet per second will have an erosive effect on earth. In order to protect the contiguous property, it may be desirable to install some form of revetment on the area in question. If, however, the adjoining tract is being put to some use incompatible with revetment, it them behooves the installation of an energy dissipator on the point discharge facility.

There are a number of energy dissipators which apply to differing situations. The designer may choose to select any one of several effective types, but for economy, ease of design, and a wide range of applications on small, urban projects, the impact basin has met with wide acceptance. Plates G-3 through G-5, Appendix "G", Part 3 portrays an impact basin configuration tested by the Bureau of Reclamation. A chart is shown which enables the designer of a relatively small project to select the proper geometric for impact basin detailing.

There are instances where rip-rap may serve effectively as a dissipator, but this primarily applies where discharge is relatively low. Plate G-2, Appendix "G", Part 3 portrays a curve defining stone size necessary to withstand stream velocities.

CHAPTER 6

100-YEAR FLOODING

(A) DETERMINATION OF 100-YEAR FLOOD LEVELS

Chapter 2 (D) portrayed a simplified method of determining the 100-Year flood level in certain circumstances where formal flood mapping was not available. Every community should have a comprehensive flood plain map which not only includes major rivers and creeks, but all of the lesser tributaries as well. Subdivision drainage easements and roadside ditches are often neglected as sources of flood damages, but in fact these drainage elements are the cause of more extensive and frequent flood damages than hat on riparian properties adjacent to main watercourses.

Sink-hole areas, in particular, are frequently slighted in flood plain mapping determinations. Chapter 4 indicated that flood easements should be impressed around sinks to an elevation which would accrue from the first three hours of a 100-Year inflow, assuming no outflow unless verified from field measurements.

Subsequent to the 100-Year flood level determination, an additional one foot of freeboard should be used for all critical damage points in designing a new project. Without this margin of safety, wave wash and flow impediments from temporary accumulations of debris can quickly nullify the computed 100-Year level.

(B) FLOOD PREVENTATIVE MEASURES

Existing flood prone buildings often have characteristics which lend to flood-proofing. Such measures are rarely desirable in a new structure, but may be the only way to minimize or prevent periodic inundation of a floor level. Typical of flood-proofing devices are beaming the rear yard in conjunction with a low-flow outlet/check valve and sump combination, construction of area-way wingwalls with a removable bulkhead, or similar measure tailored to meet a specific need. These methods, however, can become impractical when flood depths are extensive.

Community financed channel improvements or storm water retention facilities should always be encouraged as a long range solution to flood prevention.

CHAPTER 7

USEFUL CONSTRUCTION PRODUCTS

There are a number of hydraulically oriented construction products which are either new or have not yet circulated into particular urban areas and which may have considerable value to the economy, functionality or durability of specific projects. It is emphasized that no product is ideal for all applications, but the designer should be aware of as many applications as possible to effectively serve both the project and community interest.

Gabions-
A gabion is a galvanized wire or polyethyline-coated wire basket which is field assembled and filled with stones of appropriate size. Gabions are manufactured in various sizes and shapes and are used as retaining walls, channel linings, groynes, ditch checks, and other hydraulic structures. They have been in use for many years in Europe and Canada, and are used extensively in certain areas within the United States. They have both advantages and disadvantages for every application, but several of the chief benefits derive from economy of placement with unskilled labor and esthetic appearance if installed properly.

Fabric-formed mats, pumped concrete-
Where existing channels must be rapidly protected or reconstructed and de-watering or construction access is a problem, at least one manufacturer produces a double-layered fabric with interconnections which can be rolled down a channel slope and filled with pumped concrete.

Mulch-netting combinations-
One manufacturer has developed a bio-degradable netting interwoven with paper mulch strips which serves the dual function of mulch and net. After seeding and fertilizing of slopes, this fabric may be rolled down a slope with reasonable assurance of establishing a vegetative cover without re-working eroded areas. This fabric is also very effective in maintaining moisture on the seeds, even in very dry periods. Generally no watering is necessary as in the case with sod or some other types of mulch.

Sloped headwalls-
While not a product per se', sloped headwalls may frequently be used in lieu of vertical headwalls as an economy and as a hydraulic performance enhancement.

All of the foregoing are but several of the many products or methods being used with which the designer should be familiar.

STORM WATER MANAGEMENT SEMINAR

PART III

BY
DAVID L. DAUGHERTY, P.E.

SPONSORED BY
CITY-COUNTY PLANNING COMMISSION OF WARREN COUNTY
OCTOBER, 1976

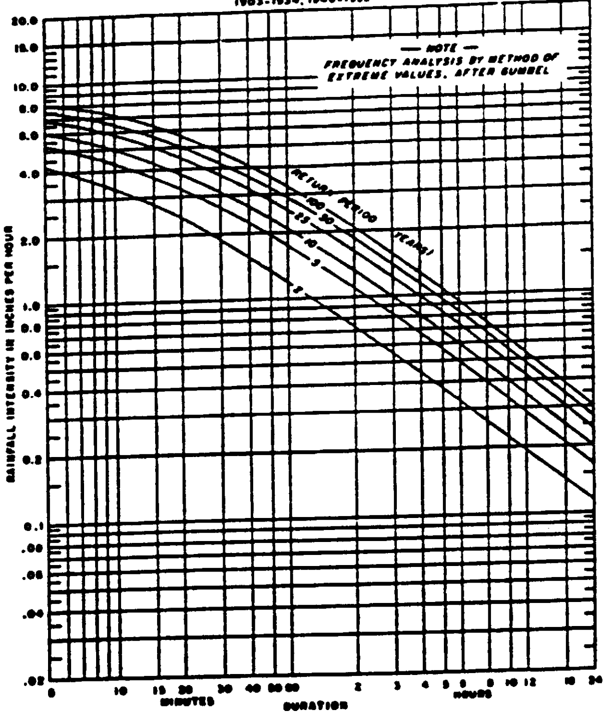
CONTENTS

INTRODUCTION

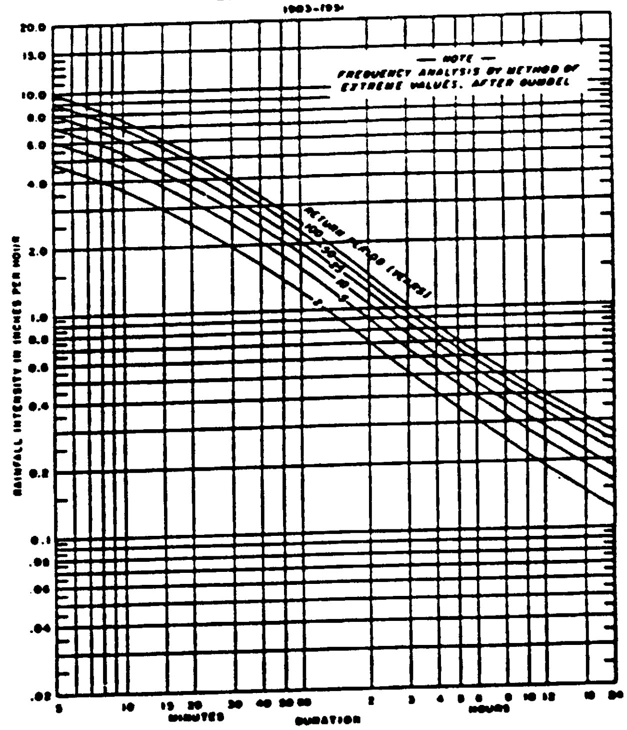
The data herein supplements the written commentary in Part 2 of this series. The computational examples shown are representative of one water management approach to each specific problem and should not be construed as suggesting that other methods are deliberately excluded. As was noted in the introduction to a preceding Part, the author is attempting to portray examples which can be understood by non-specialists.

RAINFALL INTENSITY-DURATION-FREQUENCY CURVES

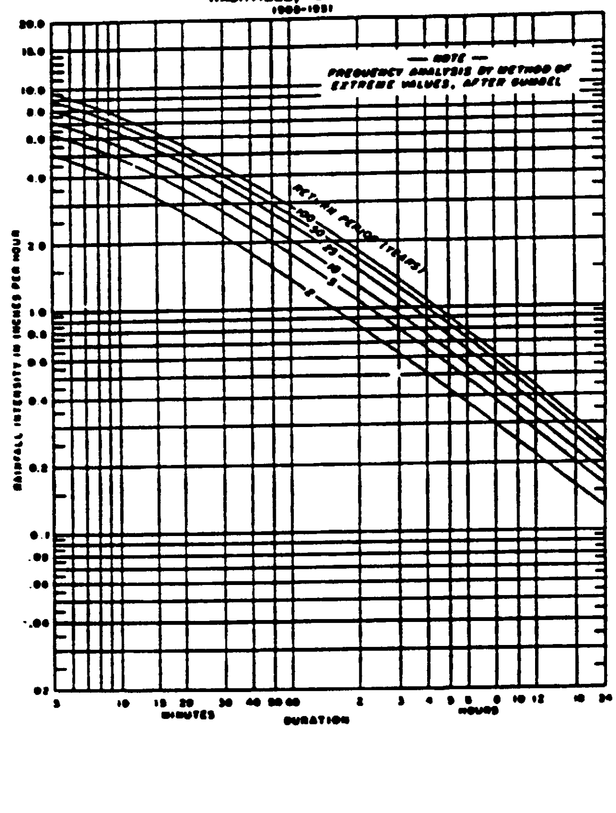
LEXINGTON, KENTUCKY
1903-1934, 1946-1953



LOUISVILLE, KENTUCKY
1903-1951



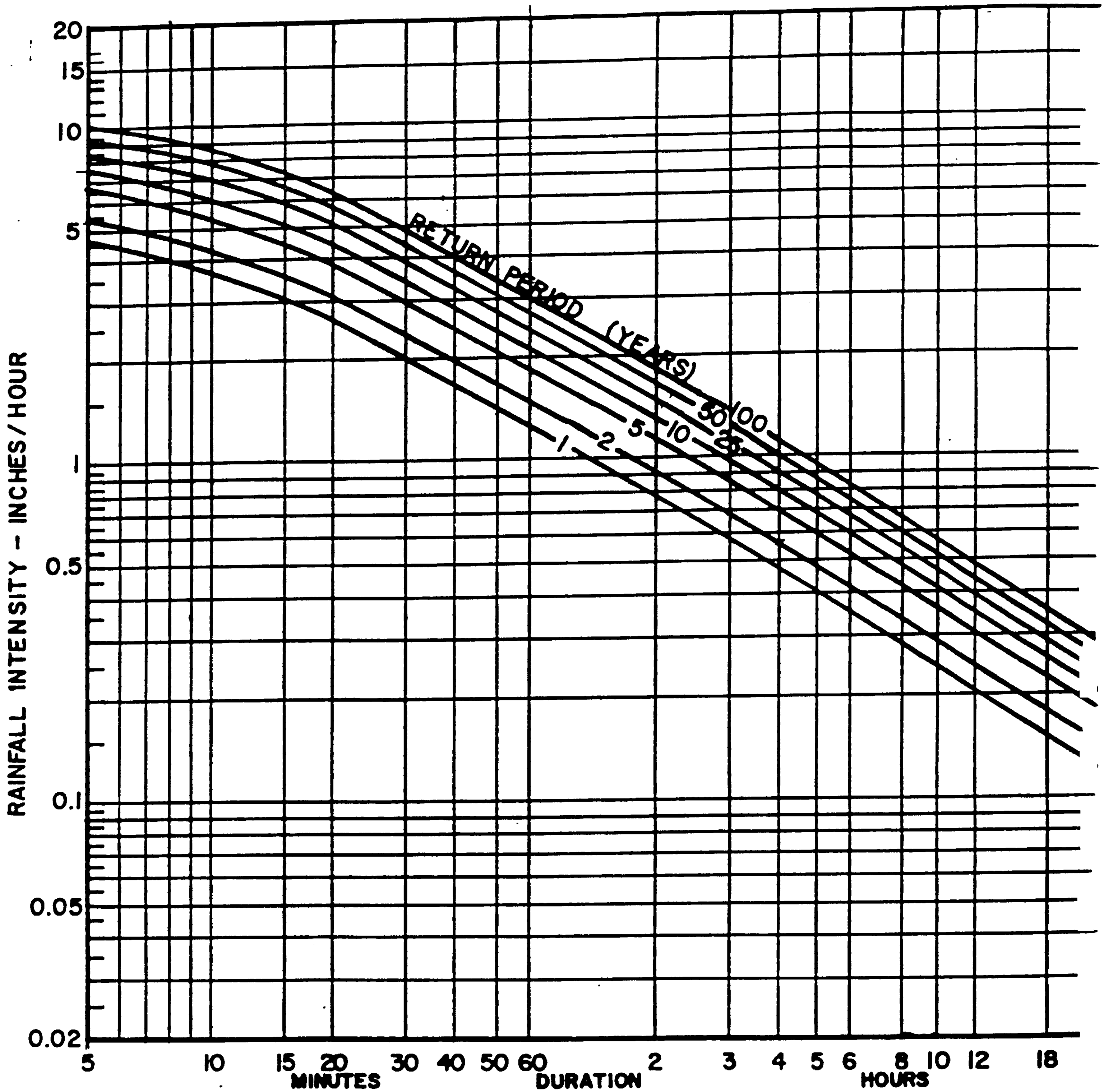
NASHVILLE, TENNESSEE
1900-1951



APPENDIX 'A'

A/2

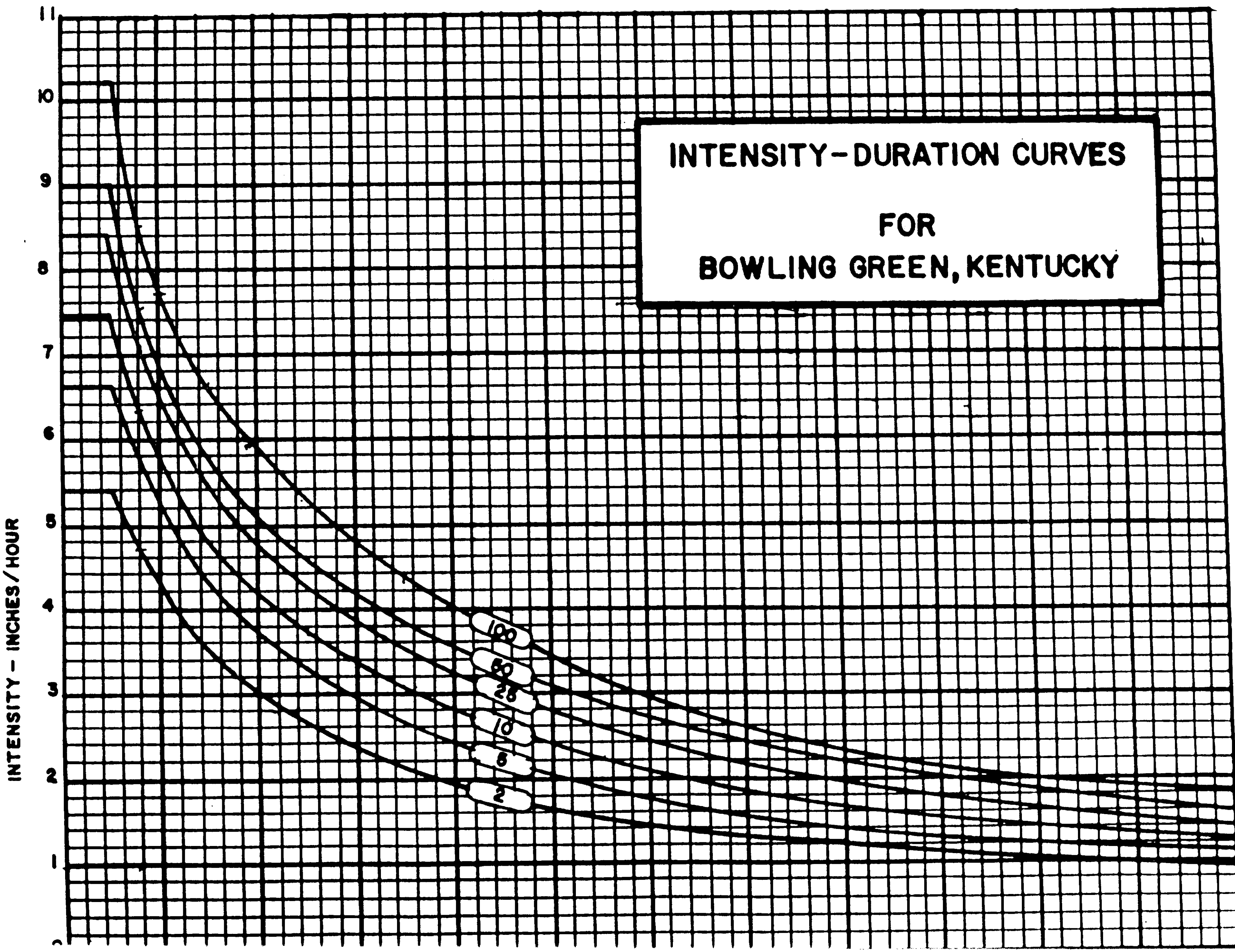
BOWLING GREEN, KENTUCKY



RAINFALL INTENSITY — DURATION FREQUENCY CURVES

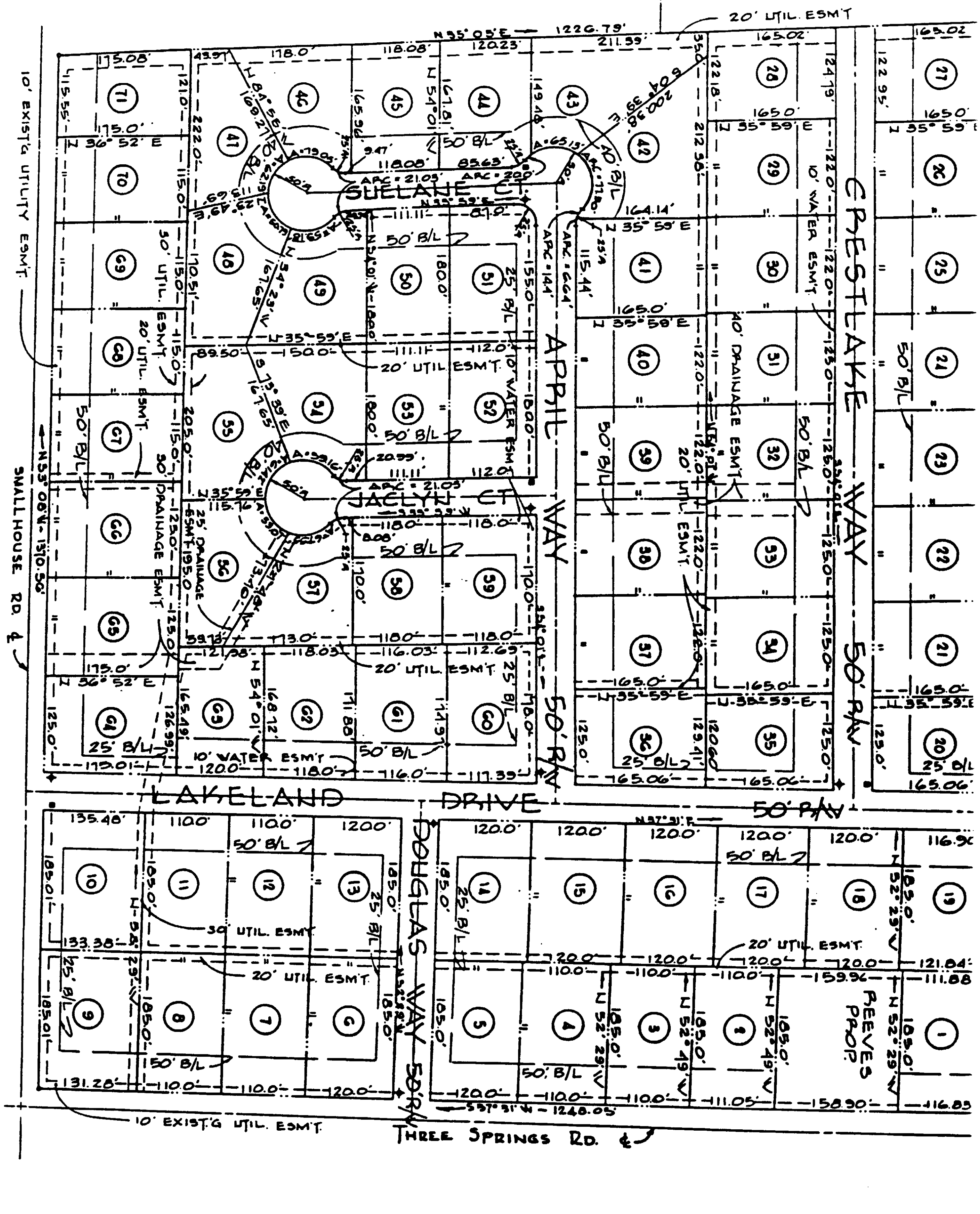
BASED ON DATA IN U.S.W.B. TECHNICAL PAPER NO. 40, DATED 1961.

