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Cost-Effective Roadway Drainage Design Using Economic Analysis

Research, Development, and Technology
Turner-Fairbank Highway Research Center
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McLean, Virginia 22101-2296

FOREWORD

This report presents a practical method for designing the road surface drainage system using the theoretic Lowest Total Economic Cost (LTEC) approach. Key parameters considered in this study include rainfall, capital cost of the storm drainage systems, and traffic losses due to flooding of roads.

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16. Abstract
A practical method for determining the road surface drainage system design with the theoretic Lowest Total Economic Cost (LTEC) is developed. The LTEC design determines the design rain which, when used in a rational-based design context, will yield the most economic choices of gutters, inlets, and laterals considering both construction costs and risk costs. The data needed to find the LTEC design rain are: the number of rains per year, the design gutter flow, duration of average rainfall, runoff coefficient, drainage area, inlet and lateral costs, capital recovery factor, and a traffic loss coefficient. A nomograph and data selection guidance are provided as design aids. Three case studies are presented in an appendix. The method, based on minimizing traffic delay costs, applies to freeways, arterials, and major collectors; local streets with low traffic may be excluded.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

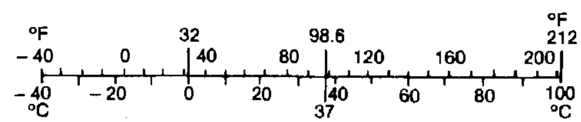
Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
	MASS			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
	MASS			
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
	TEMPERATURE (exact)			
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

(Revised April 1989)

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INTRODUCTION

The goal of this paper is to present a practical engineering risk-based method for design of pavement and surface drainage systems. The target systems include curbs, gutters, inlets, and storm sewers. Risks are defined as the expected or average annual losses associated with random, rain-related failures of the drainage systems. The sum of construction costs and costs associated with risks is the total cost of a facility. The mathematical objective is to find the design that has the Lowest Total Economic Cost (LTEC) design. This paper synthesizes hydrology, hydraulics, statistics, and traffic economics; it presents a systems analysis.

One of the most perplexing questions in flood control and drainage design is what return period, or failure probability, should be used. In the absence of a risk-based design, best engineering judgment is required. Since public agencies responsible for the safety of the populace are reluctant to rely on uniform application of judgement, fixed criteria are the rule. For example, the required return periods for Pennsylvania and Virginia highway drainage facility design are:

- Pennsylvania
  - a. Interstate highways - 50 years
  - b. Primary highways - 25 years
  - c. Secondary highways - 10 years

- Virginia
  - a. Interstate highways - 50 years
  - b. Primary and secondary highways with no depressed roadways - 10 years
  - c. Primary and secondary highways with depressed roadways - 50 years

In theory, with an LTEC design, the need for fixed return periods vanishes. The LTEC methodology supplies the needed design factors. In practice, public agencies are also wary of imposing a complicated design method and presumably fear nonuniformity of application by a wide range of novice to experienced designers. The results of this paper are geared to the simplest, yet still practical, approach to LTEC design for surface drainage systems.

The historical development of this approach begins with box culverts.[15,16] Young developed an LTEC methodology to size box culverts that considers the risk cost components of traffic losses, system damages, and off-site flooding. Figure 1 illustrates a typical relationship between culvert size and risk cost, construction cost, and total cost. The uncertainties addressed are solely associated with random flows. Mays addresses the same design problem, but considers additional uncertainties such as cost variance.[8] Tseng evaluates flood risk factors in the design of highway stream crossings.[13] Corry applies LTEC methodologic developments to the design of encroachments on flood plains using risk analysis.[3] Corry's work