APPENDIX 'A'


INTENSITY - DURATION CURVES
FOR
BOWLING GREEN, KENTUCKY
INTENSITY - INCHES/HOUR
0
1
2
3
4
5
6
7
8
9
10
11
100
50
25
10
5
2

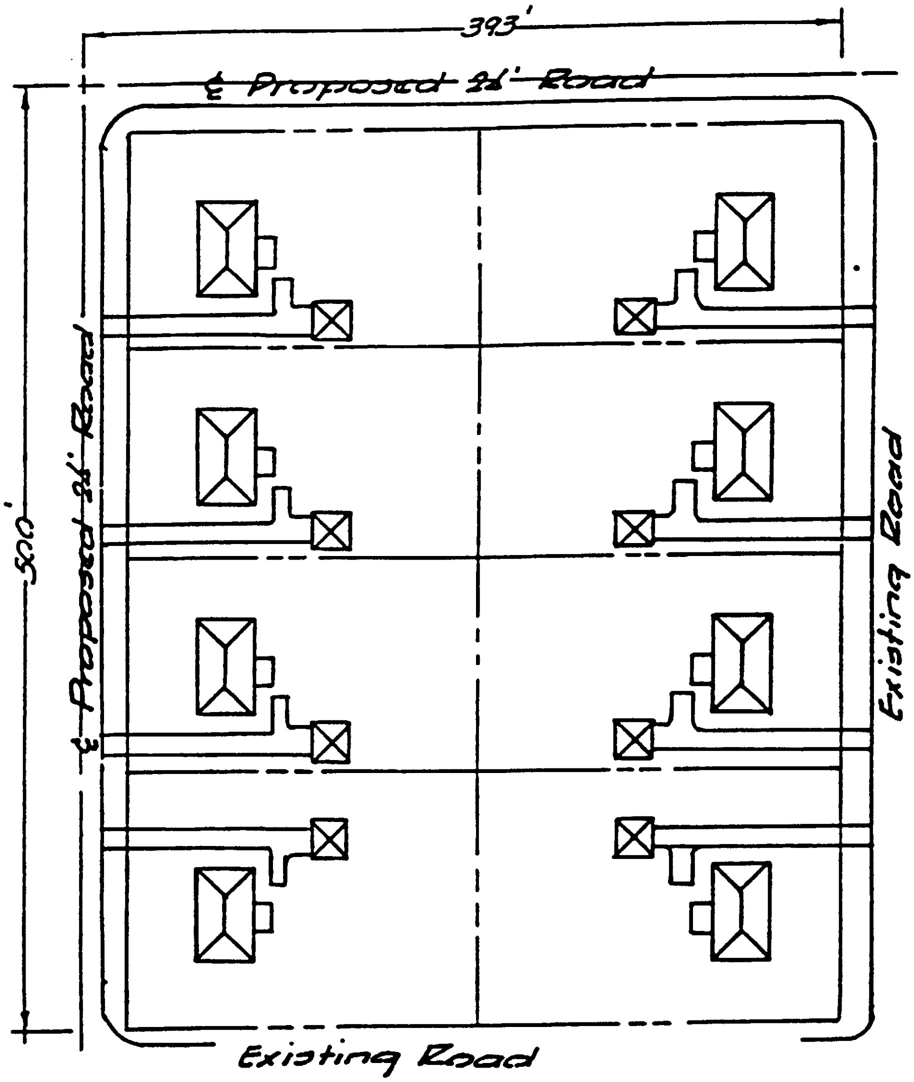
A/3

B/1



20' UTIL. ESM'T
N 35° 05' E — 1226.79'
SUELANE CT.
JACLYN CT.
APRIL WAY
50' R/W
CRESTLAKE WAY
50' R/W
LAKELAND DRIVE
50' R/W
DOUGLAS WAY 50' R/W
10' EXIST'G UTILITY ESM'T
SMALLHOUSE RD. ℄
N 33° 08' W - 1310.56'
30' UTIL. ESM'T
20' UTIL. ESM'T
30' DRAINAGE ESM'T
25' DRAINAGE ESM'T
10' WATER ESM'T
40' DRAINAGE ESM'T
20' UTIL. ESM'T
10' WATER ESM'T
REEVES PROP.
S 37° 31' W - 1248.05'
10' EXIST'G UTIL. ESM'T
THREE SPRINGS RD. ℄

CHANGE IN 'C' FACTOR



TYPICAL PARTIAL DEVELOPMENT PLAN

EXISTING CONDITIONS
Area = 393 × 500
= 196,500 sf
$C_N = 0.25$

PROPOSED CONDITIO[NS]
Area =

Impervious Areas
Houses (8)
(8) × 1500 = 12,000 sf
Patios (8)
(8) × 150 = 1,200 s[f]
Garages (8)
(8) × 440 = 3,520 sf
Drives (8)
(8) × 110 × 10 = 8,800 s[f]
(8) × 20 × 20 = 3,200
12,000
Total Impervious -
28,720 sf

Roadways (prop.)
(500 + 393 - 12) × 12'
= 10,572 sf

Total Lot + Road
Impervious Areas
39,292 sf

$C_I = 0.95$

Computation of C_M:

$$\frac{(C_N \times A_P) + (C_I \times A_I)}{A} + k\left(\frac{A_P}{A}\right) = C_M$$

where: $C_N = 0.25$ & $C_I = 0.95$
$A = 196{,}500$
$A_I = 39{,}292$
$A_P = 157{,}208$
$k = .20$

$$\frac{(.25 \times 157{,}208) + (.95 \times 39{,}292)}{196{,}500} + 0.2\left(\frac{157{,}208}{196{,}500}\right) = C_M$$

$$0.39 + 0.16 = \underline{\underline{0.55}} = C_M$$

Note: "k" factor of 0.20 shown represents drainage enhanceme[nt] from re-grade of pervious areas, and reflects the engineer's judgement.

RUN-OFF FOR PURPOSE OF RETENTION

As an example of run-off to be retained, the foregoing "TYPICAL PARTIAL DEVELOPMENT PLAN" portrayed a method of computing the modified run-off factor, C_M, and this value of $C_M = 0.55$ will be applied to the entire 39.5 acre project.

$C_M = 0.55$ $\quad\quad \Delta C = C_M - C_N = 0.55 - 0.25 = 0.30$
$A = 39.5$ acres
$R = 2.95''/12$ (for one hour of 100-Yr. storm)

For change in run-off in one hour and a gravity-surfa… project outlet:

Retention $= \Delta C\, R\, A$
$= 0.30 \times \frac{2.95}{12} \times 39.5$ acres
$= \underline{2.91}$ acre-ft.

For total run-off in three hours and a sinkhole out…

Retention $= C_M\, R\, A$
$= 0.55 \times \frac{4.0}{12} \times 39.5$ acres
$= \underline{7.17}$ acre-ft.

4. COLORADO URBAN HYDROGRAPH PROCEDURE

For basins that are larger than about 200 acres and for some complex basins that are less than 200 acres, it is recommended that the design storm runoff be analyzed by deriving synthetic unit hydrographs. The unit hydrograph principle was originally developed by Sherman in 1932 (12). The synthetic unit hydrograph, which is used for analysis when there is no rainfall-runoff data for the basin under study, as if often the case in the Denver region, was developed by Snyder in 1938 (13). the presentation given in this chapter is termed the Colorado Urban Hydrograph Procedure (CUHP) because coefficients are based upon data collection and studies financed by the City of Denver, the Denver Regional Council of Governments and the Urban Drainage and Flood Control District.

4.1 Definition

A unit hydrograph is defined as the hydrograph of one inch of direct runoff from the tributary area resulting from a unit storm. A unit storm is a rainfall of such duration that the period of surface runoff is not appreciable less for any rain of shorter duration. The unit hydrograph thus represents the integrated effects of factors such as tributary area, shape, street pattern, channel capacities, and street and l and slopes (14, 15, 16, 17, 18, 19).

To apply the unit hydrograph the effective precipitation depth for the "unit storm" periods are multiplied by the ordinates of the unit hydrograph and added to obtain a design storm runoff.

the basic premise of the unit hydrograph is that individual hydrographs resulting from the successive increments of rainfall excess that occur throughout a storm period will be proportional in discharge throughout their length, and that when properly arranged with respect to time the ordinates of the individual unitgraphs can be added to give ordinates representing the total storm discharge. The hydrograph of total storm discharge is obtained by summing the ordinates of the individual hydrographs.

4.2 Basic Assumptions

The derivation and application of the unit hydrograph are based on the following assumptions:

1. The rainfall intensity is constant during the storm that produces the unit hydrograph.

2. The rainfall is uniformly distributed throughout the whole area of the drainage basin.

3. The base or time duration of the design runoff due to an effective rainfall of unit duration is constant.

4. The ordinates of the design runoff with a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph.

5. The effects of all physical characteristics of a given drainage basin, including shape, slope, detention, infiltration, drainage pattern, channel storage, etc., are reflected in the shape of the unit hydrograph for that basin.

4.3 Equations

There are two basic equations used in defining the limits of the synthetic unit hydrograph. The first equation defines the lag time of the basin in terms of time to peak, t , which, for the CUHP Method, is defined as the time from the center of the unit storm duration to the peak of the unit hydrograph as shown in Figure 4-6. For most urban studies the unit storm duration should range between 5 to 50 minutes.

$$t_p = C_t\,(L\,L_{ca})^{.3} \qquad (4\text{-}1)$$

Where t_p = time to peak of hydrograph from midpoint of unit rainfall in hours.

L = length along stream from study point to upstream limits of the basin in miles.

L_{ca} = distance from study point along stream to the centroid of the basin in miles.

C_t = a coefficient reflecting time to peak.

The second equation defines the unit peak of the unit hydrograph.

$$q_p = \frac{640\,C_p}{t_p} \qquad (4\text{-}2)$$

Where q_p = peak rate of runoff in cfs per square mile

C_p = a coefficient related to peak rate of runoff.

For discussions of C_p and C_t values refer to paragraph 4.4.

4.4 C_p and C_t Data from Denver Watersheds

The C_p and C_t values in equations 4-1 and 4-2 are determined from the following equations:

$$C_t = \frac{7.81}{(I_a)^{0.78}} \tag{4-3}$$

$r^2 = 0.95$ (coefficient of determination)

where I_a = percent of watershed which is impervious.

$$C_p = 0.89\,(C_t)^{0.46} \tag{4-4}$$

$r^2 = 0.21$ (coefficient of determination)

Equations 4-3 and 4-4 were developed from a statistical analysis of ninety=six 5-minute unit hydrographs derived from flood events measured on nineteen different urban watersheds in the Denver-Boulder metropolitan region during the period from 1967 to 1973. The 5-minute unit hydrographs were derived from the measured floods using the HEC1 computer program (35). The Snyder Time and Peak Coefficients, C_t and C_p, were obtained from these derived unit hydrographs. The percent of impervious watershed existing at the time of the flood event was determined from aerial photographs. The time to peak, t_p, of the unit hydrographs is shown as a function of the watershed parameter LL_{ca} on Figure 4-1. It was assumed that the equation of the line through the data would follow the general form of Equation 4-1 with percent of impervious watershed, I_a, as the third parameter. A line was first drawn through the data for a I_a = 50% because there were more data available over a larger range of the watershed parameter, $L\,L_{ca}$. Lines for I_a = 8%, 30%, 40%, and 100% were subsequently drawn parallel to the 50 percent line on the lag curve.

The scatter of the data on Figure 4-1 is attributed to the fact that the floods observed during the 1976 to 1973 period were mainly small floods. Based on unit hydrograph research in this field (17, 32, 33, 34), there is a tendency for non-linearity and scatter to exist amounts the unit hydrograph parameters when the unit hydrographs were derived from small amounts of rainfall excess.

(This section will be revised as additional data becomes available.)

The values of C_t and C_p can be estimated either from equations 4-3 and 4-4, or estimated graphically from Figure 4-2 and 4-3. Some additional data from unit hydrograph studies elsewhere in the United States are shown on Figure 4-2 to assist in defining the curve.

The percent of the impervious watershed, I_a, for an urban watershed in the first stages of planning may be estimated using the values suggested in Table 2-1. Alternatively the percent of impervious watershed could be estimated from aerial photographs of an existing urban watershed having a similar plan of development adjacent to the planned watershed.

C

For estimating C_t: Add 10% for sparsely sewered areas. Subtract 10% for fully sewered areas.

Add 10% for very flat basins. Subtract 10% for steep basins.

For estimating C_p: Subtract 10% for sparsely sewered areas; add 10% for fully sewered areas.

Subtract 10% for very flat basins; add 10% for steep basins.

For estimating: See Table 2-1 for percent impervious data.

4.5 Unit Hydrograph Shape

The shape of the unit hydrograph is a function of the physical characteristics of the watershed. The shape is developed from empirical relationships.

The peak rate of discharge, q_p, is determined from equation4-2. The value of q_p can be checked by consulting Figure 4-4. The regression line shown on Figure 4-4 is:

$$q_p = \frac{Q_p}{A} = 1387\,(A)^{-0.348} \qquad (4\text{-}5)$$

If the basin is excessively long or flat, the value should be somewhat below the regression line. If the watershed is excessively steep or has high velocity in its channels, the value of q_p should be above the regression line.

Both Figures 4-4 and 4-5 were prepared from the characteristics of the 5-minute unit hydrographs derived from the floods measured on 19 Denver metropolitan region urban watersheds. Equations 4-6 and 4-7 may be used to estimate the width of the unit hydrograph at 50 percent and 75 percent of the peak discharge:

$$W_{@50\%\,Qp} = \frac{500}{\frac{Q_p}{A}} \qquad (4\text{-}6)$$

$$W_{@75\%\,Qp} = \frac{260}{\frac{Q_p}{A}} \qquad (4\text{-}7)$$

These values could also be obtained from Figure 4-5.

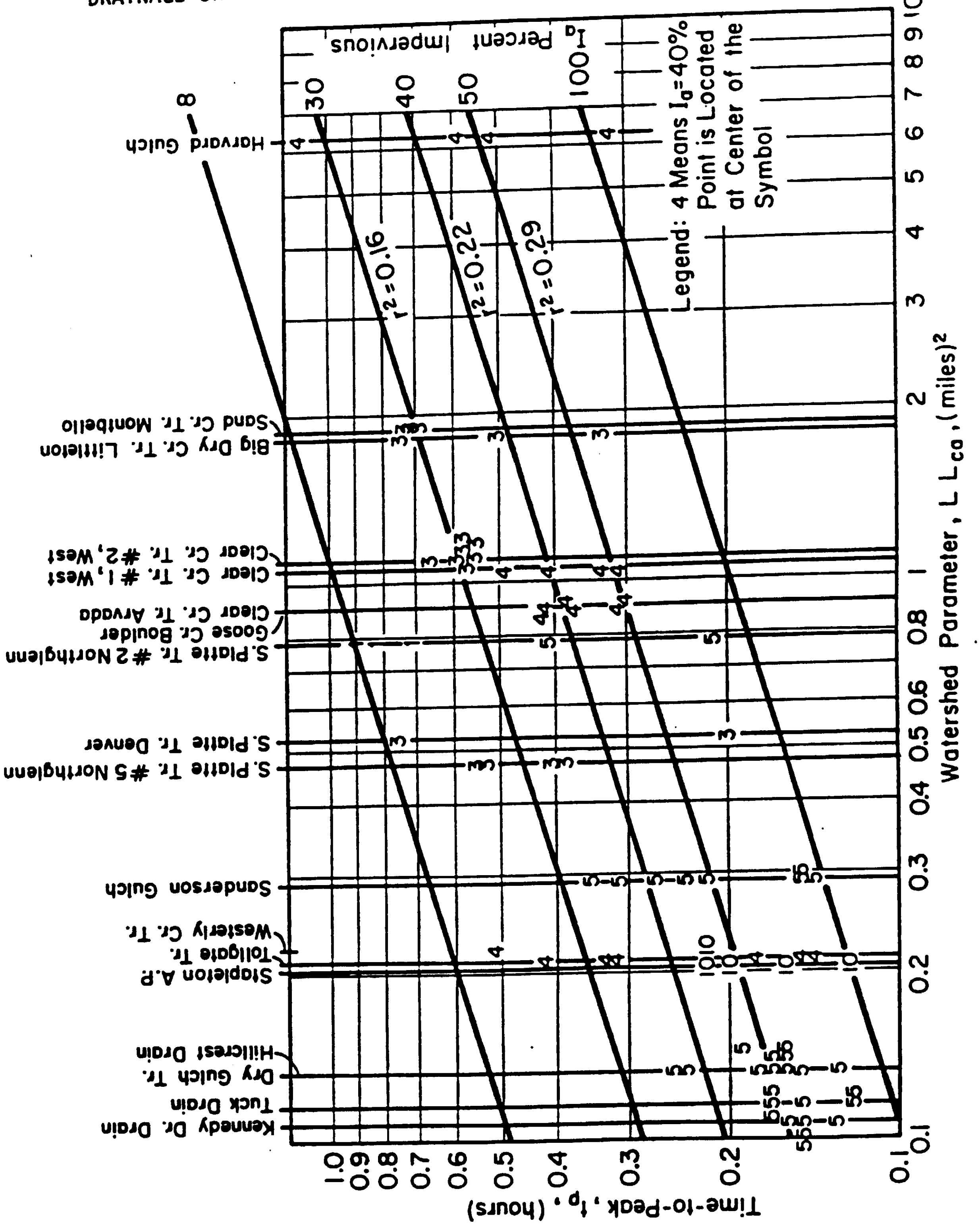


Figure 4-1 LAG CURVE FOR DENVER URBAN WATERSHEDS

1-15-69
Revised 5-15-75 follows paragraph 4.4 and Equation 4-4

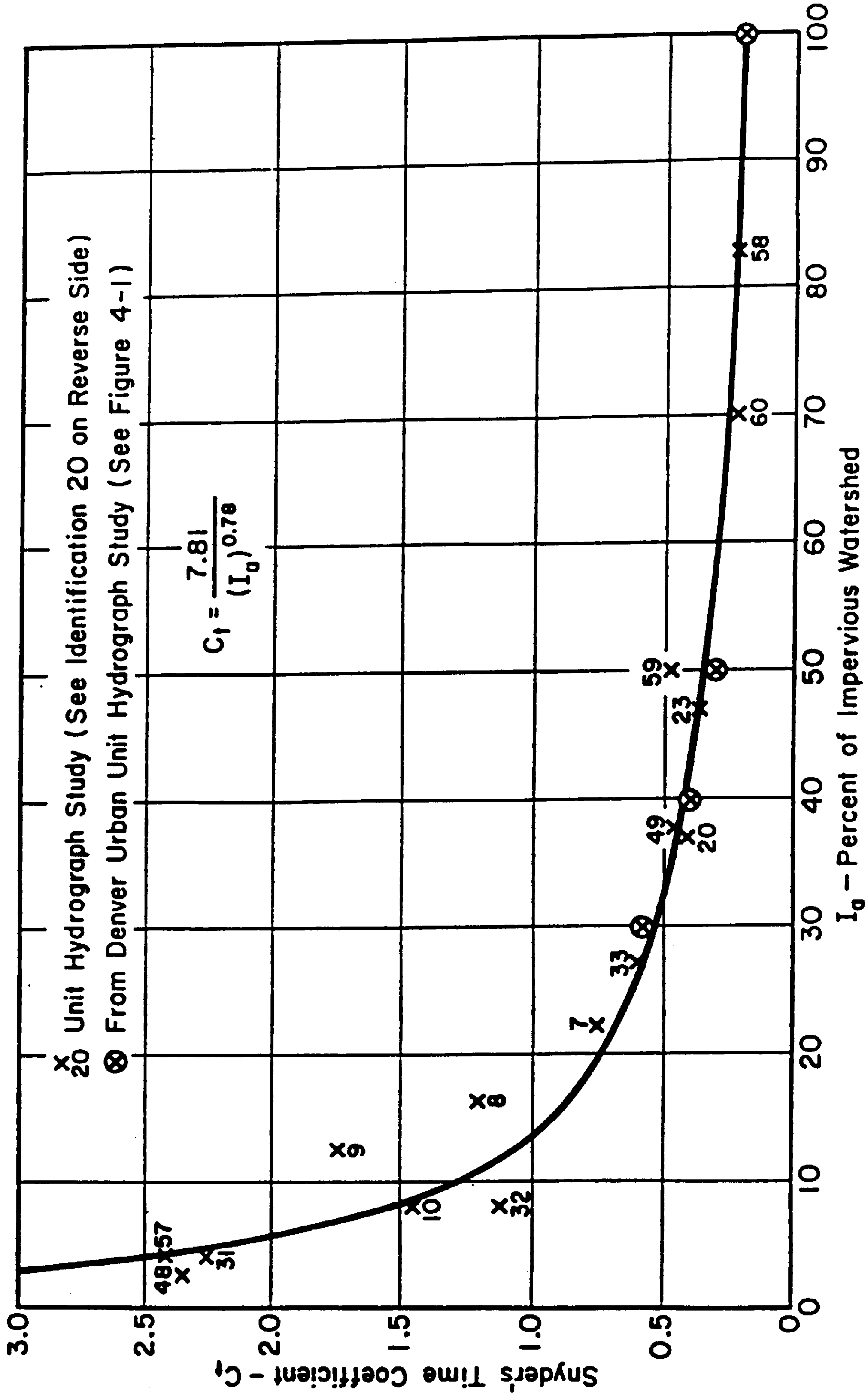


Figure 4-2 RELATIONSHIP BETWEEN C_t AND IMPERVIOUSNESS

Identification Number	Stream	Drainage Area (sq.mi.)	I_a (%)	C_t
7	Wooden Bridge Run, Philadelphia, PA	3.35	22.1	0.76
9	Poquessing Cr. at Trevose Rd., Philadelphia, PA	5.1	12.5	1.74
8	Wissahichon Cr. at Bells Mill Rd., Philadelphia, PA	53.6	16.3	1.22
10	Pennypack Cr. at Pine Rd., Philadelphia, PA	37.9	9.1	1.45
20	Brushy Cr. at Hiway 311, Winston-Salem, NC	0.55	37	0.41
23	Turtle Cr. Dallas, TX	7.98	47	0.37
31	Cole Cr. at Guhn Rd., Houston, TX	7.05	4	2.25
32	Brickhouse Gully at Costa Rica St., Houston, TX	10.5	8	1.13
33	Waller Cr. at 38th St., Austin, TX	2.31	27	0.51
48	Anacostia Cr., IL	72.4	2.7	2.36
49	Boneyard Cr. at Urbana, IL	4.45	37.4	0.45
57	Salt Fork, West Br. IL	71.4	4	2.42
58	Louisville at 17th st. KY	0.22	83	0.22
59	Louisville, North Trunk Sewer	1.9	50	0.26
60	Louisville, West Outfall, KY	2.77	70	0.21

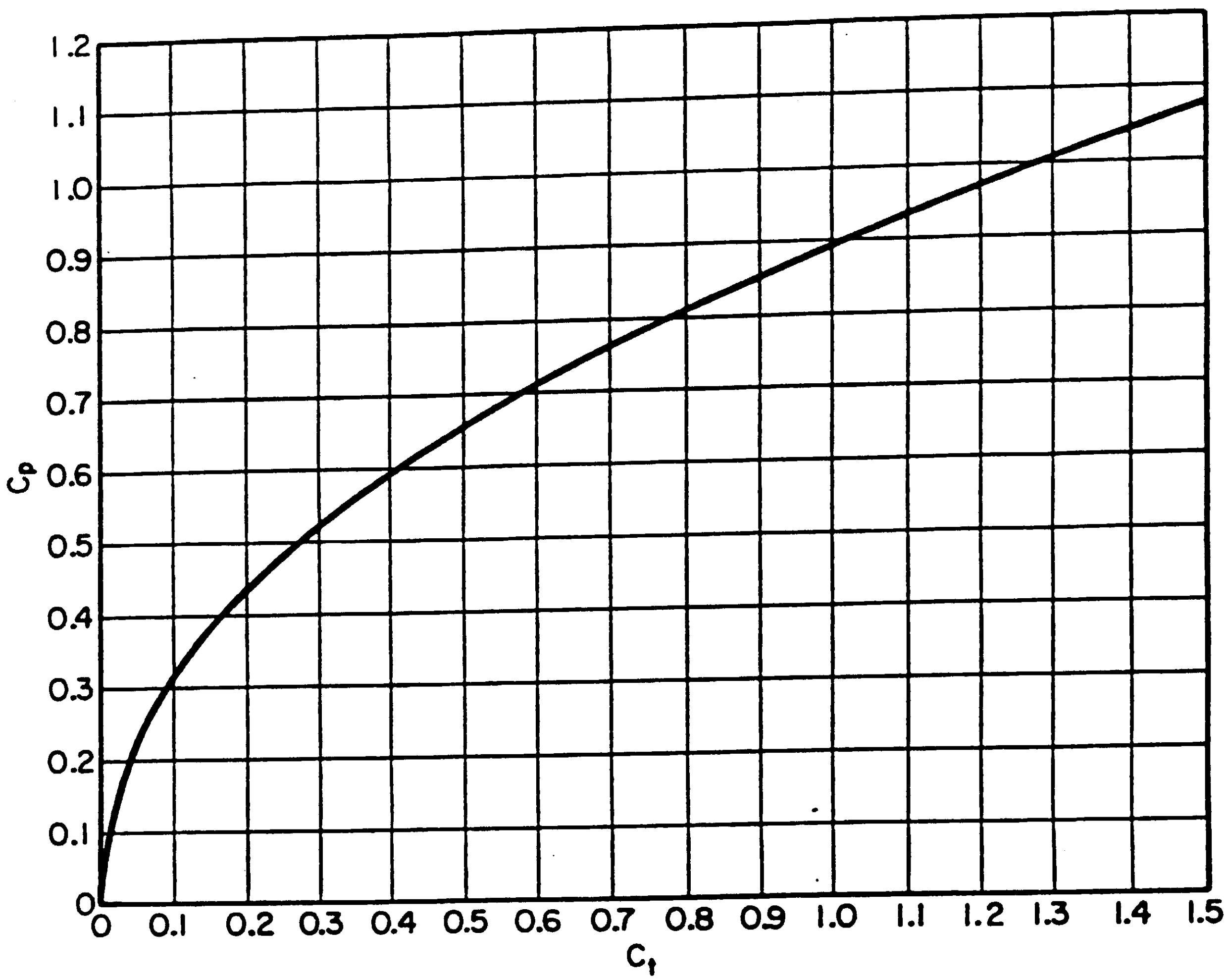


Figure 4-3 RELATIONSHIP BETWEEN C_p AND C_t

4.6 Drawing the Unit Hydrograph

Once q_p is determined from Equation 4-2, Q_p, the maximum unit hydrograph peak for the basin, can be computed by:

$$Q_p = q_p A \qquad (4\text{-}8)$$

Where A is the area of the basin in square miles.

The time from the beginning of rainfall to the peak of the unit hydrograph is determined by:

$$T_p = 60\ t_p + 0.5\ t_u$$

Where t_u = time of unit rainfall duration in minutes, and

T_p = time from beginning of unit rainfall to peak of hydrograph in minutes.

Once Q_p is located, the unit hydrograph can be sketched with the aid of the approximate widths $Q_{50\%}$ and $Q_{75\%}$. After the hydrograph is sketched, the area under the hydrograph should be planimetered to determine the volume of runoff in acre feet of other suitable units.

This volume should equal the volume of l inch of runoff from the entire basin, or Vol. = Area in acres x 1/12. If the two volumes are within 5 percent, then the sketched unit hydrograph is acceptable. If the volume from the drawn hydrograph should be adjusted to within 5 percent of one inch of runoff. the final step is to define the unit hydrograph in tabular form showing time vs. rate of flow in cfs. If Q_p does not fall on a chosen time interval so that the tabulation does not represent the graph. then the graph may be shifted so that the table will more truly represent the graph.

4.7 Design Storm Runoff

Now that the unit hydrograph has been calculated (4.6) and the effective precipitation from the design storm determined (2.4), the design storm hydrograph can be calculated. The time units of the unit hydrograph abcissa should be the same as the time units of the excess precipitation which for convenience should all be equal to the unit storm duration, and can generally be taken as 10 minutes for an urban area less 5 square miles. Unit times of 5 minutes can be used for small basins up to 0.5 square miles, and unit times of 15 minutes and more for larger basins.

Set up a table such as Table 4-1, putting time intervals in the first column and unit hydrograph ordinates in the second column. Place the design excess precipitation values as determined in Column 13 of Table 2-3 across the top, and then multiply the first excess precipitation value (.02 in example) times all the unit hydrograph ordinates in Column 2 and put answers in the third column. Next multiply the second excess precipitation value (.05) times the unit hydrograph ordinates lagged one time unit as shown in Column 4. Multiply each succeeding precipitation value times the unit hydrograph value and lag them appropriately in the table. Finally, add up all the multiplied values horizontally to obtain the design storm runoff hydrograph.

4.8 Example

Given: A basin when fully developed is expected to have the following characteristics:

Area = 0.85 square miles = 544 acres.

$L = 1.21$ miles
$L_{ca} = 0.85$ miles
60% = pervious area
40% = impervious area

Use a unit duration of 10 minutes.

Determine a 10 year design runoff from the basin, using the CUHP method.

Step 1. Determine C_t given the percent of impervious cover using Equation 4-3. Alternatively C_t may be estimated from Figure 4-2.

$$C_t = \frac{7.81}{(40)^{0.78}} = 0.44$$

Step 2. Determine t_p using Equation 4-1.

$$t_p = C_t (L\ L_{ca})^{.3} = .44\ (1.21 \times .85)^{.3}$$

$$= .44\ (1.008) = 0.44 \text{ hour} = 27 \text{ minutes}$$

Step 3. Determine C_p using Equation 4-4 and value of C_t found in Step 1.

$$C_p = 0.89\ C_t^{0.46} = .89\ (.44)^{.46} = 0.61$$

Step 4. Determine q_p using Equation 4-2.

$$q_p = \frac{640 C_p}{t_p} = \frac{640\ (.61)}{.44} = 887 \text{ cfs/sq.mi.}$$

Step 5. Determine $Q_p = q_p A = 887\ (.85) = 754$ cfs,
say 750 cfs.

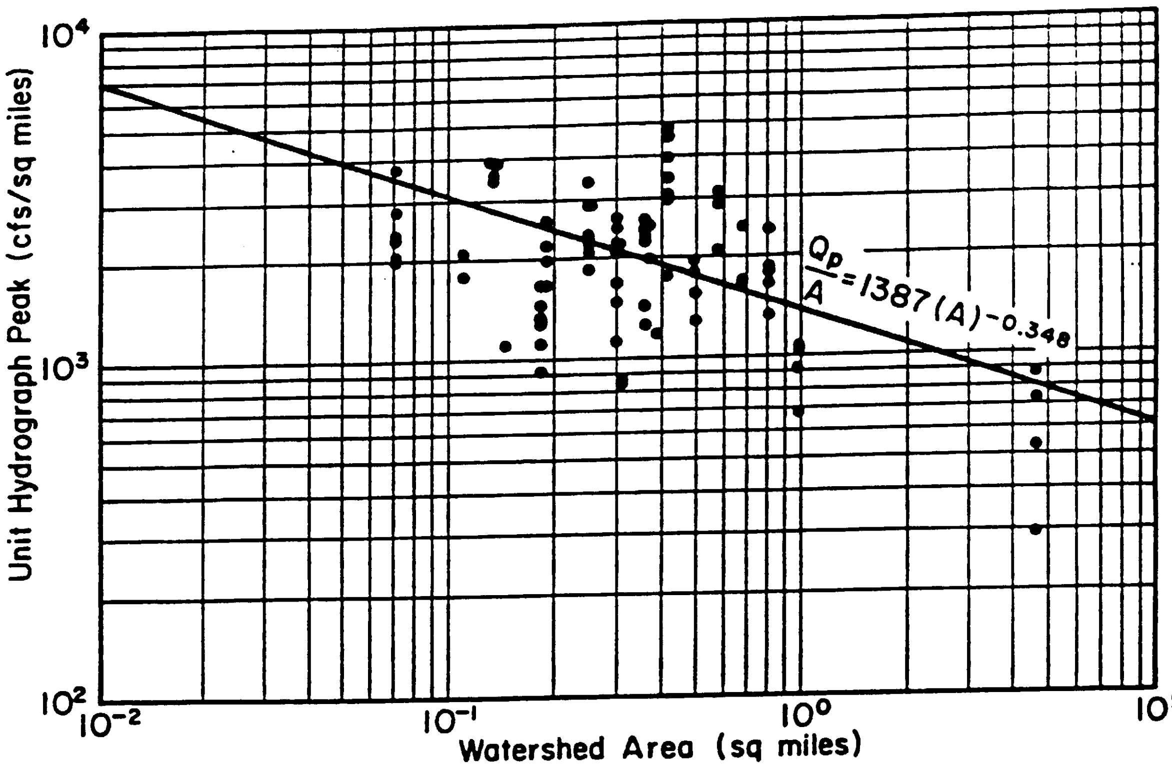


Figure 4-4 Check for Unit Hydrograph Peak

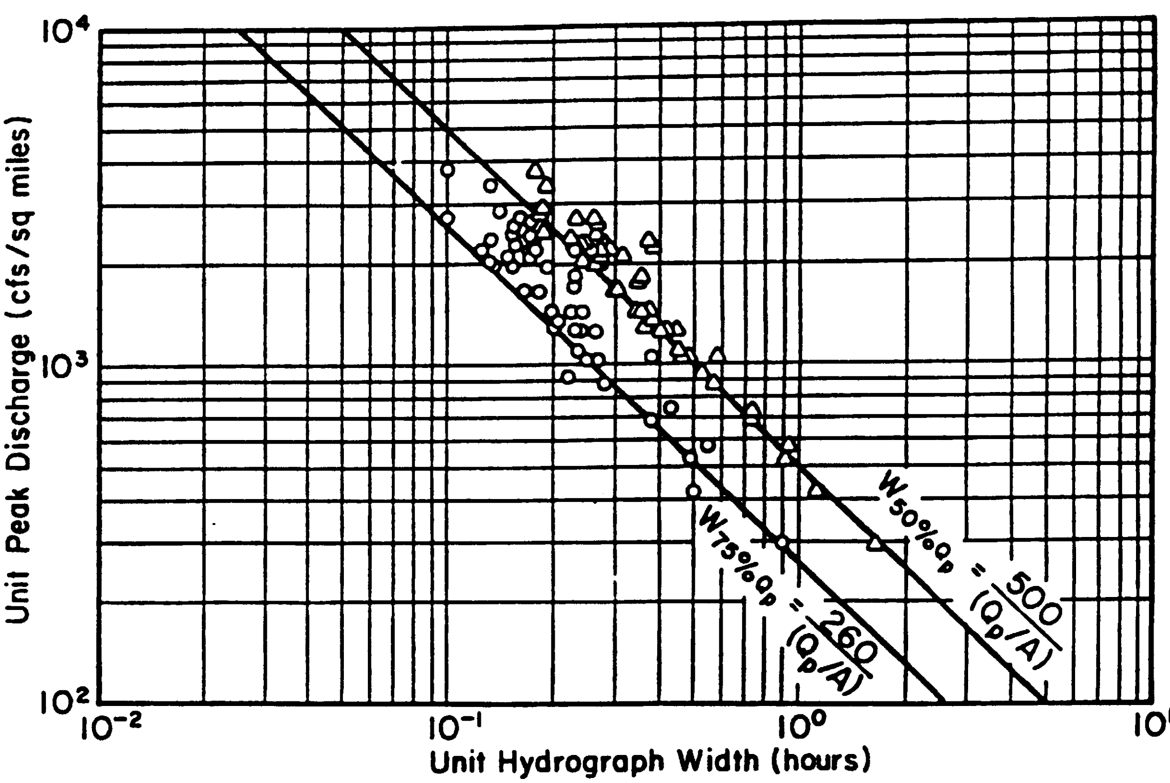


Figure 4-5 UNIT HYDROGRAPH WIDTHS

TABLE 4-1

Determination of Storm Hydrograph (Example)

Time (min) (1)	Unit Hydrograph (cfs) (2)	Excess Precipitation in inches														Storm Hydrograph (cfs) (17)
		0.02 (3)	0.05 (4)	0.69 (5)	0.24 (6)	0.16 (7)	0.06 (8)	0.03 (9)	0.03 (10)	0.02 (11)	0.02 (12)	0.02 (13)	(14)	(15)	(16)	
0	0	0														0
10	160	3	0													3
20	460	9	8	0												17
30	750	15	23	110	0											148
40	570	11	38	317	38	0										404
50	390	8	29	518	110	26	0									691
60	265	5	20	393	180	74	10	0								682
70	185	4	13	269	137	120	28	5	0							576
80	135	3	9	183	94	91	45	14	6	0						444
90	100	2	7	128	64	62	34	23	14	3	0					337
100	75	2	5	93	44	42	23	17	23	9	3	0				261
110	50	1	4	69	32	30	16	12	17	15	9	3				208
120	40	1	3	52	24	22	11	18	12	11	15	9				168
130	30	1	2	35	18	16	8	6	8	8	11	15				128
140	20	0	2	28	12	12	6	4	6	5	8	11				94
150	10		1	21	10	8	5	3	4	4	5	8				69
160	0		1	24	7	6	3	2	3	3	4	5				48
170			0	7	5	5	2	2	2	2	3	4				32
180				0	2	3	2	1	2	2	2	3				17
190					0	2	1	1	1	1	2	2				10
200						0	1	1	1	1	1	2				7
210							0	0	1	1	1	1				4
220									0	0	1	1				2
											0	1				1
												0				0

Step 6. Determine the width of the unit hydrograph at 50% and 75% of the using Q_p Figure 4-5 and q_p = 890 cfs/sq.mi.

$$W_{50\%q_p} = 0.56 \text{ hours} = 34 \text{ minutes}$$

$$W_{75\%q_p} = 0.29 \text{ hours} = 17 \text{ minutes}$$

Step 7. Determine the time to peak from the beginning of rainfall using Equation 4-9.

$$T_p = 60\, t_p + \frac{t_u}{2} = 27 + \frac{10}{2} = 32 \text{ minutes}$$

Step 8. Using the results of Steps 5, 6, and 7, sketch a unit hydrograph. See Figure 4-6.

Step 9. The volume of the unit hydrograph should be:

544 acres x 1 inch/12 = 45.3 ac. ft.

Planimeter the area under the hydrograph and determine the actual volume.

The volume for the first trial was 50.3 ac.ft. which was about 10% too high. The unit hydrograph was revised as shown in Figure 4-6. The volume was 44.6 ac.ft. which is a trifle too small but easily falls within the 5% criterion given in paragraph 4.6. The revised hydrograph as shown is thus accepted.

Step 10. Repeat Steps 8 and 9 until the runoff volume under the hydrograph is equal to 45 ± acre feet. then present the unit hydrograph in tabular form as shown on Figure 4-6.

Step 11. Obtain the design excess precipitation values in 10-minute (unit Duration) increments. This is done in Table 2-3.

Step 12. Set up Table 4-2.

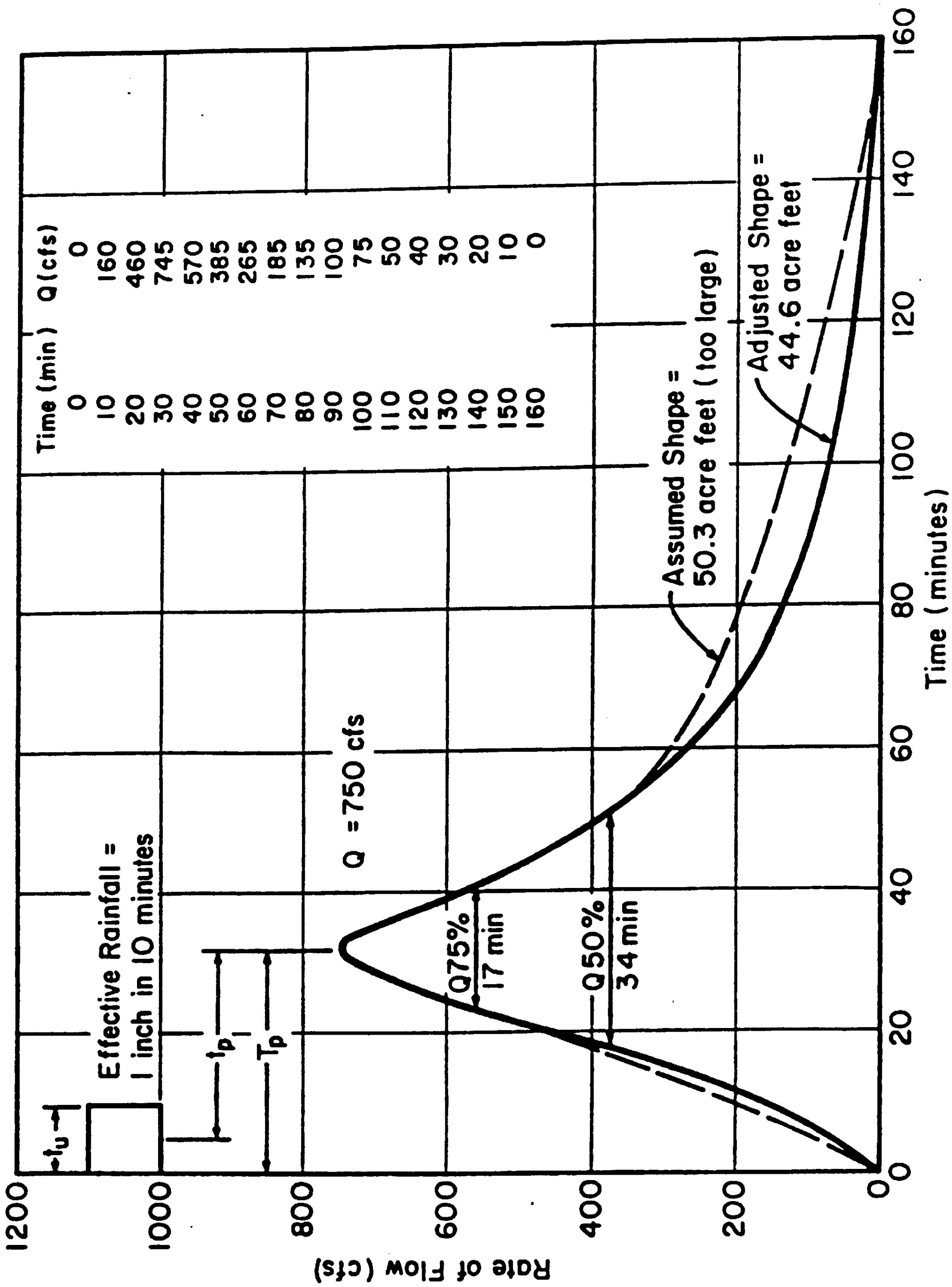
Step 13. Multiply the precipitation value at the top of Column 3 by each of the unit hydrograph ordinates and put in Column 3 for the corresponding time. Next multiply the precipitation value in Column 4 by each of the unit hydrograph ordinates and place in Column 4 lagged one time from the corresponding unit hydrograph time. Proceed to multiply each of the precipitation values times the unit hydrograph ordinates, each time lagging the new hydrograph by one more time unit.