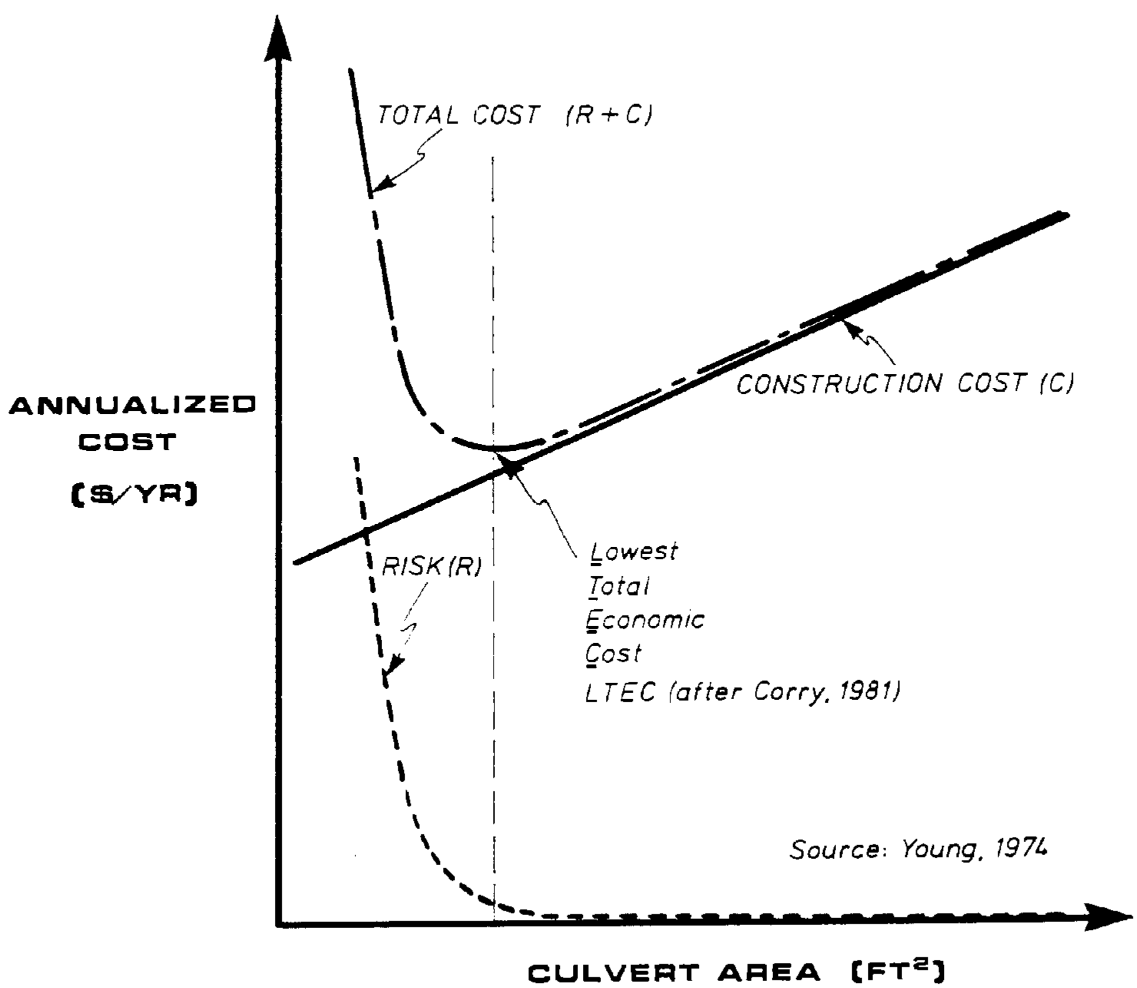


Figure 1. Culvert cost response curve.

coins the acronym "LTEC" to describe the approach. For his case, trial applications of LTEC finds replacement costs to be dominant and leads to the conclusion that LTEC is unnecessary as it always gives the same answer: build the bridge as large as feasibly possible.

Thus far, the development of LTEC involves bridges and culverts. This present work extends the approach to consider surface drainage of streets and highways using gutters, grate and curb inlets, and storm sewers.

Table 1 generalizes the dominant risks associated with these three types of drainage facilities: bridges, culverts, and inlets and sewers.

Table 1. Dominant risk components of three types of drainage facilities.

Facility Type	Dominant Risk Components		
	System damage	Off-site flooding	Traffic losses
Bridges	X		
Culverts	X	X	X
Inlets/sewers			X

Experience with risk assessment supports this generalization that may be qualified for specific cases. Consider the disfunction of the three facility types: If a bridge or its approach embankment fails, reconstruction costs will be extremely high; other damage costs exist, but are secondary to the expensive proposition of fixing the stream crossing as quickly as possible. While culverts effectively use upstream ponds to minimize construction costs, the ponds can cause property damage; also, a small culvert can develop sufficient backwater to flood and damage its embankment or to stop traffic. Inlets and surface drains, if flooded, stop or slow traffic, but seldom are physically damaged or cause adjacent property damage.

The relative costs of the various types of facilities on a risk component basis are ranked as bridges over culverts over individual surface drains. However, surface drainage systems are pervasive land development features that relieve flooding of collector streets in residential, industrial, and commercial zones, as well as main streets and highways. Therefore, the sum total cost of all the smaller, less expensive surface drainage elements becomes a major part of overall infrastructure cost.

A risk analysis requires that there be a probability distribution for a controlling variable. Stream flow, or sizable drainage basin hydrographs, usually provide the random variable for bridge and culvert analysis, which complicates assignment of probabilities and hydraulic routing computations. For surface drains, however, rainfall, with a triangular surface hyetograph is used for the random variable.[14] This fact simplifies risk computations as this analysis will demonstrate.

This paper presents:

1. A discussion of simulation experiments that exploit usage of a personal computer to gain insight into the performance of surface drainage inlet and storm sewer systems.

2. Analysis of rainfall and its probability distribution for incorporation into the risk-based design.

3. Mathematical model formulations to calculate costs and risks. The model leads to an estimate of design rain based on site features such as area, imperviousness, local costs of installed inlets and storm sewers, traffic loss characteristics, gutter geometry, and local rainfall statistics. (The site can be any road-related drainage system from small subdivision to major thoroughfare.) The designer uses this design rain in determining the gutter inlet and storm sewer components of the LTEC design.