Step 14. Column 17 is the design runoff hydrograph obtained by summing horizontally the individual hydrographs in Column 3 through 12. Note that in this example time zero is the beginning of excess rainfall and not the beginning of rainfall. This is important when lagging and routing several hydrographs from different basins together.

4.9 Acquisition of additional data

In 1969 the basic Snyder synthetic unit hydrograph method was modified for use in the Denver Metropolitan region. The Denver Regional Council of Governments and later the Urban Drainage and Flood Control District in cooperation with the U.S. Geological Survey began a systematic data acquisition program.

In 1975 the significant flood events measured in the Denver Metropolitan network were analyzed. The 5-minute unit hydrographs derived from 96 flood events measured on 19 different watersheds have provided a valuable insight into the effect of urbanization on the unit hydrograph parameters. As the span of records and the magnitude of the floods increases, the equations and graphs will be modified reflecting the more complete knowledge about the formation of floods in the urban environment.



Time (min)	Q (cfs)
0	0
10	160
20	460
30	745
40	570
50	385
60	265
70	185
80	135
90	100
100	75
110	50
120	40
130	30
140	20
150	10
160	0

Figure 4-6 UNIT HYDROGRAPH EXAMPLE

TABLE 2-3

DETERMINATION OF EFFECTIVE RAINFALL (Example)

Location: Sec. 31, T.2N., R.70W.

Design Storm: 10 Year Recurrence Interval

				PERVIOUS AREA 60 %				IMPERVIOUS AREA 40 %				
Time min. (1)	Total Precipitation (in.) (2)	Incremental Precipitation (in.) (3)	Rearranged Incremental Precipitation (in.) (4)	Maximum Infiltration (in.) (5)	Detention & Depression Storage (in.) (6)	Effective Precipitation (in.) (7)	60 % Effective Precipitation (in.) (8)	Detention & Depression Storage (in.) (9)	Loss (in.) (10)	Effective Precipitation (in.) (11)	40 % Effective Precipitation (in.) (12)	Total Average Effective Precipitation (in.) (13)
0	0	0	0	0	0	0	0	0	0	0	0	0
10	.89	.89	.05	.08	0	0	0	.05	0	0	0	0
20	1.18	.29	.10	.08	.02	0	0	.05	0	.05	.02	.02
30	1.40	.22	.13	.08	.05	0	0	0	.01	.12	.05	.05
40	1.53	.13	.89	.08	.23	.58	.35	0	.04	.85	.34	.69
50	1.64	.11	.29	.08	0	.21	.13	0	.01	.28	.11	.24
60	1.74	.10	.22	.08	0	.14	.08	0	.01	.21	.08	.16
70	1.82	.08	.11	.08	0	.03	.02	0	.01	.10	.04	.06
80	1.89	.07	.08	.08	0	0	0	0	0	.08	.03	.03
90	1.95	.06	.07	.08	0	0	0	0	0	.07	.03	.03
100	2.00	.05	.06	.08	0	0	0	0	0	.06	.02	.02
110	2.04	.04	.04	.08	0	0	0	0	0	.04	.02	.02
120	2.08	.04	.04	.08	0	0	0	0	0	.04	.02	.02
	Totals	2.08	2.08		.30	.96	.58	.10	.08	1.90	.76	1.34

DERIVATION OF UNIT HYDROGRAPHS BY THE CLARK METHOD

There are many different unit hydrographs for the same basin because the shapes of the hydrographs vary with different unit storm durations. To define a generalized unit hydrograph for a basin, C.O. Clark developed a technique (reference 1) which uses the concept of the instantaneous unit hydrograph (IUH). This is theoretically the hydrograph that would result from one unit of excess occurring over the basin in a specified areal pattern and zero time. The IUH can then be used to compute a unit hydrograph for any unit duration equal to or greater than the time interval used in the computations.

The Clark method translates incremental runoff from subareas within a basin to the basin outflow location according to travel times and then routes this runoff through a linear reservoir in order to account for the storage effects of the basin and channels. The time of concentration (t_c) is defined as the travel time of water particles from the most upstream point (timewise) in the basin to the outflow location. This lag time may be estimated by measuring the time between the end of effective rainfall and snowmelt over the basin and the inflection point on the recession limb of the surface runoff hydrograph, as illustrated in figure 1. When the time of concentration has been determined, the basin is divided into incremental runoff-producing areas that have equal travel times to the outflow location. The distance from the most upstream point in the basin is measured along the principal watercourse to the outflow location. Dividing this distance by t_c gives the rate of travel or the distance traveled in unit time. Isochrones representing equal travel time to the outflow location are laid out using the distance traveled per unit time to establish the location of the lines. The increment of time used to subdivide the basin need only be small enough to adequately define the arel distribution of runoff. The areas between the isochrones are then measured and tabulated with the corresponding travel time (from O to t_c) for each incremental area.

The time period selected as the computation interval should be approximately equal to the unit duration of excess. A plot of percent of length/versus accumulative area is useful in determining time-area relationships. Such a curve facilitates rapid development of unit hydrographs for various computation intervals and unit durations of excess. This is especially helpful when making flood predictions for basins where t_c is not firmly established, as unit hydrographs may be easily modified to reflect subsequent changes in t_c. Also, it is possible to refine the curve by considering the variation of velocity from stream reach to stream reach and specified contributions of excess (as ratios of basin-mean contribution) in different portions of the basin. Another advantage is that the unit duration can be changed without deriving a new time-area relationship.

The runoff from the contributing areas (between the isochrones) which has been translated to the outflow location is in units of volume (in-mi^2) or mm-km^2) and these must be converted to the proper units of discharge. This conversion is shown below.

$$I_i = K a_i /\Delta t \tag{1}$$

where:

I_i = ordinate in proper units of discharge (cfs or m^3 /s) of the time-area runoff volumes at the end of period i.

a_i = ordinate in units of depth-area of excess (inch-mile^2 or mm/km^2) of the time-area runoff at the end of period i.

K = conversion factor to convert inch-mile^2 /hour to cfs (K = 645) or mm-km^2 /hour to m^3 /s (K=.278).

Δt = time period of computation interval in hours.

The routing of the translated runoff through storage at the outflow location is accomplished as follows:

$$O_i = CI_i + (1 - C) O_{i-1} \tag{2}$$

where:

O_i = outflow from the basin at end of period i in cfs (m^3 /s).

I_i = inflow or runoff from each area at end of period i in cfs (m^3 /s).

C = dimensionless routing constant.

The above routing equation results from setting the Muskingum "X" equal to zero in the coefficient method of routing (reference 2). The routing constant is:

$$C = \frac{2\Delta t}{2R + \Delta t} \tag{3}$$

where:

Δt = time period of computation interval.

R = attenuation constant having the dimension of time.

It can be shown that when inflow into the principal storage reach has ceased (Muskingum "X" = 0),

$$R = -\frac{Q}{dQ/dt} \tag{4}$$

The magnitude of R can be approximately evaluated at the point of inflection of the recession limb of the observed surface runoff hydrograph. The above ratio decreases to a minimum at the point of inflection and, in theory, remains constant thereafter. Therefore, R may be estimated by dividing the ordinate of the surface runoff hydrograph at the point as shown in figure 1. Another technique is to compute the volume of runoff remaining under the recession limb of the surface runoff hydrograph following the point of inflection and divide by the discharge at the same point. In either case, R should be an average value determined and verified with several hydrographs.

The hydrograph that results from routing these flows from the incremental areas is the instantaneous unit hydrograph. The instantaneous unit hydrograph can be converted to a unit hydrograph of a unit duration Δt by simply averaging two instantaneous unit hydrographs spaced at interval Δt apart as follows:

$$O_i = O_i \tag{5}$$

$$O_i = 0.5\,(O_i + O_{i-1}) \quad \text{for } i \geq 2$$

The instantaneous unit hydrograph can be converted to a unit graph of some unit duration other than Δt, provided that it is an exact multiple of Δt, by taking n successive averages of the instantaneous unit hydrograph ordinates where n is the multiple of Δt for the desired unit graph duration. The first average is taken of the instantaneous unit hydrograph ordinates, as in equation (5) above and the second average is taken of the just computed O_i 's , the third from the results of the second, etc., repeating the procedure n times in total. The ordinate at any time, i, for a unit graph of duration D and tabulation interval of Δt is:

$$Q_i = 1/n\,(.5O_{i-n} + O_{i-n+1} + \ldots + O_{i-1} + .5O_i) \tag{6}$$

where:

Q_i = ordinate at time i of unit graph of duration D and tabulation interval Δt

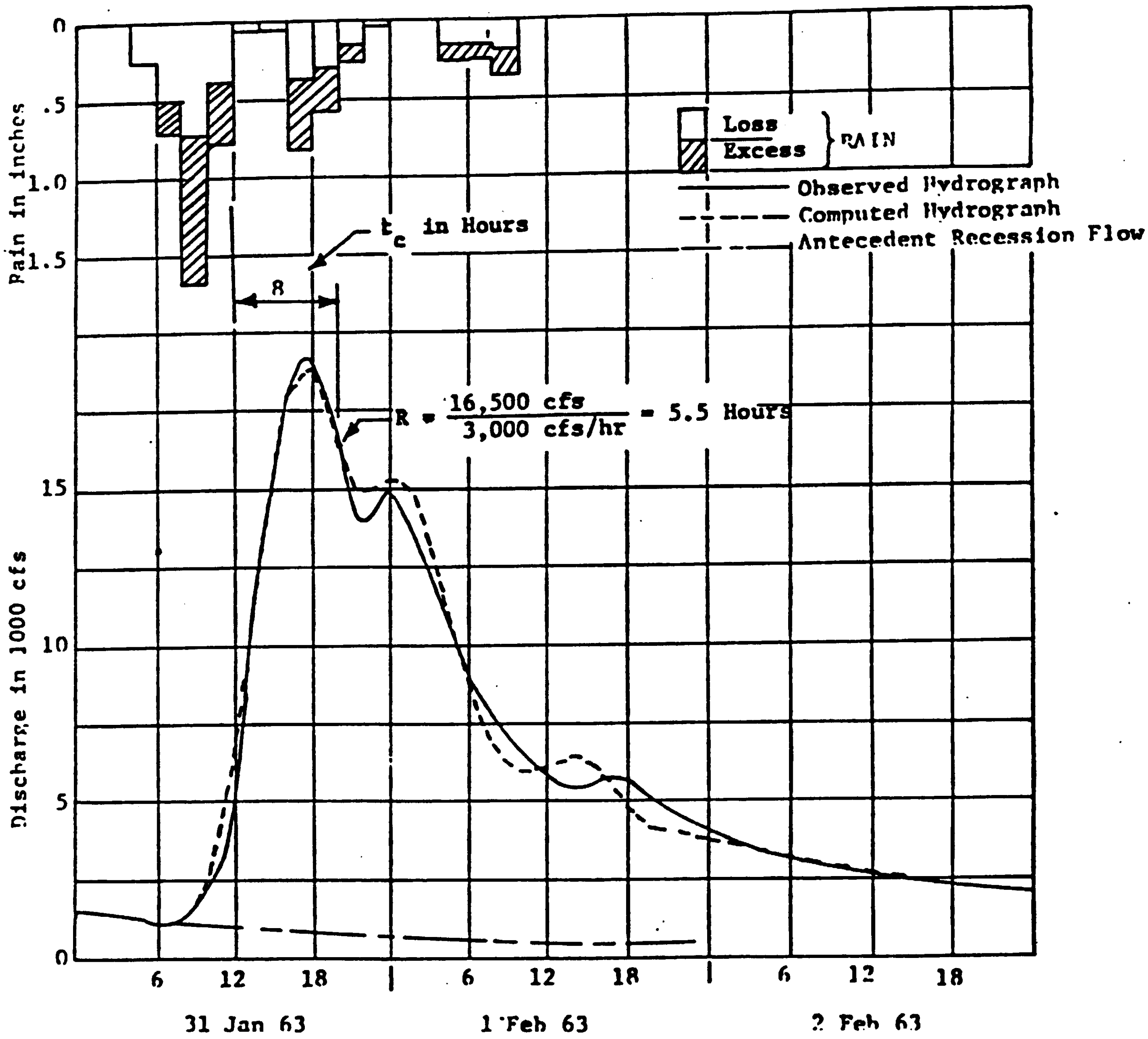
$n = \dfrac{D}{\Delta t}$

D = unit graph duration

Δt = tabulation interval

D/A

Figure 1. Determination of Clark Coefficients and Flood Reconstitution



Drainage Area: 190 sq. mi.

To illustrate the complete Clark procedure, a step-by-step example is worked out for the 31 January-4 February 1963 flood that occurred on Thomes Creek, Paskenta, California, U.S.A.

Step 1

Draw lines (isochrones) which subdivide the basin into a chosen number of parts as illustrated in figure 2. These isochrones are constructed so that the travel time along a water course is the same from one isochrone to another. For simplicity, they are usually drawn equal distances apart from the outflow location to the uppermost head of the basin. The number of isochrones used is ordinarily chosen so that a convenient scale may be used and a reasonable good definition of the time vs. area relation obtained.

Step 2

Measure the areas between each pair of isochrones (figure 2). If a nonuniform pattern of excess is assumed, multiply each area by the average excess within that subdivision.

Step 3

Plot the curve of time vs. area (or excess) as shown on figure 3. Tabulate increments between points one computation interval apart.

Step 4

Convert the interval volume inflows to flow rates (columns 2 and 3 of table 1) using equation (1) so that the total volume equals the unit hydrograph volume corresponding to one unit of runoff.

Step 5

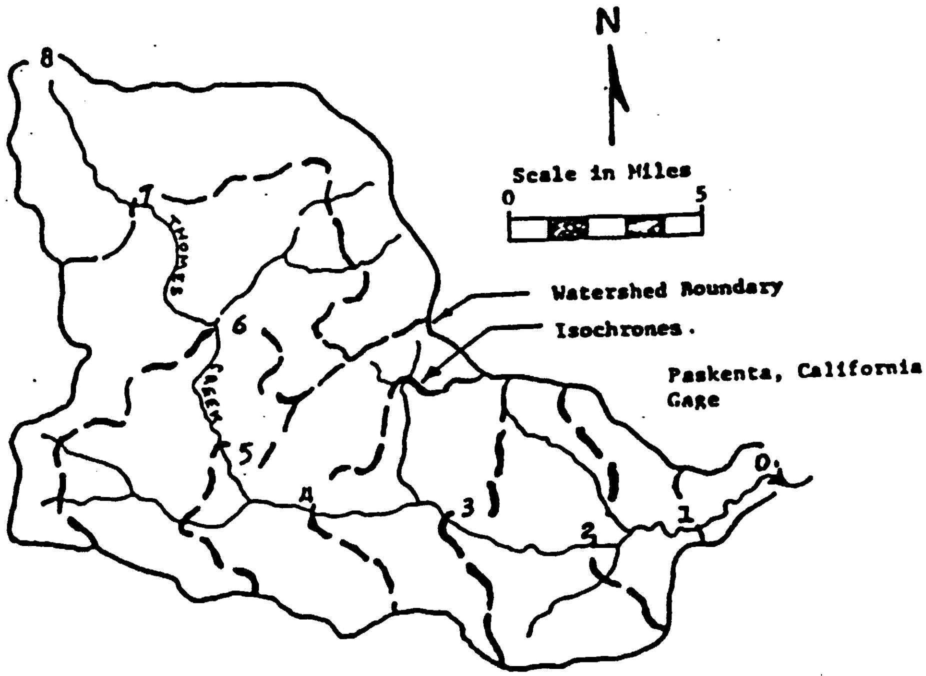
Route the inflows (column 3 of table 1) from step 4 through storage at the outflow location (column 4 of table 1) using equations (2) and (3). This procedure results in the instantaneous unit hydrograph.

Step 6

Average the ordinates of the instantaneous unit hydrograph with those of the same instantaneous unit hydrograph one computation interval, Δt, earlier, equation (5). The resulting hydrograph is the unit hydrograph of duration equal to the computation interval, Δt (2-hour). The 4-hour unit graph is computed by averaging the ordinates of the 2-hour unit graph.

D/6

Figure 2. Computation of the Time-Area Relation



Travel Time from "8" to Gage is 8.0 Hours for the 32 Miles

Map Area Number	Planimeter Values from Map: Incremental units	Planimeter Values from Map: Accumulated units	Accumulated area (sq.mi.) (Col 3)·(58.8)	Travel Time in Percent [(1/8)·(100)]
(1)	(2)	(3)	(4)	(5)
1	0.08	0.08	5	12.5
2	0.15	0.23	14	25.0
3	0.40	0.63	37	37.5
4	0.36	0.99	58	50.0
5	0.45	1.44	85	62.5
6	0.45	1.89	111	75.0
7	0.66	2.55	150	87.5
8	0.68	3.23	190	100.0
Total	3.23			

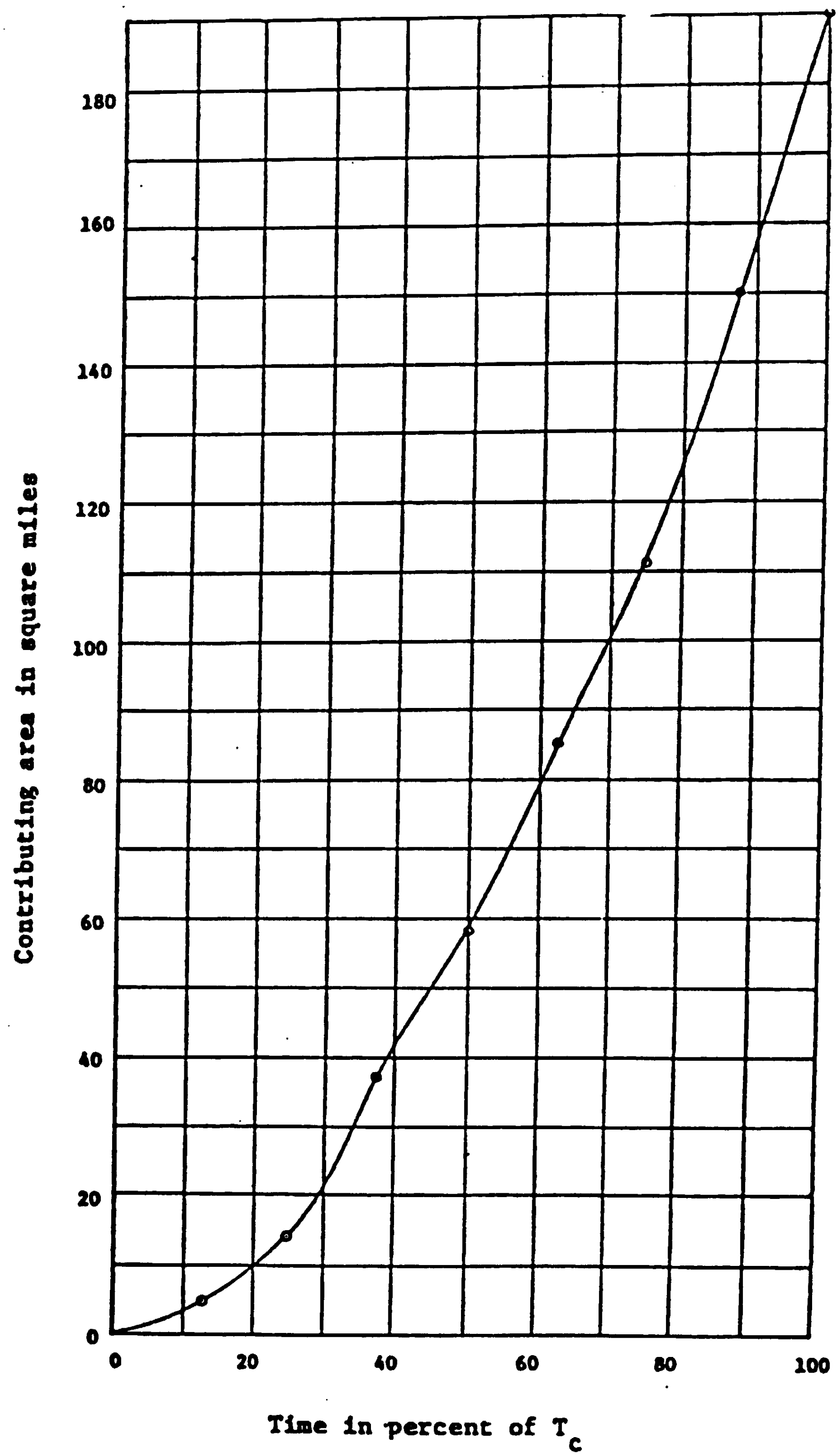
Sq.mi./Planimeter unit = 190/3.23 = 58.8

Drainage Area = 190 square miles

APPENDIX 'D'

D

Figure 3. Watershed Time-Area Relation



APPENDIX 'D

D/8

Table 1. Unit Graph Computation Clark Method

(Thomes Creek at Paskenta, California)

DRAINAGE AREA = 190 SQUARE MILES
TIME OF CONCENTRATION (Tc) = 8.0 HOURS (See Figure 1)
ATTENUATION VALUE (R) = 5.5 HOURS (See Figure 1)
TIME INTERVAL (Δt) = 2.0 HOURS

EQUATIONS (Subscript i refers to current period)

$$I_i = a_i 645/\Delta t$$

$$C = \Delta t/(R + .5\Delta t) = 0.308$$

$$O_i = CI_i + (1-C)O_{i-1}$$

$$Q_i = .5(O_{i-1} + O_i)$$

TIME	INFLOW (Fig. 2)		INSTANTANEOUS UNIT GRAPH	2-HOUR UNIT GRAPH
hr	a_i sq.mi.-in.	I_i cfs	O_i cfs	Q_i cfs
(1)	(2)	(3)	(4)	(5)
0	0	0	0	0
2	14	4,515	1,391	700
4	44	14,190	5,333	3,360
6	53	17,093	8,955	7,150
8	79	25,478	14,043	11,500
10	0	0	9,717	11,880
12			6,724	8,220
14			4,653	5,690
16			3,220	3,940
18			2,228	2,720
20			1,542	1,890
22			1,067	1,300
24			738	900
26			510	630
28			352	430
30			242	300
32			168	200
34			116	140
36			81	100
38			55	70
40			39	50
42			26	30
44			19	20
46			13	20

APPENDIX 'D'

The clark method has two advantages that make it particularly attractive. First, the procedure described herein provides a means of direct computation of unit hydrographs for electronic computer applications. Most other procedures require trial-and-error adjustments of the computed unit hydrograph. Second, the fact that a time-area curve is used provides a means of adjusting objectively for changes in drainage patterns resulting from urbanization or construction of reservoirs, channels, or diversions without requiring that the basin be subdivided into many subareas. This is accomplished simply by constructing a time-area curve (with modified t_c and R) that corresponds to new travel times through reaches and reservoirs.

The Clark unit hydrograph coefficients, t_c and R, are given physical significance in the previous discussion, but in practice, uncertainties of the concepts and of recorded data usually preclude their reliable determination in a simple fashion. It is know that t_c and R are not rigid, and by analyzing several different storms on the same basin, different values will probably be obtained for different storms. For instance, t_c for a storm centered over the head of the basin will probably be larger than one centered over the foot of the basin.

If discharge and rainfall records and snowmelt data are available, t_c and R can be estimated from observed events. As illustrated in figure 1, t_c can be estimated as the time from the end of heavy excess to the inflection point on the recession limb of the flood hydrograph. Likewise, R can be estimated by dividing the discharge at the inflection point by the rate of change of flow at that point on the hydrograph. However, the shapes of hydrographs reflect many irregularities of rainfall, snowmelt and stream patterns, and estimates obtained in this manner are usually satisfactory only for first approximation.

REFERENCES

1. Clark, C.O., "Storage and the Unit Hydrograph," Trans. American Society of Civil Engineers, Vol. 110, pp. 1419-1488, 1945.

2. U. S. Army Corps of Engineers, Engineering and Design, "Routing of Floods through River Channels," EM 1110-2-1408, 1 March 1960.

PROBLEM "A": Plate ".3" depicts a school site proposed at Highway 39- and South Fork Currys Fork in Oldham County, Ky. It is desir to determine the 100-Year flood level to preclude building innundation.

SOLUTION:

(a) A quadrangle mosaic of the watershed is shown on Plate .4. This un-gaged drainage area of 7.52 sq.mi. (see sheet ".6") is composed of rolling to moderately / hilly terrain with pasture and woodland. Not only the lack of stream gage records, but the absence of exten. visual data in this sparsely populated region makes accurate 100-Year discharge determination difficult.

(b) The unitgraph method employed for this study is the "Clark Method" (see A.S.C.E. Transactions, Vol. 110, 1945 & Corps of Engineers EM 1110-2-1408, 1960). In view of the proven accuracy of this method, it appears that the only elemen. subject to question would be that of the "attenuation constant", "R", and a value of 8.0 was selected on the basis of experience. Sheets .5 thru .8 portray unitgraph derivation.

(c) The 100-Year flood hydrograph is tabulated on sheet .9 and plotted on sheet .10. A peak of 1458 cfs at 3.75 hrs was obtained.

(d) It was reported that a flood of record (date unknown, caused overtopping of Hwy. 393 by a slight amount. It is not known to what extent debris clogging of the bridge waterway contributed to overtopping, but in view of the substantial woodland areas in this water-shed, some clogging would appear likely.

(e) Sheets .11 & .12 portrays a channel rating computation and plot. The 1,458 cfs 100-Year flow has a depth of 6.2 feet, and this assumes overbank areas do not convey flow because of dense tree growth on each bank. This computed flood plain is shown on the site plan, sheet .3, and the building floor leve is about 5 feet above the flood plain. In view of the likelihood of channel clogging during future floods and nature's propensity for bringing storm intensitie greater than the "100-Year", this 5-foot margin is desireable.

(f) Sheets .13 & .14 are the determination of bridge swellhead. Although not directly related to the proble under investigation, this engineer desired an indic tion of flow characteristics at the bridge as a general indicator of the reasonableness of the compu

PROBLEM "B": The un-named tributary which flows under the access road to the school site should have a cross-drain of sufficient size to preclude overtopping of the road during a localized 100-Year rainfall pattern, and to preclude undue backwater to the upstream property. The proposed culvert is to be checked for these factors

SOLUTION:

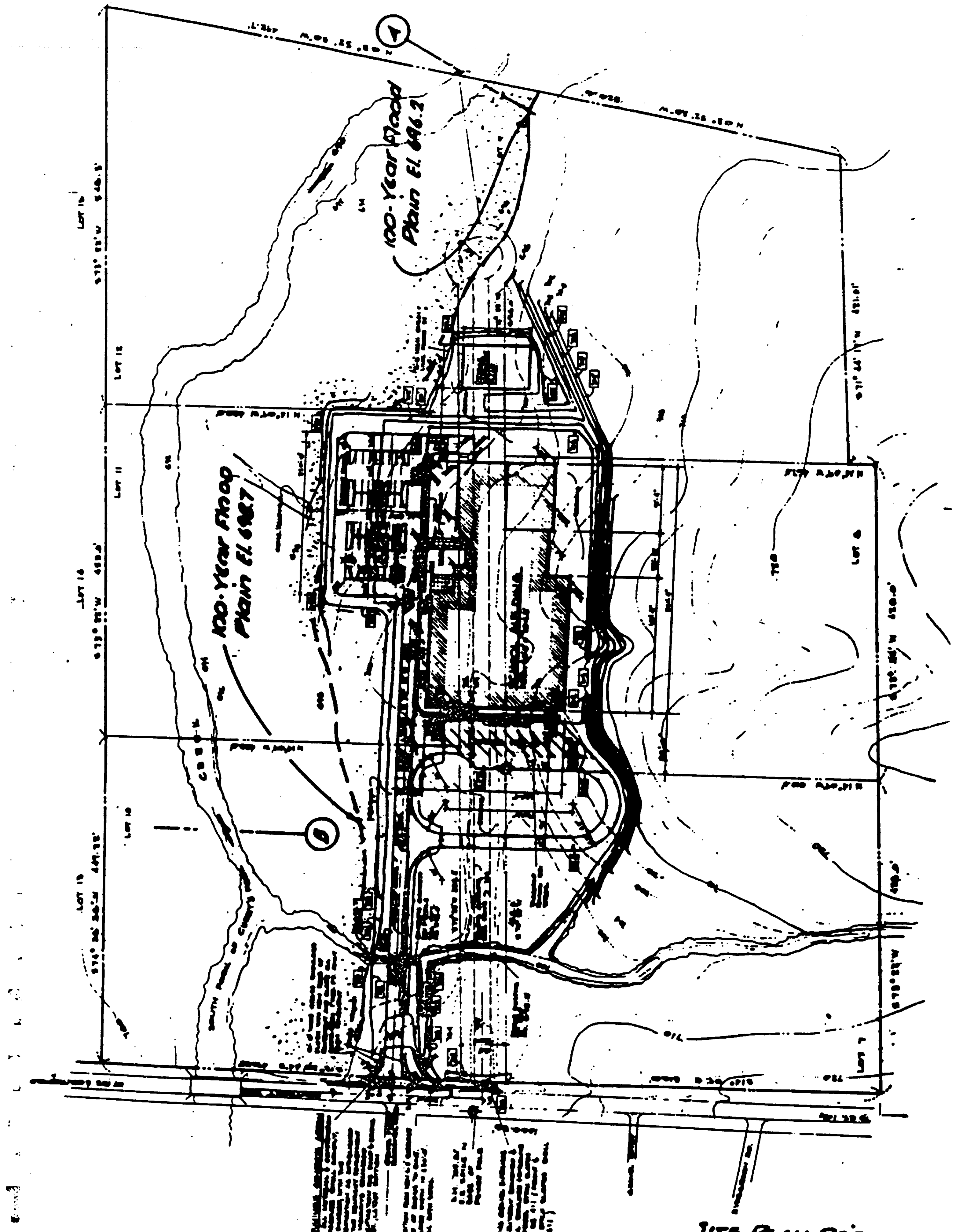
(a) Page .15 portrays the drainage area determination of 1928 acres. In view of the undeveloped state of this watershed and its' relatively small size, it was considered appropriate to use the Rational Method to determine the 100-Year flow of 216 cfs.

(b) Page .16 portrays the existing 72"φ culvert configuration in relation to the main stem. Since there is no backwater, the steady flow analysis indicates a culvert discharge of 259 cfs when flowing full. This is 43 cfs greater than the 100-Year. A head-water elevation thus produced is 0.4' below the existing road.

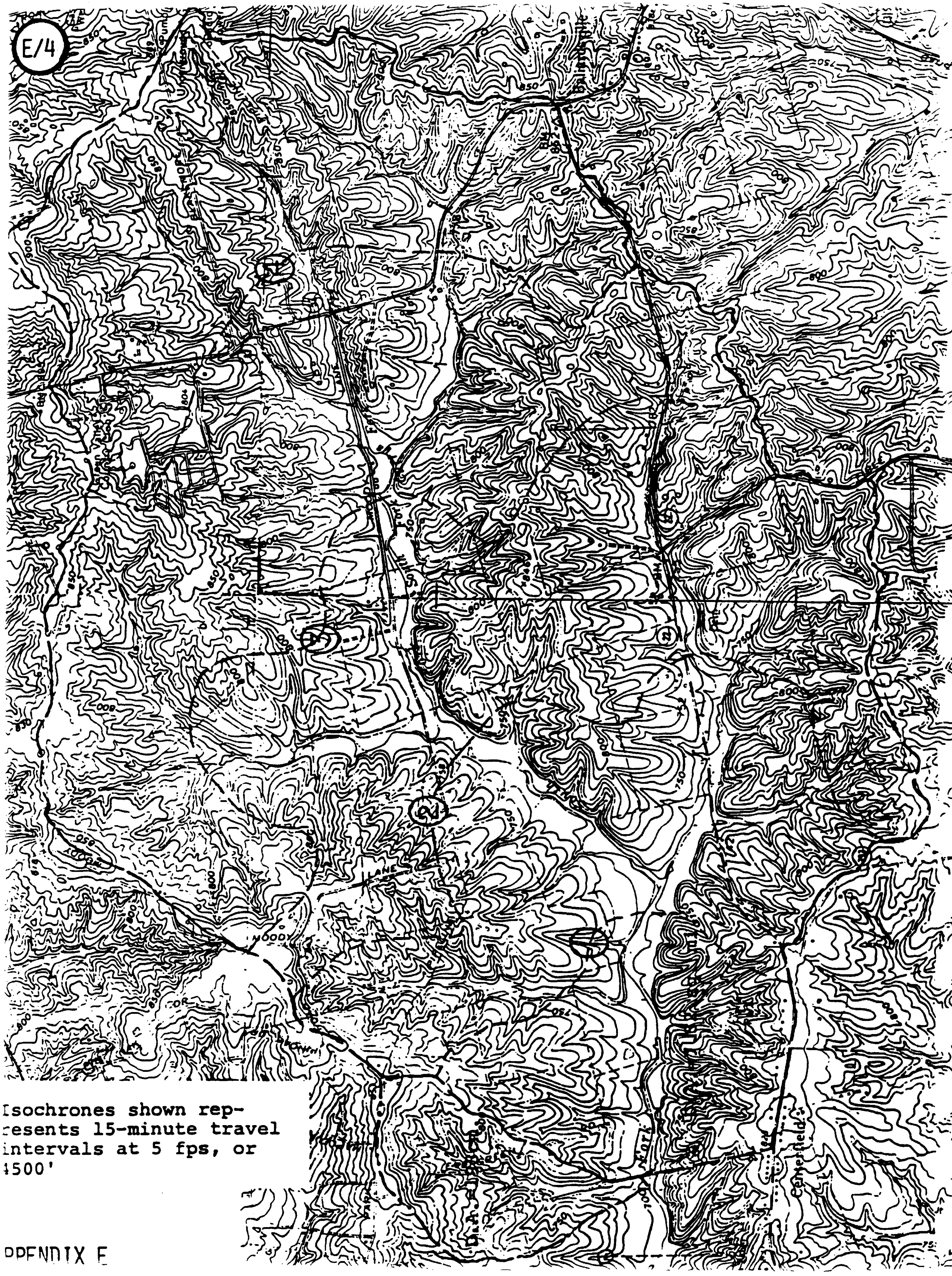
(c) The existing 72"φ RCP is hydraulically adequate, thus verifying the abscence of any washes on the road.

(d) Despite the upstream woodland, this un-attended pipe remains clear of debris. In view of its large diameter, there appears to be no hydraulic need for a trash structure, nor should there be any need on a comparable replacement.

(e) Since the existing road is to be re-alligned and the surface lowered, a new multi-barreled, low-head culvert will be needed. Page .17 portrays the design of (2) 43"x 68" ellipticals which will convey 250 cfs with only 0.15' of access road overtopping. This assures no overtopping under 100-Year, 216 cfs conditions. Because of the 68" width, it appears that trash structures are not needed.



SITE PLAN DEPICTING



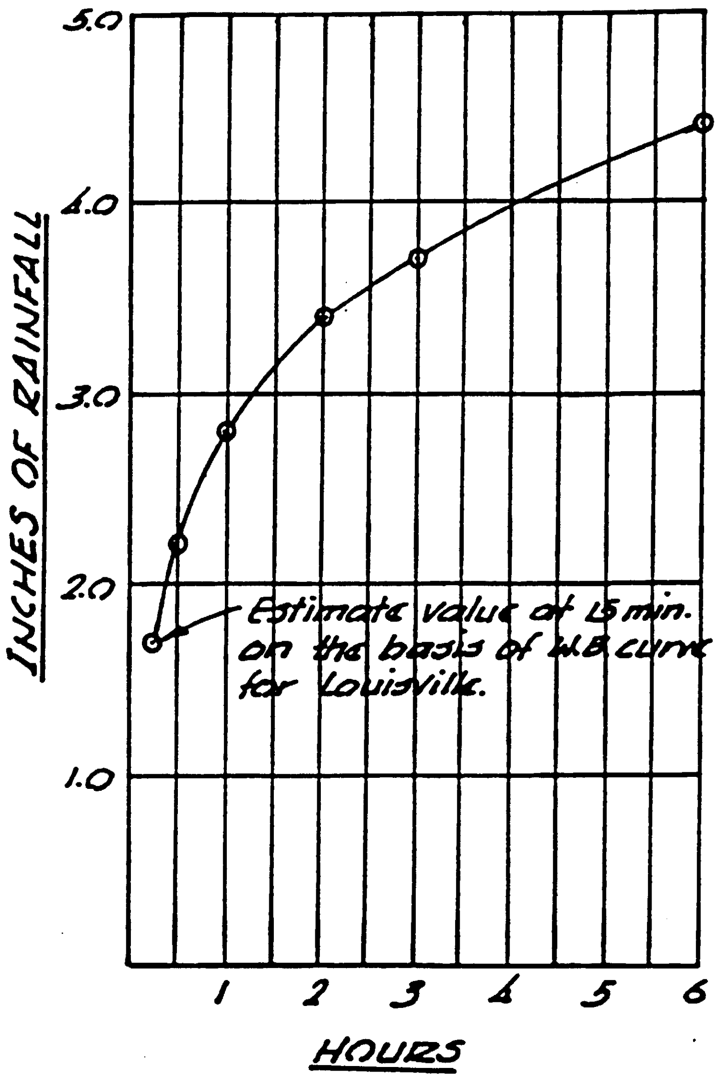
Isochrones shown represents 15-minute travel intervals at 5 fps, or 4500'

APPENDIX E

DAVID L. DAUGHERTY
August 1976

100-YEAR RAINFALL - OLDHAM CO., KY.

Note: Values designated with an asterisk are taken from "Rainfall Frequency Values for Kentucky" dated 1971.



HOURS	CUMULATIVE RAIN	INCREMENT RAIN
.25	1.70"	1.70"
.50	2.20"*	.50"
.75	2.56"	.36"
1.0	2.80"*	.24"
1.25	2.98"	.18"
1.50	3.14"	.16"
1.75	3.28"	.14"
2.0	3.40"*	.12"
2.25	3.49"	.09"
2.50	3.56"	.07"
2.75	3.63"	.07"
3.0	3.70"*	.07"
3.25	3.77"	.07"
3.50	3.84"	.07"
3.75	3.91"	.07"
4.0	3.98"	.07"
4.25	4.04"	.06"
4.50	4.10"	.06"
4.75	4.15"	.05"
5.0	4.20"	.05"
5.25	4.25"	.05"
5.50	4.30"	.05"
5.75	4.35"	.05"
6.0	4.40"*	.05"

DRAINAGE AREA OF SOUTH FORK CURRYS FORK AT HWY. 393

Planimeter Values : Trial #1 - 5240 units
Trial #2 - 5244 units
Use - 5242 units

Conversion : For 2000 scale map, use $\frac{100}{2000^2} = \frac{Value}{D.A.}$

Drainage Area = 40,000 × 5242 = 209,680,000 sf
= 4,813.6 acres
= 7.52 sq.mi.

TIME OF CONCENTRATION "T_c"

Overland : 1000' at 1.5 fps = 11.1 min.
Streamflow: 21000' at 4.0 fps ave. = 87.5 min.
Total Estimated Travel Time = 98.6 min. for 100-Yr. intensity

DESCRIPTION OF TERRAIN

Un-developed, pasture & woodland

"CLARK METHOD" UNITGRAPH COEFFICIENTS

D.A. = 7.52 sq.mi.
T_c = 1.64 hrs.
R (attenuation value) = Use 8.0 (based on engineering experience
Δt = 0.25 hrs. (unitgraph time interval)

Isoch

Isochrone Number	Planimeter Units	Incremental Area (k=.001435) (Sq.Mi.)	Cumulative Area (Sq.Mi.)	Travel Time in Percent
1	460	0.66	0.66	20
2	872	1.25	1.91	40
3	1400	2.01	3.92	60
4	1475	2.12	6.04	80
5	1038	1.48	7.52	100

Note: Same data tabulated on following page in graphical form.