4. Approaches to estimate traffic-related drainage losses to support the LTEC determination.

5. An applications guide giving required data and a nomograph to select the design rainfall for use in performing LTEC design.

6. Conclusions that summarize the approach and present areas for future refinements.

7. Four appendixes: references, three case studies, a list of symbols used, and a unit conversion table.

SIMULATION EXPERIMENTS

The purpose of these simulation analyses is to develop an insight into road conditions during heavy rainstorms for use in design and analysis of roadway drainage networks. The methods described in Drainage of Highway Pavements - Hydrologic Engineering Circular 12 and the sewer software package PFP-HYDRA are used to set up the network for the simulations.[6,4] HEC-12 methodology is applied in determining inlet spacing, while PFP-HYDRA is used to generate inlet and system hydrographs, with modifications made to consider ponding at the roadway low point. A hypothetical four-lane urban road is used as a test example.

Consider the representative test drainage system in figure 2. The user must supply PFP-HYDRA with inlet spacing, drainage areas, and interceptor

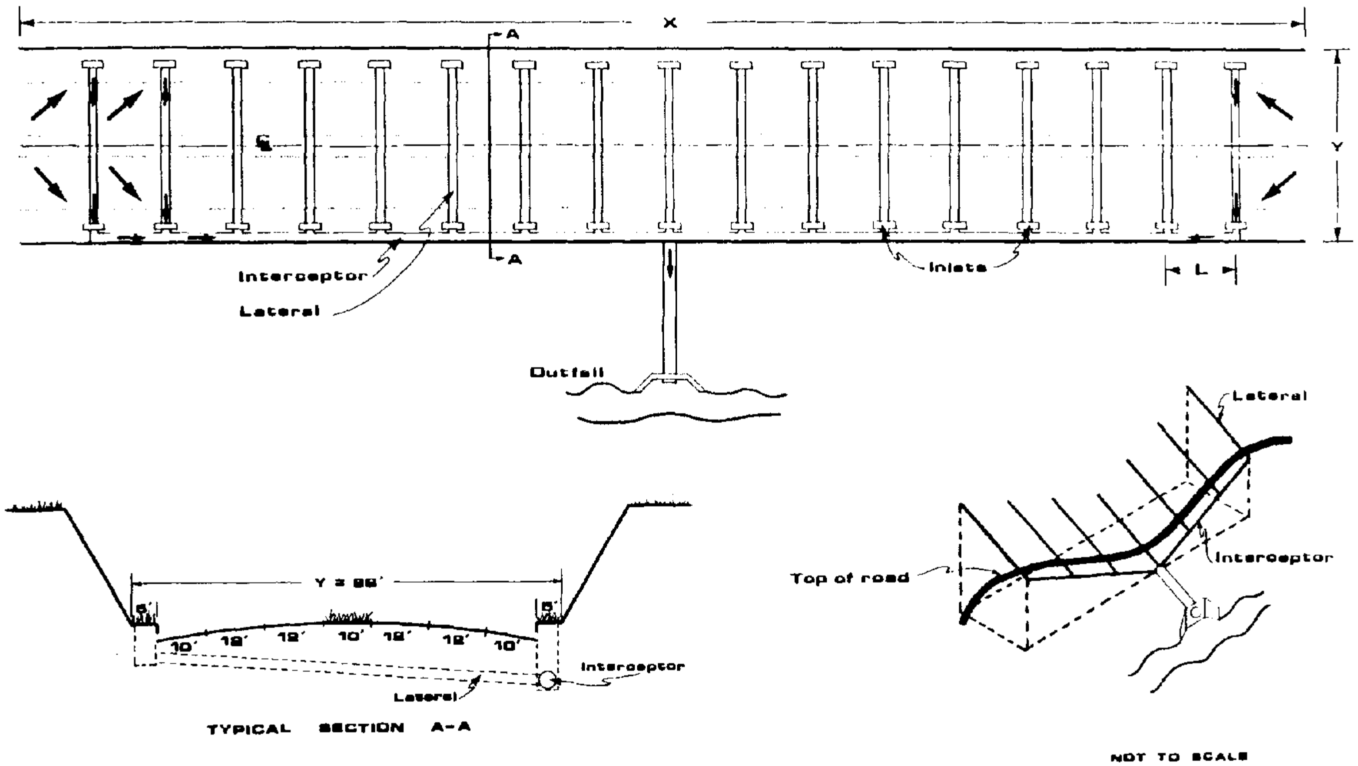


Figure 2. Representative payment drainage system.

segment lengths; therefore, HEC-12 methods are first applied to determine inlet spacing, **L**. The following information must be known or assumed:

- Latitude/longitude of site - assume 40°45′ N
  111°53′ W
  (Salt Lake City, Utah)
- Design storm return period - assume 2 years
- Gutter Manning's n - let n = 0.016
- Gutter cross slope, S_x - let S_x = 0.02 ft/ft
- Gutter longitudinal slope, S - let S = 0.015 ft/ft
- Shoulder width (allowable spread), T - let T = 10 ft
- Runoff coefficient, C - let C = 0.854
- Time of concentration, t_c - let t_c = 5 min (0.0833 hr)

From HEC-12, gutter capacity, q_T, is computed using the equation:

$$q_T = \frac{0.56}{n} \cdot S_x^{1.67} \cdot S^{0.5} \cdot T^{2.67} \quad \text{(1)}$$

For the example above,

$$q_T = \frac{0.56}{0.016} \cdot (0.02)^{1.67} \cdot (0.015)^{0.5} \cdot (10)^{2.67},$$

q_T = 2.92 cfs.

The flow accumulated at a specific point in a gutter can also be written as a function of inlet drainage acreage, a, (and therefore also as a function of inlet spacing) by applying the rational formula.(7)

$$q_T = C \cdot i \cdot a \quad \text{(2)}$$

where i = rainfall intensity (in/hr)

$$q_T = C \cdot i \cdot (L \cdot w / 43560) \quad \text{(3)}$$

where L = inlet spacing (ft)
w = width of the pavement contributing runoff to one inlet, or 44 feet in figure 2. (ft)
43560 = factor to convert ft^2 to acres.

For the example above,

q_T = 2.92 cfs = 0.854 · **i** · (L·44/43560)
or
L = 4950/i ....(4)

If **i** is unknown for the desired frequency and duration, the software package HYDRO, that implements HEC-19, can be used to generate an Intensity-Duration-Frequency (IDF) curve at any given latitude/longitude in the continental United States.[4,7] Assuming duration equals time of concentration, a 2-year, 5-minute storm corresponds to an intensity of 2.876 in/hr for Salt Lake City. Thus, for this example,

L = 4950/2.876
= 1721.1 feet

Inlet separation, **L**, is then incorporated into a PFP-HYDRA input file.

The computer program PFP-HYDRA analyzes an existing collection system for a particular site or designs a new system based on user-defined restrictions. To perform hydrologic simulation on an existing or proposed system, the user creates an input file which includes site information: drainage area, rainfall intensity or hyetograph definition, and street gutter, inlet, and pipe data.

The PFP-HYDRA input file consists of a series of commands that describe the drainage network. Each command is made up of a three-letter "key word" specifying the function to be performed followed by the appropriate data required by HYDRA to perform the function. For example, the pipe command **PIP 1720 988.3 962.5**, tells PFP-HYDRA to transport flow through a circular pipe 1720 feet long with upstream ground elevation of 988.3 feet and downstream ground elevation of 962.5 feet.

PFP-HYDRA models the situation in which gutter flow occurs between inlets during periods of rainfall; the program also routes flow past overloaded inlets. The hydrographs generated by PFP-HYDRA are then used to determine the width of water spread at the inlets. Spread can be computed from the PFP-HYDRA flow data using HEC-12 methods, while an ad hoc FORTRAN program calculates ponding depth at the low point or sump.

A two-step simulation analysis is thus conducted:

1. A rainfall intensity is assumed and a gutter spacing is determined for the representative pavement drainage system. This involves calculation of q_T and determination of **L** using equations (1) and (3).

2. A triangular hyetograph is simulated with HYDRO using a maximum intensity equal to the intensity assumed in item one. The

triangular hyetograph uses the Yen and Chow formulation and yields a total rainfall of:[14]

$$X = i \cdot \bar{t} / 2 \qquad (5)$$

where $\bar{t}$ = duration of storm (hours).

The triangular hyetograph and its salient features are shown in figure 3. Gutter response to higher and lower intensity storms is also simulated.

The simulations portray flooding conditions and spread in the gutter and the sump for a variety of inlet spacings. This spread plotted against time will be termed a "spreadograph." An example input rain hyetograph and output spreadograph is shown in figure 4 for an inlet spacing of 703 feet and a total rainfall of 4 inches.

The experiments can be reproduced using the PFP-HYDRA computer program. The computations lead to the following observations which appear to be general for street surface drainage in gutters and pipes with free outfalls:

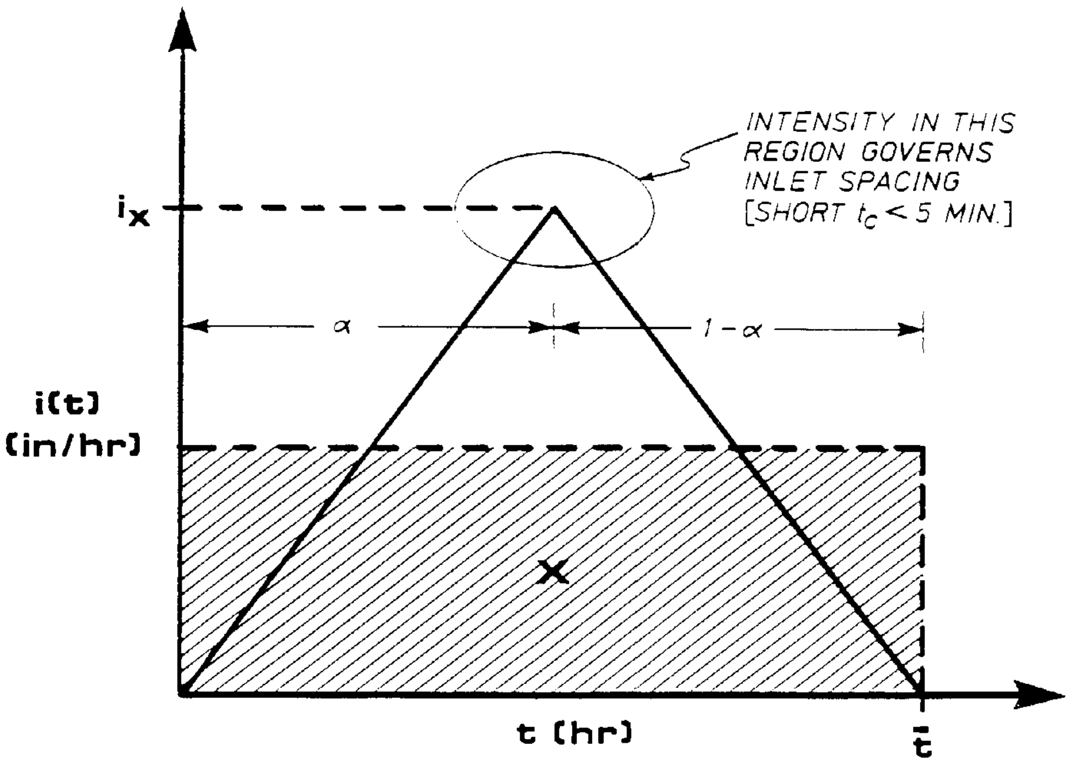
1. The inlet flow is the controlling hydraulic parameter for the "upper" regions of the network. The laterals are only flowing part full: these pipes are sized at 15 to 18 inches for nonhydraulic considerations of durability and clogging prevention and the inlets do not generate sufficient input flow to fill the laterals.

2. At the sump, the inlet will act more as a weir than as an orifice for almost all situations. The water does not get deep enough for reasonably sized inlets to flood out the weir flow regime and generate orifice flow. This may not be the case for very small inlets or bridge scuppers.

3. The spread is set by the apex value of the Yen and Chow hyetograph.[14] The time of concentration to each inlet is very short (less than 5 minutes), and the intensity derived from an IDF curve is usually at the left hand maximum limit. This value corresponds to the apex of the triangular hyetograph because the apex is the maximum instantaneous value.

4. The street and its gutters are a surface drainage system in, and of, themselves. They convey bypassed inlet flows to the low point or sump at or near the outfall of the storm sewer system. When the inlets are overloaded, the excess flow rapidly moves to the sump and the sump inlets are quickly inundated since they handle their own design flows plus the excess of all the other inlets.

5. Surcharge of the underground system rapidly pushes water out of free outfall pipes. The speed of this release of water exceeds the speed with which water can enter the pipes through the inlets. Therefore, the inlet flow is also the controlling hydraulic



$$X = AREA = TOTAL\ RAIN\ [INCHES/EVENT] = \frac{i_x\,\bar{t}}{2}$$

- $i_x = \dfrac{2X}{\bar{t}}$
- α tabulated in Yen & Chow, 1983

Figure 3. Triangular hyetograph.[14]