HYDRAULIC STUDY
OLDHAM CO. SCHOOL

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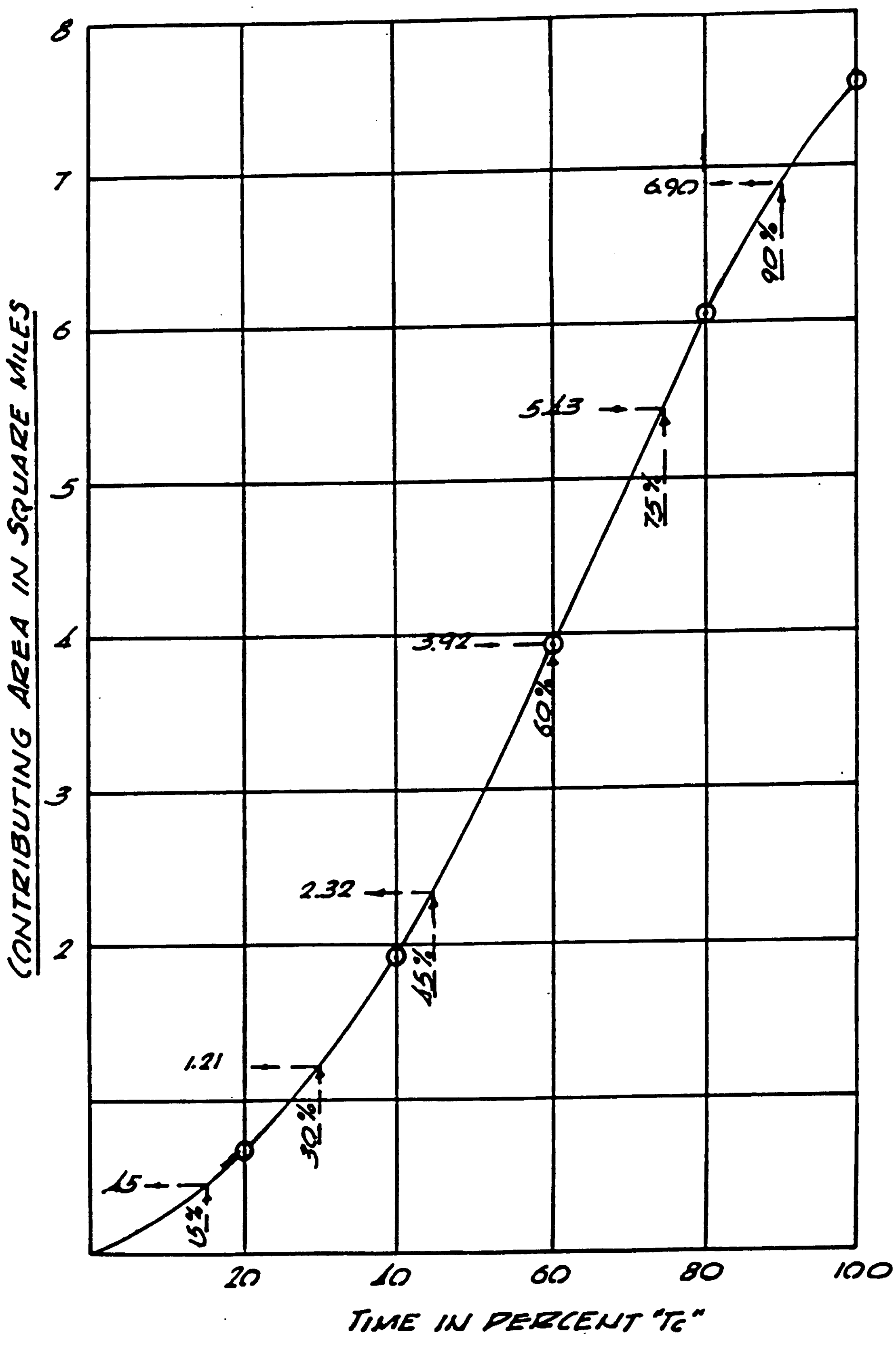


UNITGRAPH DETERMINATION - "CLARK METHOD"

D.A. = 7.52 sq.mi.
T_c = 98.6 min. = 1.64 hrs.
R = 8.0
Δt = 15 min. = 0.25 hrs.

$I_i = a_i\, 645/\Delta t = 2616\, a_i$
$C = \Delta t/(R + .5\Delta t) = .0308$
$O_i = C I_i + (1-C) O_{i-1} = .0308 I_i + .9692 O_{i-1}$
$Q_i = .5\,(O_{i-1} + O_i)$

TIME	Σa_i from curve	a_i	I_i	O_i	Q_i (Unitgraph)
.25	.45	.45	1177.2	36	18
.50	1.21	.76	1988.2	96	66
.75	2.32	1.11	2903.8	182	139
1.0	3.92	1.60	4185.6	305	244
1.25	5.43	1.51	3950.2	417	361
1.50	6.90	1.47	3845.5	523	470
1.75	7.52	0.62	1621.9	557	540
2.00				540	548
2.25				523	531
2.50				507	515
2.75				491	499
3.0				476	484
3.25				462	469
3.50				448	455
3.75				434	441
4.0				421	427
4.25				408	414
4.50				395	402
4.75				383	389
5.0				371	377
5.25				360	365
5.50				349	355
5.75				338	343
6.0				328	333
7.0				289	
8.0				255	
9.0				225	
10.0				199	
11.0				175	
12.0				154	
14.0				120	
16.0				93	
18.0				72	

DAVID L. DAUGHERTY
August 1976

E/

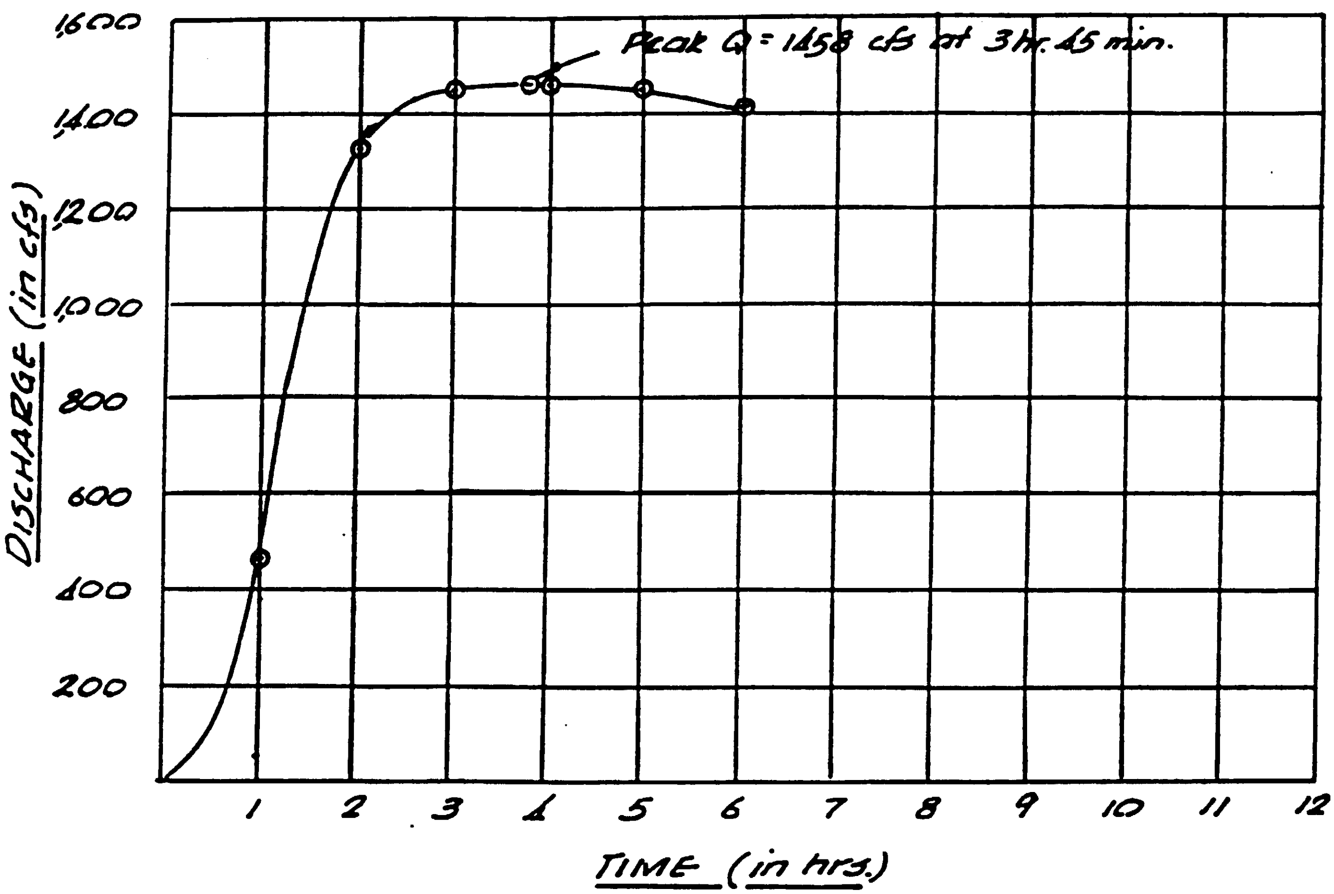
100-YEAR FLOOD HYDROGRAPH

Time (hrs.)	.25	.50	.75	1.0	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00	4.25	4.50	4.75		Time (in hrs.)
Rainfall	1.70	0.50	0.36	0.24	0.18	0.16	0.14	0.12	0.09	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.06	0.06	0.05		
Infiltration	0.10	0.10	0.10	0.10	0.05	0.04	0.03	0.02	0	0	0	0	0	0	0	0	0	0	0		
Excess	1.60	0.40	0.26	0.14	0.13	0.12	0.11	0.10	0.09	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.06	0.06	0.05		
Unitgraph																					
18	29																			29	
66	106	7																		113	
139	222	26	5																	253	
244	390	56	17	3																466	1.0
361	578	98	36	9	2															723	
470	752	144	63	19	9	2														989	
540	864	188	94	34	18	8	2													1208	
548	877	216	122	51	32	17	7	2												1324	2.0
531	850	219	140	66	47	29	15	7	2											1375	
515	824	212	142	76	61	43	27	14	6	1										1406	
499	798	206	138	77	70	56	40	24	13	5	1									1428	
484	774	200	134	74	71	65	52	36	22	10	5	1								1444	3.0
469	750	194	130	72	69	66	59	47	32	17	10	5	1							1452	
455	728	188	126	70	67	64	60	54	42	25	17	10	5	1						1457	
441	706	182	122	68	65	62	58	55	49	33	25	17	10	5	1					1458	Peak
427	683	176	118	66	63	60	57	53	49	38	33	25	17	10	5	1				1454	4.0
414	662	171	115	64	61	58	55	52	48	38	38	33	25	17	10	5	1			1453	
402	643	166	111	62	59	56	53	50	46	37	38	38	33	25	17	10	4	1		1449	
389	622	161	108	60	57	55	52	48	45	36	37	38	38	33	25	17	8	4	1	1445	
377	603	156	105	58	56	53	50	47	44	35	36	37	38	38	33	25	15	8	4	1442	5.0
365	584	151	101	56	54	51	49	46	42	34	35	36	37	38	38	33	22	15	8	1435	
355	568	146	98	54	52	50	47	44	41	33	34	35	36	37	38	38	28	22	15	1429	
343	549	142	95	53	51	48	46	43	40	32	33	34	35	36	37	38	32	28	22	1422	
333	533	137	92	51	49	47	44	41	38	31	32	33	34	35	36	37	33	32	28	1413	6.0

Note: Hydrograph values at 5.0 thru 6.0 hrs. are compiled to include appropriate 15 min. increments not shown because of space limitations.

APPENDIX

100-YEAR HYDROGRAPH



HYDRAULIC STUDY
OLDHAM CO. SCHOOL

DAVID L. DAUGHERTY
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E
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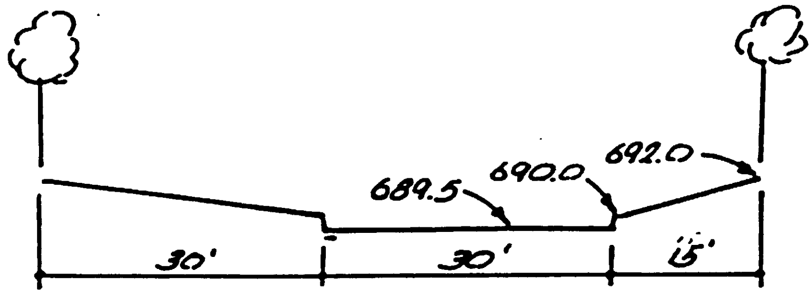
SECTION "A" - SOUTH FORK CURRYS FORK



Note: Trees on each bank offer such a major impediment to flow that the engineer considers them as barriers to overbank discharge inclusion in any computations

Depth	Area	Wetted Perimeter	R (A/W.P.)	$R^{2/3}$
2.5'	120 sf	80'	1.500	1.310
6.0'	382.5 sf	87'	4.397	2.685
10.0'	682.5 sf	95'	7.184	3.726

SECTION "B" - (South Fork Currys Fork)

Located 1200' upstream of Section "A".
Channel invert elevation 693.0.
Essentially same cross-section as that at "A".

CHANNEL RATING

$$Q = A \times \frac{1.486}{n} \times R^{2/3} \times S^{1/2}$$

where: A = flowage area
n = channel roughness (use
R = hydraulic radius
S = energy gradient slop
= 35/1200 = .0029

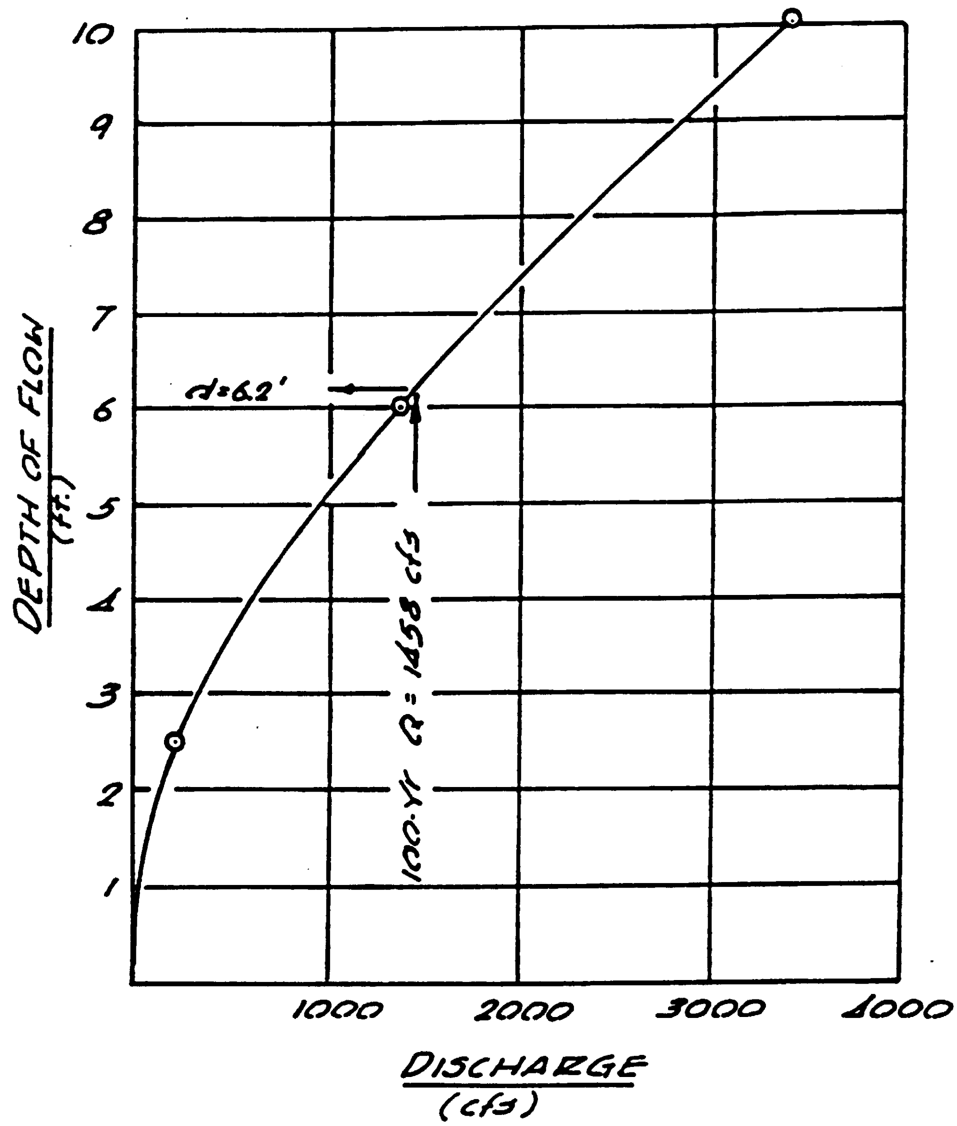
Note: The energy gradient slope "S" is equal to the channel slope for steady flow in the absence of backwater effects. Peak flow is steady & no major backwater is assumed.

$$Q_1 = 120 \times \frac{1.486}{.06} \times 1.310 \times .0029^{1/2} = 210 \text{ cfs}$$

$$Q_2 = 382.5 \times \frac{1.486}{.06} \times 2.685 \times .0029^{1/2} = 1374 \text{ cfs}$$

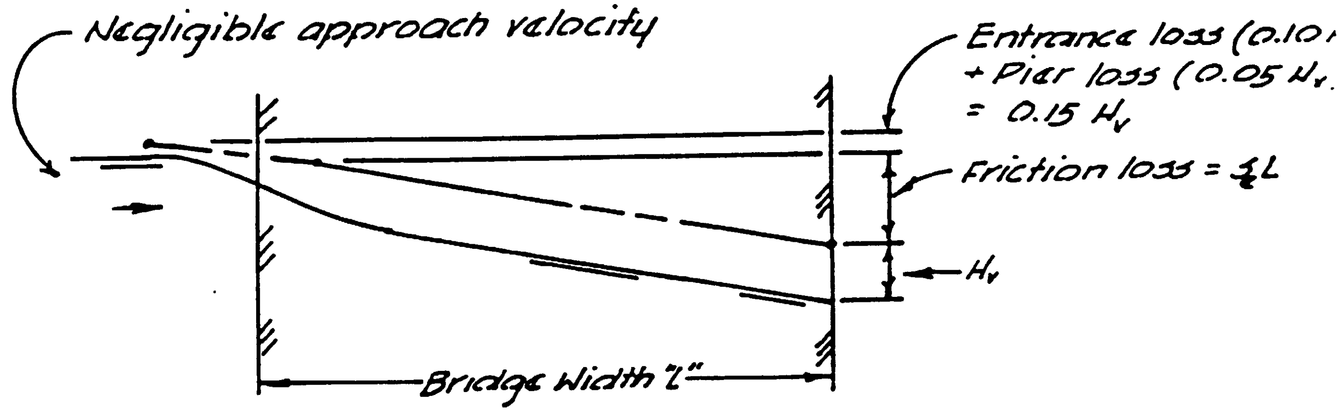
$$Q_3 = 682.5 \times \frac{1.486}{.06} \times 3.726 \times .0029^{1/2} = 3400 \text{ cfs}$$

CHANNEL RATING (cont'd)



SWELLHEAD THROUGH A BRIDGE*

* (Where the velocity of approach is negligible in comparison to velocity thru the structure.)