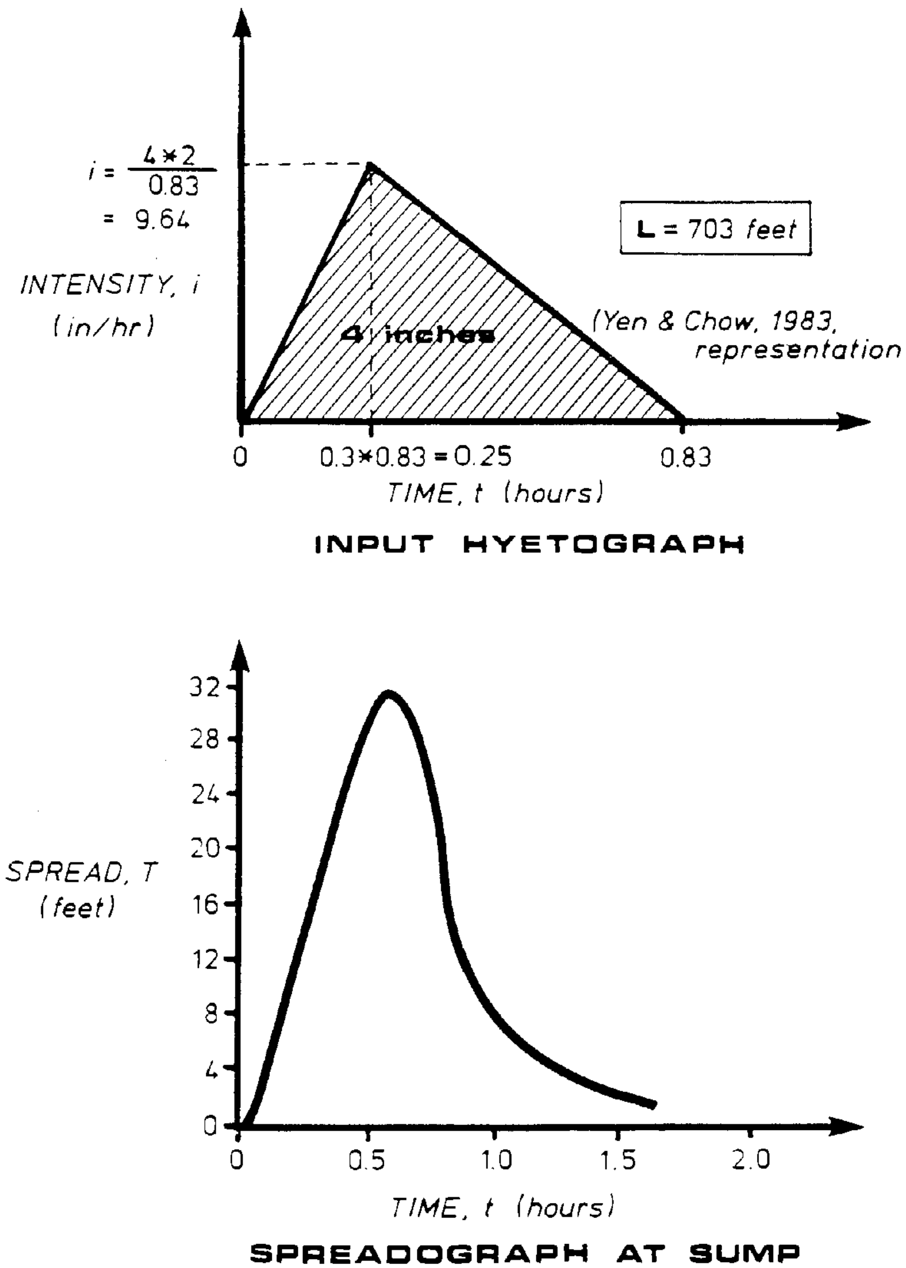


Figure 4. Input/output example.

parameter in the "lower" regions of the network. This may not be the case for outfalls that are submerged by larger elements of the downstream surface drainage system.

The following points summarize the findings of this section supporting the overall risk analysis:

1. The apex of the triangular hyetograph sets the design inlet spacing.

2. The subsurface laterals are oversized hydraulically and are not a factor in surface flooding.

3. The subsurface interceptor and outfall pipes can transport water under pressure faster than the inlets can generate inflow to create pressure. Thus, a subsurface pipe system designed for a 10-year storm is sufficient for much larger storms because the inlets will automatically pond thereby reducing pressure.

4. Beyond the time on the hyetograph when individual inlets are overloaded, the major flooding problem is quickly transmitted to the sump.

5. For the purpose of risk-based design, the total rainfall becomes a major state variable in the risk analysis. This is because the total rainfall can be linked to the principal parameter associated with pavement flooding, the inlet spacing, by means of the apex of a triangular hyetograph. Furthermore, risk analysis requires a probability distribution for a controlling variable, and in this case, the daily rainfall probability distribution serves as the required function.

RAINFALL DISTRIBUTION

A rainfall distribution, $f(x)$, is needed to make a risk assessment. This section discusses the justification for selecting total rain as the appropriate state variable. It is assumed, for practical purposes, that there is a single rainfall duration per day.

A plot of cumulative probabilities on normal probability paper of 1983 daily rainfall data for Nashville, Tennessee and Annapolis, Maryland is shown in figure 5. Similar plots for 20 other cities east of the Mississippi River show similar results: the right hand tail plots as a straight, nearly 45 degree line on normal probability paper. Since only the larger rains cause flooding problems, the right hand tail is an appropriate segment of the rain distribution to represent as a normal curve.

This section analyzes:

1. The slope or standard deviation that is appropriate for the normal curve assumption.

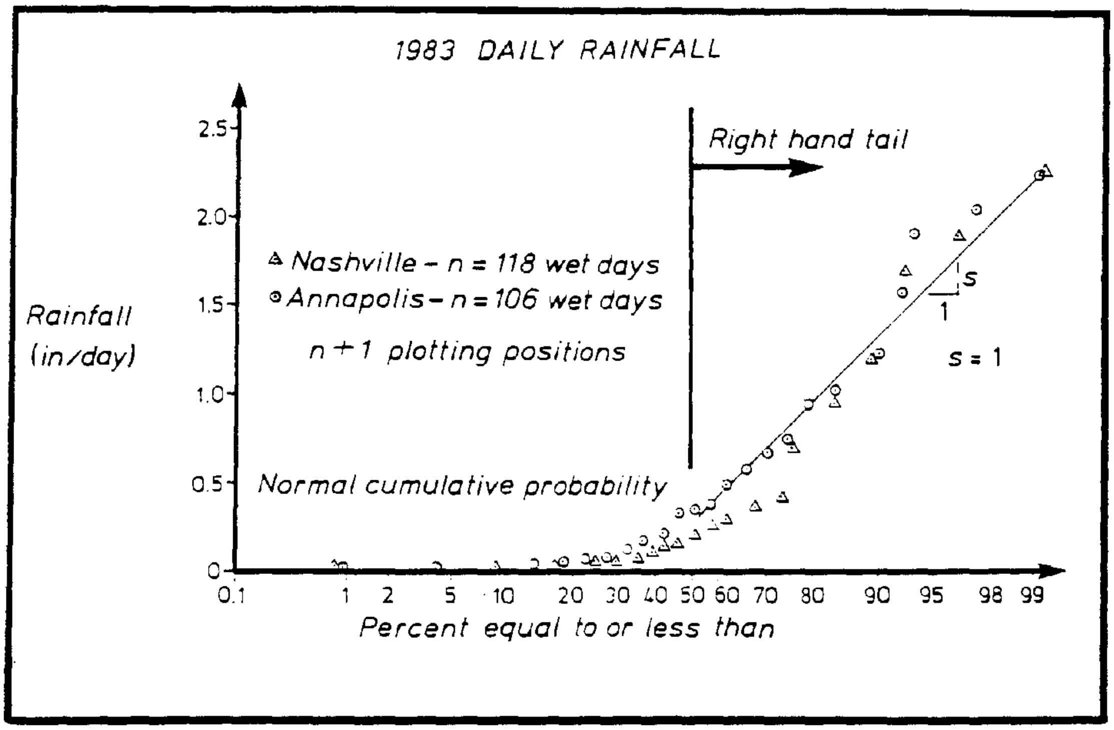


Figure 5. Fainfall data.

2. The duration of a rainstorm, since the average rainstorm duration affects expected duration of flooding and possible roadway traffic disfunction.

Precipitation data, recorded at 60-minute intervals, for cities east of the Mississippi for the year 1983 were acquired from the National Weather Service (NWS). The data were analyzed to determine the standard deviation of all observed storms exceeding the median, as listed in table 2. The average median rainfall is 0.2 inches. The standard deviation is computed as the root mean square of the observations exceeding the median.

Table 2. Standard deviation of observed storms for 20 cities.

Site Location	Total Rain Days	Standard Deviation
Albany, NY	131	0.77
Annapolis, MD	106	1.07
Atlanta, GA	112	1.09
Augusta, ME	130	0.96
Boston, MA	133	0.89
Charleston, SC	108	0.59
Columbia, SC	98	1.02
Columbus, GA	133	0.71
Concord, MA	131	0.80
Frankfort, KY	99	0.78
Harrisburg, PA	127	0.86
Hartford, CT	129	0.92
Montgomery, AL	89	1.41
Montpelier, VT	133	0.67
Nashville, TN	118	1.01
Providence, RI	129	1.21
Raleigh, NC	118	0.90
Richmond, VA	116	1.12
Tallahassee, FL	126	1.18
Atlantic City, NJ	113	0.82
Average $\pm$ Sd. Dev.	119 ± 13	0.94 ± 0.20

Thus, the expected square root of the second moment of the observations greater than the median (the upper 50%) is 0.94 ± 0.20. Because unity falls well within this range, and unity is associated with textbook normalized tabulations, the value of 1 inch is used to fit the right hand tail of the rainfall distribution to a N(0,1) approximation; i.e. the right hand tail of daily rainfall is fit with a mean of 0 and a standard deviation of 1.

Furthermore, the average number of daily rainstorms is 119 with a standard deviation of 13. Only one half of these storms have rainfall greater than 0.2 inches, and it is these storms that are used in the risk analysis. Thus, on the average, about 60 rainstorms a year generate sufficient downpour to cause potential roadway flooding problems in the study case cities east of the Mississippi.

Precipitation data, recorded at 15-minute intervals, acquired from NWS were analyzed for a large spatial network of nine stations near Washington, D.C., spanning a region from the Chesapeake Bay to the mountains of West Virginia. The average duration was determined for those storms with precipitation exceeding 0.2 inches. The results are listed in table 3.

Table 3. Average duration of storms with precipitation greater than 0.2 inches.

Gauge Location	Average Duration (hours)
Beltsville	0.62
Ft. McHenry	0.58
Patuxent River	0.64
Elkwood	0.60
Fredericksburg	0.62
Mt. Weatner	0.62
Piedmont	0.64
Remington	0.62
The Plains	0.62
Average ± Sd. Dev.	0.62 ± 0.02

The sample sizes at individual gauges ranged from 820 to 4138. Thus, for a wide range of stations, in the Virginia/Maryland/West Virginia region, the expected duration of a rainfall in excess of 0.2 inches is 0.62 hours or 37 minutes, with a standard deviation of 0.02 hours or 1.2 minutes.