Swellhead = (SW) = $H_v + S_e L + 0.15\,H_v$

(SW) = $S_e L + 1.15\,H_v$

where: $S_e = \dfrac{n^2 Q^2}{A^2 \times 1.486^2 \times R^{4/3}}$ (Mannings Equation)

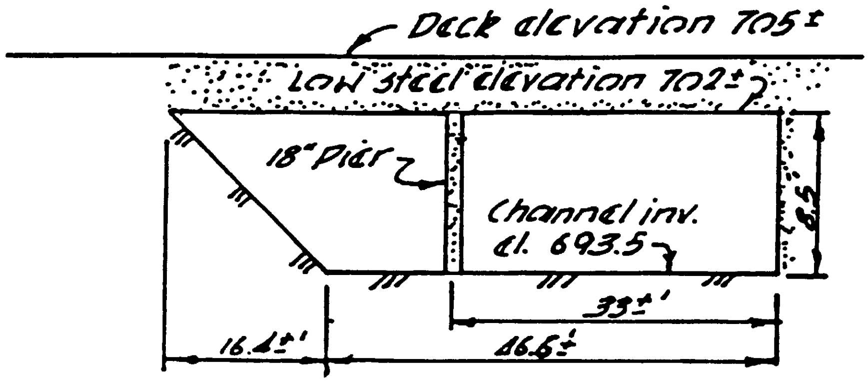
$H_v = \dfrac{V^2}{2g} = \dfrac{Q^2}{2g A^2}$

(SW) = $\dfrac{n^2 Q^2 L}{A^2 \times 2.21 \times R^{4/3}} + \dfrac{1.15\,Q^2}{2g\,A^2}$

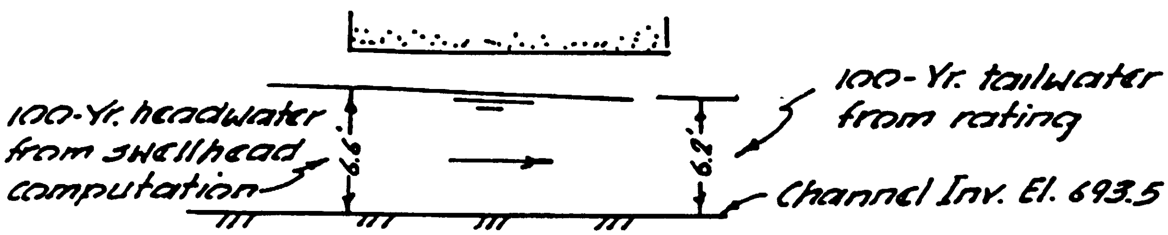
(SW) = $\left(\dfrac{Q^2}{A^2}\right)\left(\dfrac{n^2 L}{2.21\,R^{4/3}} + .0179\right)$

where: (SW) = Headwater - Tailwater
Q = Discharge
A = Flowage area at tailwater (for simplicity)
n = Bridge roughness
L = Bridge width with stream axis
R = Hydraulic radius

APPROXIMATE BRIDGE SECTION AT HWY 393



BRIDGE SWELLHEAD



$$(SW)_{100\text{-}Yr.} = \left(\frac{Q^2}{A^2}\right)\left(\frac{n^2 L}{2.21\, R^{4/3}} + .0179\right)$$

$$= (21.02)\left(\frac{.02^2 \times 30}{2.21 \times 7.14} + .0179\right)$$

$$= .40'$$

where: $Q = 1458$ cfs

$A = (6.2 \times 32.25) + (6.2 \times 17.85) + (6.2 \times \frac{12.4}{2}) = 318$ sf

$n = .020$

$L =$ say $30'$

$R = 318/(6.2 + 32.25 + 6.2 + 6. + 12.85 + 8.8)$

$= 4.386$

DRAINAGE AREA OF UN-NAMED TRIBUTARY TO SOUTH FORK CURRYS FORK IMMEDIATELY WEST OF HIGHWAY 393

Planimeter Values: Trial #1 - 200 units
Trial #2 - 220 "
Trial #3 - 210 "
} Use 210 units

Conversion: For 2000 scale map, use $\frac{100}{2000^2} = \frac{\text{Value}}{\text{D.A.}}$

Drainage Area: 40,000 × 210 = 8,400,000 sf
= 192.8 acres

TIME OF CONCENTRATION "T_c"

Overland: 1400' @ 0.75 fps = 31.1 min.
Streamflow: 4200' @ 4 fps = 17.5 min.
Total est. travel time = 48.6 min. - Use 49 min.

DESCRIPTION OF TERRAIN

Un-developed, pasture & woodland

RATIONAL METHOD

Intensity "I" of 100-Yr. storm (Louisville Station, U.S.W.B. T.P.40
at time 49 min. = 3.20 in./hr.

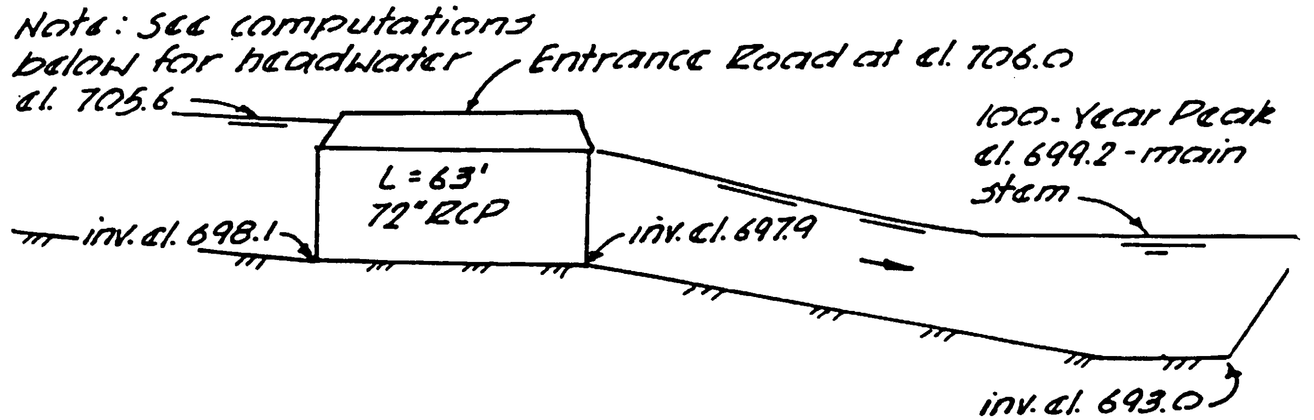
C = 0.35

A = 192.8

Q = CIA = 0.35 × 3.20 × 192.8

Q = 216 cfs

EXISTING FLOW CONDITIONS AT CULVERT UNDER ENTRY



Discharge - 72"φ flowing full

A = 28.26 s.f.

n = 0.012

$R = \frac{28.26}{18.84} = 1.50$ $\qquad R^{2/3} = 1.311$

$S = \frac{0.2}{63} = .0032$ $\qquad S^{1/2} = 0.0565$

$Q = \frac{A \times 1.486}{n} R^{2/3} S^{1/2} = 259$ cfs

$V = \frac{Q}{A} = 9.16$ fps

$H_v = \frac{V^2}{2g} = 1.30'$

$0.15\, H_v = 0.20$ (entrance loss)

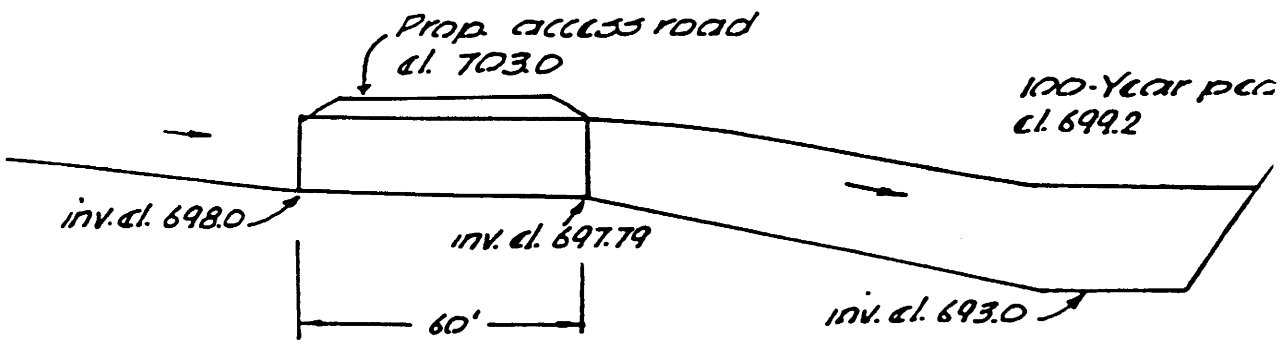
Headwater Determination (neglecting velocity of appr.)

Inv. El. 698.1 + dia. + H_v + 0.15 H_v = HW

Headwater = 698.1 + 6.0' + 1.30' + 0.20 = 705.6

Headwater will be 0.4' below road for a discharge of 259 cfs. The 100-Year flow is computed to be 216 cfs.

PROPOSED ENTRY CULVERT



pipe slope = $\frac{0.21}{60}$ = .0035

For simplicity, refer to "Concrete Pipe Design Manual", 1970 edition, page 186.

(1) 48"x76" elliptical @ .0035 = 155 cfs
(1) 43"x68" " " = 125 cfs

Try (2) 43"x68" ellipticals = 250 cfs

Headwater = $1.15\,H_v + d + 698.0$

Where: $A = (2) \times 15.896 = 31.79$ sf
$Q = 250$ cfs
$V = \frac{Q}{A} = 7.86$ fps
$H_v = \frac{V^2}{2g} = 0.96$ say 1.0'

Headwater = 1.15 x 1.0 + 4.0' + 698.0 = 703.15

Since the headwater would only be 0.15' (i.e.-703.15-703.0) over the road for a Q=250 cfs, there would be no overtopping under 100-Year Q=216 cfs.

Use (2) 43"x68" elliptical RCP's.

BRECKINRIDGE ESTATES RETENTION BASIN

TOTAL HYDROGRAPHS FOR NORTH AND EAST AREAS

TIME	0	5	10	15	20	25	30	35	40	45	50	55	60	65	70
21 July 73 Rainfall		.38	.40	.40	.20	.07	.17	.10	.04	.17	.05	.04	.04		
100-Yr. Rainfall		1.00	.56	.38	.24	.13	.07	.07	.07	.07	.08	.09	.12		

NOTE: The foregoing rainfall increments are transcribed from a preceding page. The <u>effective</u> rainfall increments are not duplicated here.

21 July 1973 Storm

TIME	0	5	10	15	20	25	30	35	40	45	50	55	60	65	70
EAST AREA	0	10	62	167	274	309	288	269	258	235	221	195	178	139	99
NORTH AREA	0	60	218	333	301	205	210	184	109	147	122	69	47	27	12
TOTAL	0	70	280	500	575	514	498	453	367	382	343	264	225	166	111

100-Yr Storm

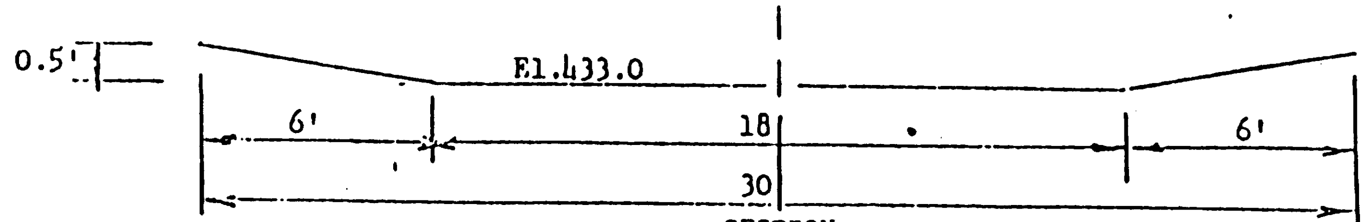
EAST AREA	0	60	230	448	552	474	382	356	306	255	216	193	179	146	113
NORTH AREA	0	345	539	573	474	327	201	117	76	56	49	58	79	54	26
TOTAL	0	405	769	1021	1026	801	583	473	382	311	265	251	258	200	139

F

BRECKENRIDGE ESTATES RETENTION BASIN

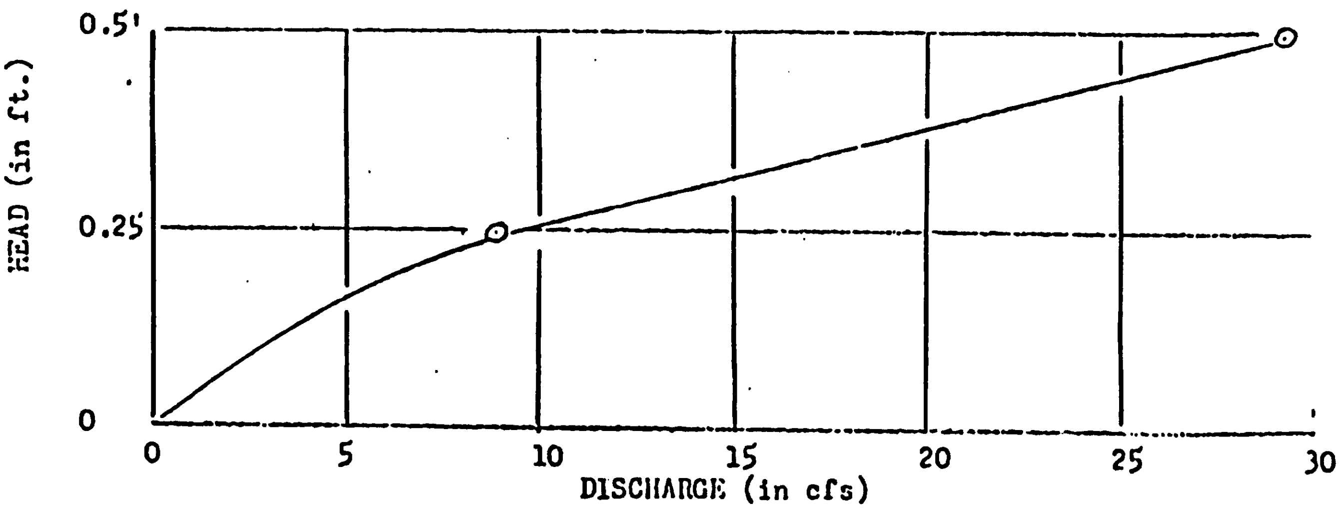
SPILLWAY RATING

F/2



SECTION

$$Q = CLH^{1.5}$$
$$Q = 3.5 \times 21 \times (.25)^{1.5} = 8.8 \text{ cfs for } H = 0.25'$$
$$Q = 3.5 \times 24 \times (.50)^{1.5} = 29.4 \text{ cfs for } H = 0.50'$$



BRECKENRIDGE ESTATES RETENTION BASIN

STORAGE CURVE DATA

Basin Depth (in ft.)	Basin Storage (in cu. ft.)	Computer Data Registe
0.2	30	01
0.4	50	02
0.6	75	03
0.8	125	04
1.0	200	05
1.2	400	06
1.4	700	07
1.6	1,100	08
1.8	1,650	09
2.0	2,400	10
2.2	3,400	11
2.4	4,700	12
2.6	6,300	13
2.8	8,200	14
3.0	10,000	15
3.2	13,000	16
3.4	15,500	17
3.6	16,500	18
3.8	22,000	19
4.0	25,000	20
4.2	29,000	21
4.4	34,000	22
4.6	38,000	23
4.8	43,000	24
5.0	48,000	25
5.2	55,000	26
5.4	62,000	27
5.6	69,000	28
5.8	76,000	29
6.0	85,000	30
6.2	94,000	31
6.4	103,000	32
6.6	112,000	33
6.8	122,000	34
7.0	133,000	35
7.2	145,000	36
7.4	157,000	37
7.6	172,000	38
7.8	185,000	39
8.0	200,000	40
8.2	216,000	41
8.4	230,000	42
8.6	250,000	43
8.8	265,000	44
9.0	280,000	45
9.2	295,000	46
9.4	310,000	47
9.6	330,000	48
9.8	345,000	49
10.0	355,000	50

BRECKINRIDGE ESTATES RETENTION BASIN

RESERVOIR ROUTINGS

F/4

TIME min.	INFLOW cfs	AVERAGE INFLOW cfs	d ft.	Q_{24} cfs	(d-4.85) d_{54} ft.	Q_{54} (two) cfs	(d-9.55) d_{splwy} ft.	Q_{splwy} cfs	Q_{total} cfs	S storage cu.ft.
NORTH & EAST INFLOWS 21 July 73 Storm										
0	0									
		35								
5	70		2.74	19	-	-	-	-	19	7,650
		175								
10	280		5.09	40	negli.	-	-	-	40	51,300
		390								
15	500		7.23	51	2.38	52	-	-	103	146,850
		538								
20	575		8.77	57	3.92	142	-	-	199	262,950
		545								
25	514		10+ (Overflows top of dam)							
ASSUMED EAST AREA INFLOW ONLY 21 July 73 Storm										
0	0									
		5								
5	10		1.44	4.8	-	-	-	-	4.8	750
		36								
10	62		2.76	20	-	-	-	-	20	7,830
		115								
15	167		4.4	35	-	-	-	-	35	34,080
		221								
20	274		6.0	45	1.15	16	-	-	61	85,980
		292								
25	302		7.2	51	2.35	56	-	-	107	148,380
		299								
30	288		8.0	54	3.15	98	-	-	152	199,230
		279								
35	269		8.4	55	3.55	120	-	-	175	233,880
		264								
40	258		8.7	57	3.85	140	-	-	197	257,280
		247								
45	235		8.9	57	4.05	152	-	-	209	270,480
		228								
50	221		9.0	57	4.15	156	-	-	213	275,580
		208								
55	195		8.9	57	4.05	152	-	-	209	274,680

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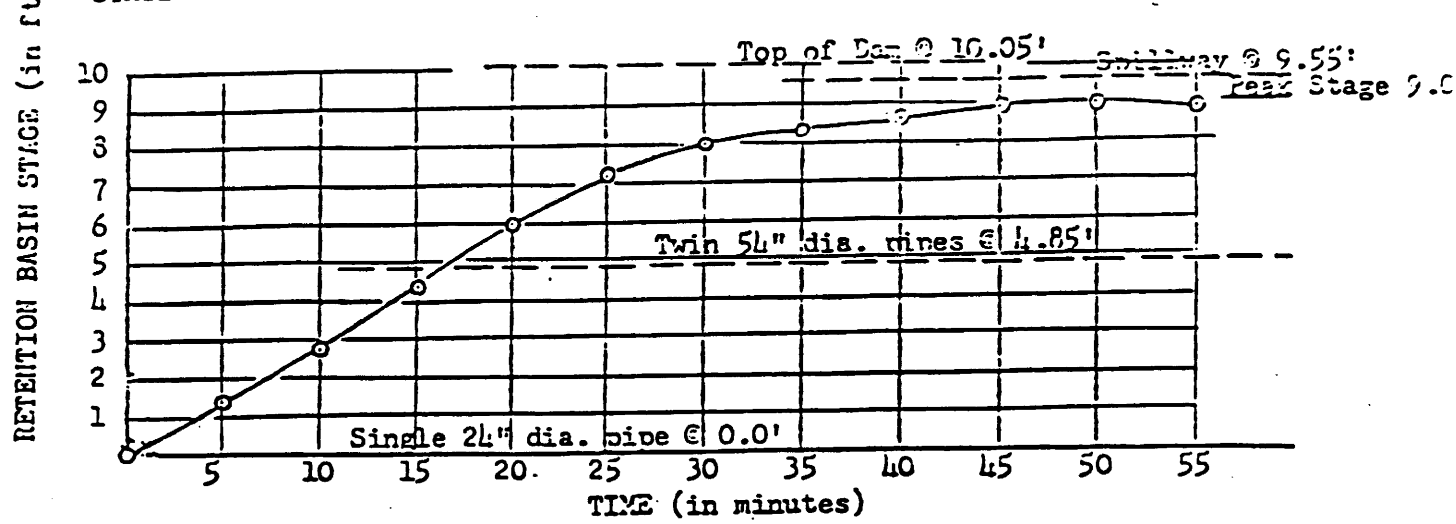
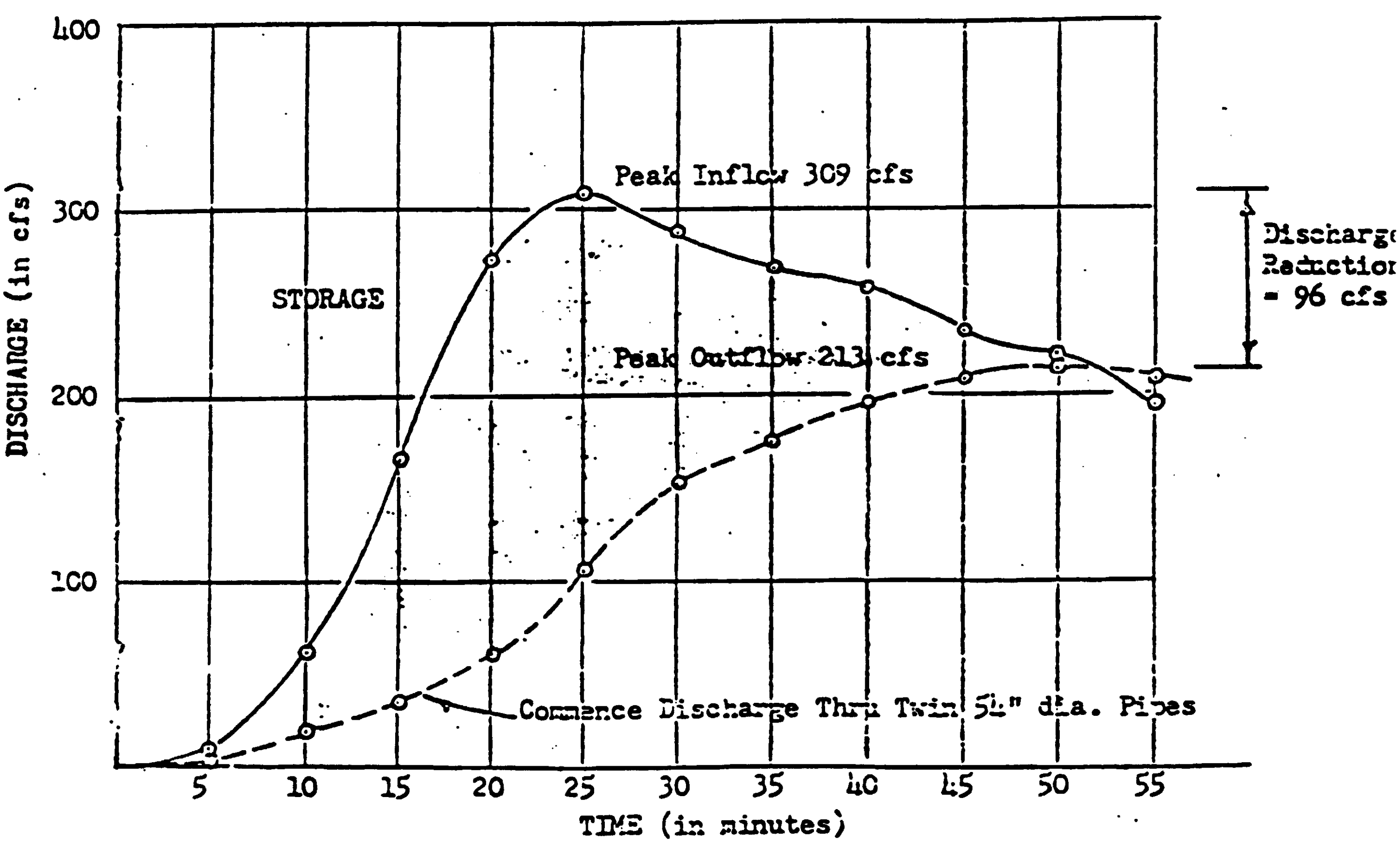
BRECKINRIDGE ESTATES RETENTION BASIN

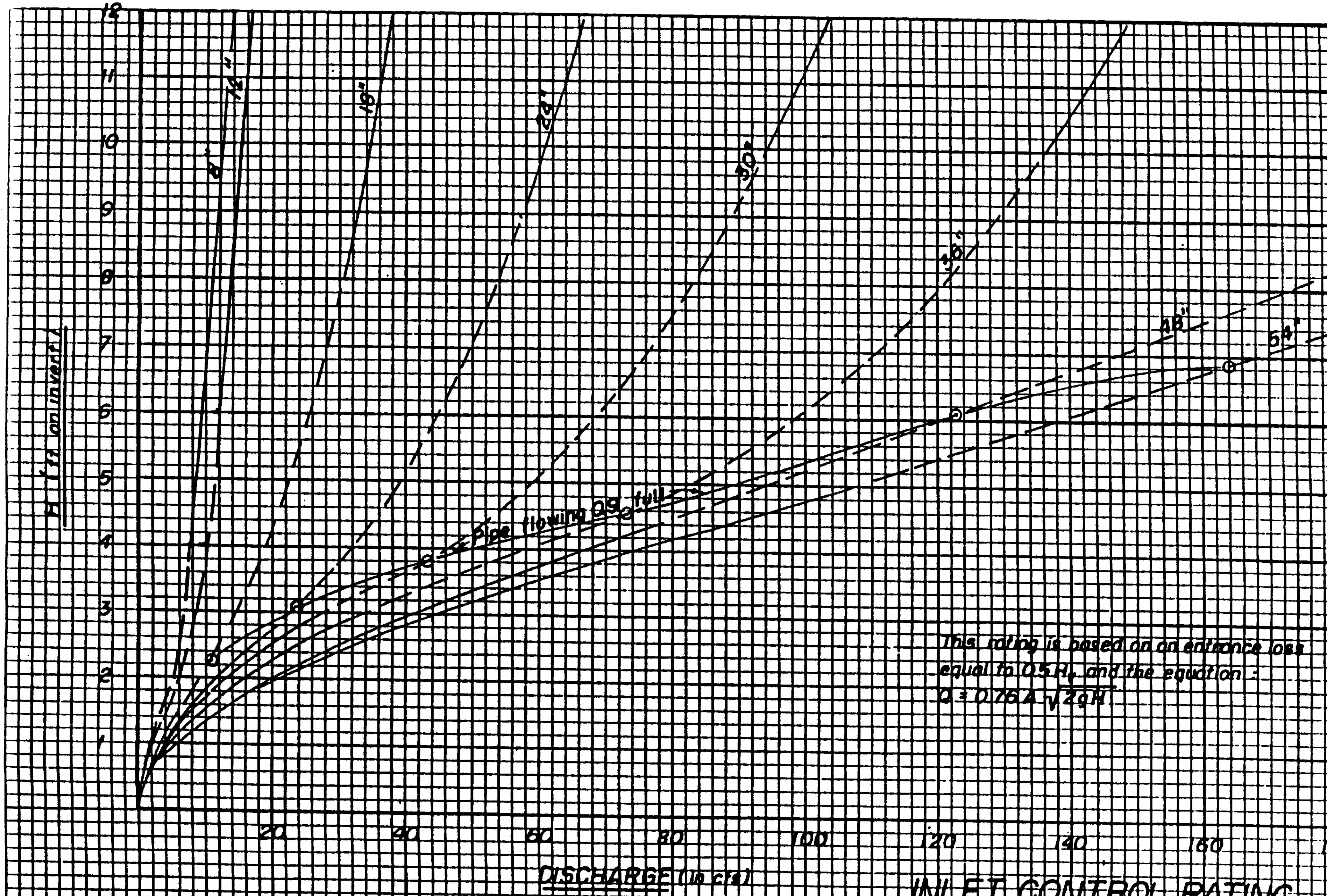
STAGE AND DISCHARGE HYDROGRAPH CURVES



STAGE HYDROGRAPH FOR RETENTION BASIN - 21 July 1973 Storm
EAST AREA INFLOW ONLY



INFLOW AND OUTFLOW DISCHARGE HYDROGRAPHS - 21 July 1973 Storm
EAST AREA INFLOW ONLY



INLET CONTROL RATING
circular conduits

Daugherty 1975

(G)

G/2

SIZE OF RIPRAP TO BE USED DOWNSTREAM FROM STILLING BASINS

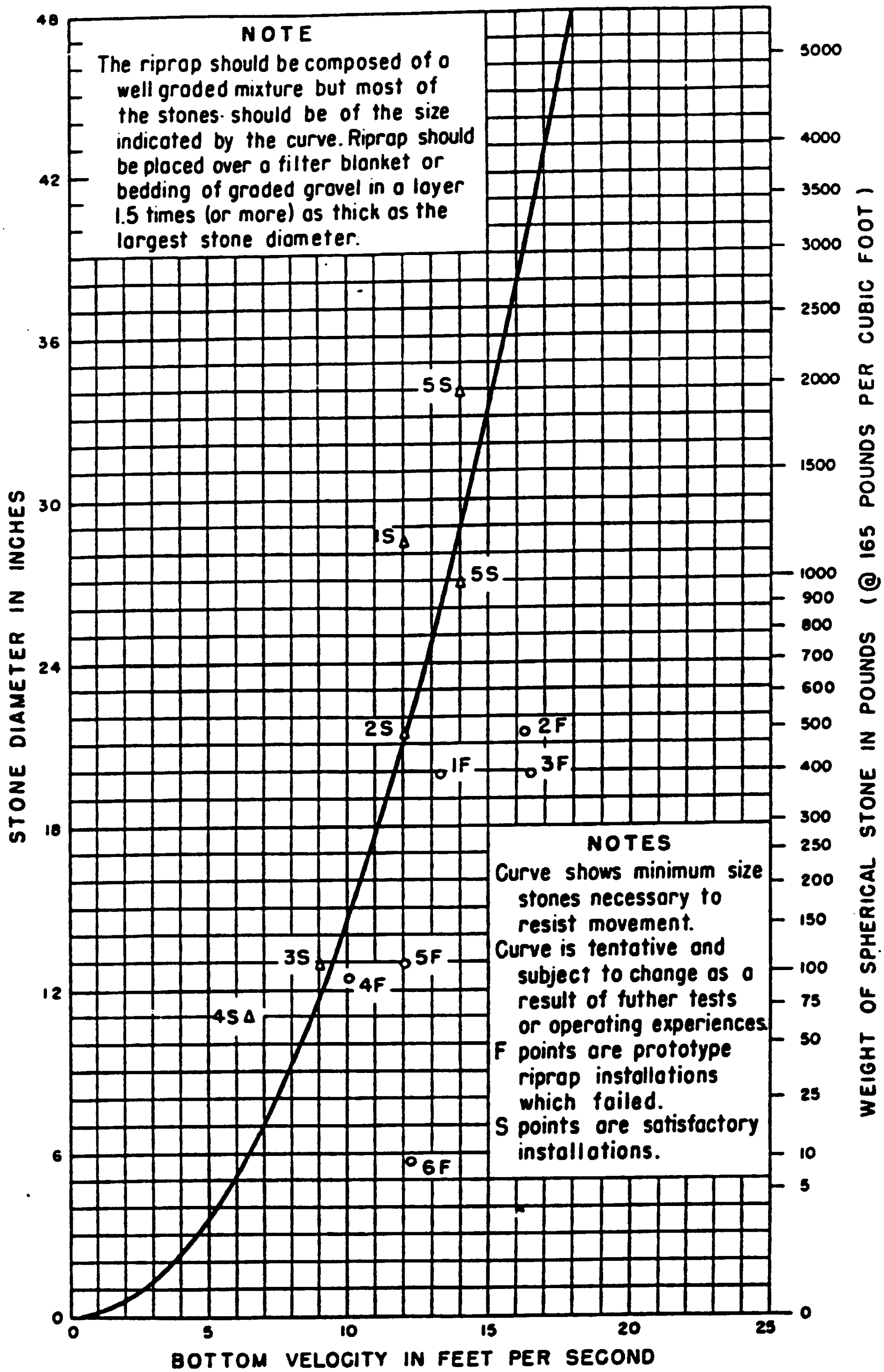


FIGURE 165.—*Curve to determine maximum stone size in riprap mixture.*

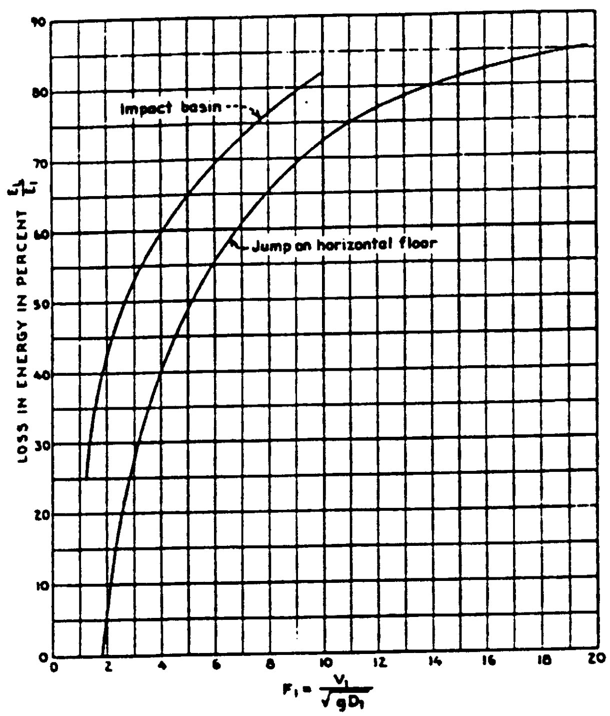


FIGURE 44.—*Comparison of energy losses—impact basin and hydraulic jump.*

Basin Design

Table 11 and the key drawing, Figure 42, may be used to obtain dimensions for the usual structure operating within usual ranges. However, a further understanding of the design limitations may help the designer to modify these dimensions when necessary for special operating conditions.

The basin dimensions, Columns 4 to 13, are a function of the maximum discharge to be expected, Column 3. Velocity at the stilling basin entrance need not be considered, except that it should not greatly exceed 30 feet per second.

Columns 1 and 2 give the pipe sizes which have been used in field installations. However, these may be changed as necessary. The suggested sizes were obtained by assuming the velocity of flow to be 12 feet per second. The pipes shown would then flow full at maximum discharge or they would flow half full at 24 feet per second. The basin operates as well whether a small pipe flowing full or a larger pipe flowing partially full is used. The pipe size may therefore be modified to fit existing conditions, but the relation between structure size and discharge should be maintained as given in the table. In fact, a pipe need not be used at all; an open channel having a width less than the basin width will perform equally as well.

The invert of the entrance pipe, or open channel, should be held at the elevation shown on the drawing of Figure 42, in line with the bottom of the baffle and the top of the end sill, regardless of the size of the pipe selected. The entrance pipe may be tilted downward somewhat without affecting performance adversely. A limit of 15° is a suggested maximum although the loss in efficiency at 20° may not cause excessive erosion. For greater slopes use a horizontal or sloping pipe (up to 15°) two or more diameters long just upstream from the stilling basin.

For submerged conditions a hydraulic jump may be expected to form in the downstream end of the pipe sealing the exit end. If the upper end of the pipe is also sealed by incoming flow, a vent may be necessary to prevent pressure fluctuation in the system. A vent to the atmosphere, say one-sixth the pipe diameter, should be installed upstream from the jump.

The notches shown in the baffle are provided to aid in cleaning out the basin after prolonged nonuse of the structure. When the basin has silted level full of sediment before the start of the spill, the notches provide concentrated jets of water to clean the basin. If cleaning action is not considered necessary the notches need not be constructed. However, the basin is designed to carry the full discharge, shown in Table 11, over the top of the baffle if for any reason the space beneath the baffle becomes clogged, Figure 45C. Although performance is obviously not as good, it is acceptable.

TABLE 11.—*Stilling basin dimensions (Basin VI). Impact-type energy dissipator.*

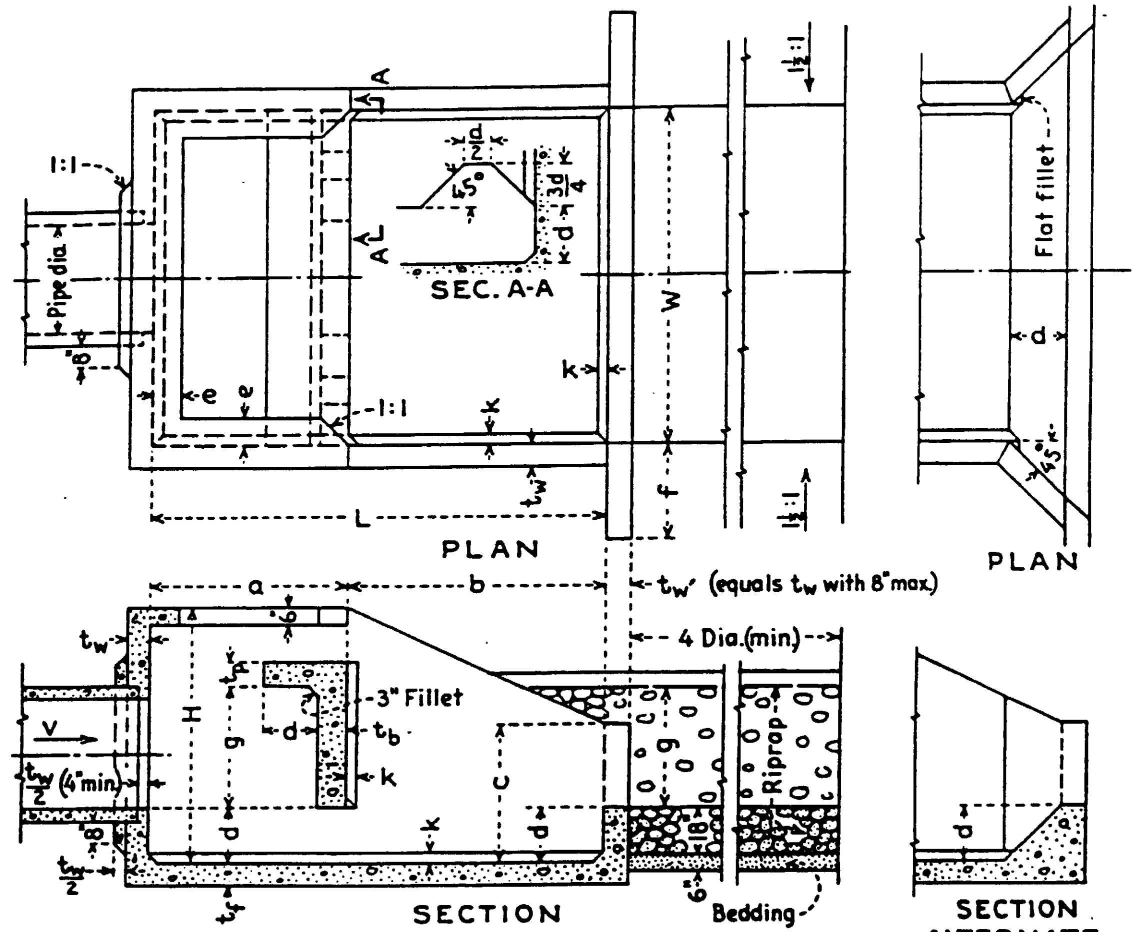
Suggested pipe size[1]		Max discharge Q	Feet and inches										Inches					
Dia in.	Area (sq ft)		W	H	L	a	b	c	d	e	f	g	t_w	t_f	t_b	t_p	K	Suggested riprap size
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)[3]
18	1.77	[2] 21	5-6	4-3	7-4	3-3	4-1	2-4	0-11	0-6	1-6	2-1	6	6½	6	6	3	4.0
24	3.14	38	6-9	5-3	9-0	3-11	5-1	2-10	1-2	0-6	2-0	2-6	6	6½	6	6	3	7.0
30	4.91	59	8-0	6-3	10-8	4-7	6-1	3-4	1-4	0-8	2-6	3-0	6	6½	7	7	3	8.5
36	7.07	85	9-3	7-3	12-4	5-3	7-1	3-10	1-7	0-8	3-0	3-6	7	7½	8	8	3	9.0
42	9.62	115	10-6	8-0	14-0	6-0	8-0	4-5	1-9	0-10	3-0	3-11	8	8½	9	8	4	9.5
48	12.57	151	11-9	9-0	15-8	6-9	8-11	4-11	2-0	0-10	3-0	4-5	9	9½	10	8	4	10.5
54	15.90	191	13-0	9-9	17-4	7-4	10-0	5-5	2-2	1-0	3-0	4-11	10	10½	10	8	4	12.0
60	19.63	236	14-3	10-9	19-0	8-0	11-0	5-11	2-5	1-0	3-0	5-4	11	11½	11	8	6	13.0
72	28.27	339	16-6	12-3	22-0	9-3	12-9	6-11	2-9	1-3	3-0	6-2	12	12½	12	8	6	14.0

[1] Suggested pipe will run full when velocity is 12 feet per second or half full when velocity is 24 feet per second. Size may be modified for other velocities by Q=AV, but relation between Q and basin dimensions shown must be maintained.

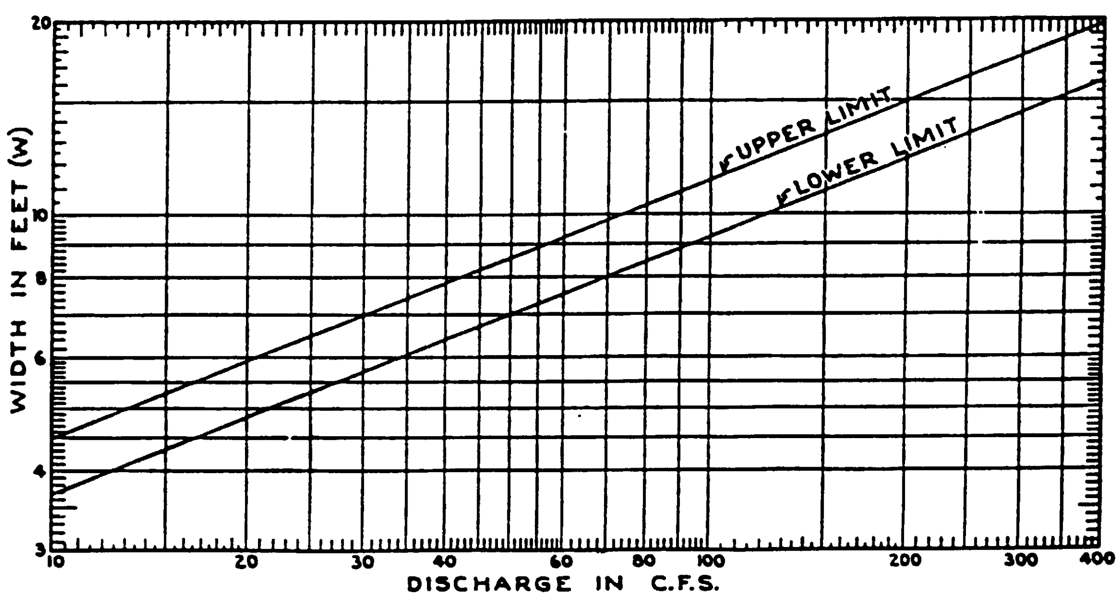
[2] For discharges less than 21 second-feet, obtain basin width from curve of Fig. 42. Other dimensions proportional to W; $H=\frac{3W}{4}$, $L=\frac{4W}{3}$, $d=\frac{W}{6}$, etc.

[3] Determination of riprap size explained in Sec. 10.

APPENDIX G



STILLING BASIN DESIGN



DISCHARGE LIMITS

FIGURE 42.—*Impact-type energy dissipator (Basin VI).*