The following points summarize the findings of this section supporting the overall risk analysis:

1. The right-hand tail of the rainfall distribution can be modeled with a high degree of confidence as a normal curve with mean of 0 and standard deviation of 1. Such a curve is idealized in figure 6; the integrated or cumulated version of this curve plots as the data shown in figure 5.

2. The expected duration of storms greater than 0.2 inches is 0.62 hours or 37 minutes, with negligible variation, for a large mid-Atlantic region.

3. The number of storms per year, in the study case cities east of the Mississippi, varies between 89 and 133 with an average of 119. Half this number have rainfall in excess of 0.2 inches and these 60 storms are assumed to govern risk determinations.

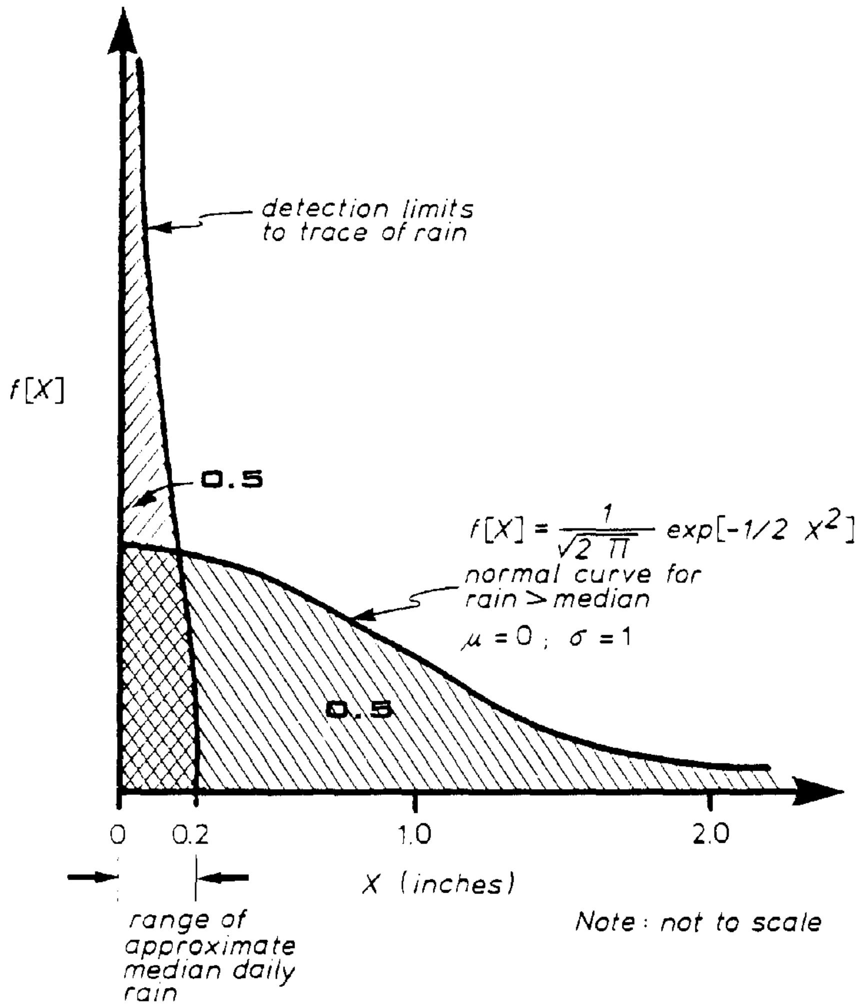


Figure 6. Daily rainfall distribution.

MATHEMATICAL MODEL FORMULATION

This section will define the variables and assumptions and formulate the expression that calculates the inches of LTEC design rainfall for a surface drainage system.

Dependent Variable.- The object of this analysis.

X_d - the design rain which, when utilized in design, will yield the LTEC design. (inches)

Independent Variables.- These variables are used in the derived decision model and nomograph.

A - The area of the surface drainage system. (acres)

C - The runoff coefficient as used in the rational method. (dimensionless)

C_1 - The cost of an inlet and its structure. ($/inlet)

C_2 - The cost of a lateral (the pipe connecting each inlet to an interceptor). ($/lateral)

K - The slope of the loss function. The assumed loss function is shown in figure 7. If the system experiences a rain , **X**, less than $\mathbf{X_d}$, there will be no loss. If the system experiences $\mathbf{X} > \mathbf{X_d}$, the loss increases linearly with slope equal to **K**. The estimation of **K** is discussed in a separate section of this paper. ($/inch)

N - The number of events per year with greater than 0.2 inches of rain. (#)

q_T - The gutter flow, for spread **T**, calculated using equation (1). This flow is assumed to be the flow received by one inlet. (cfs)

r - The ratio of laterals to inlets. Figure 8 shows a schematic of typical drainage networks with r = 1/2 for highway networks (oblong drainage areas) and **r** = 1 for business and residential networks (circular drainage areas).

$\bar{t}$ - The average duration of rainstorms in the vicinity of the drainage system. (hours)

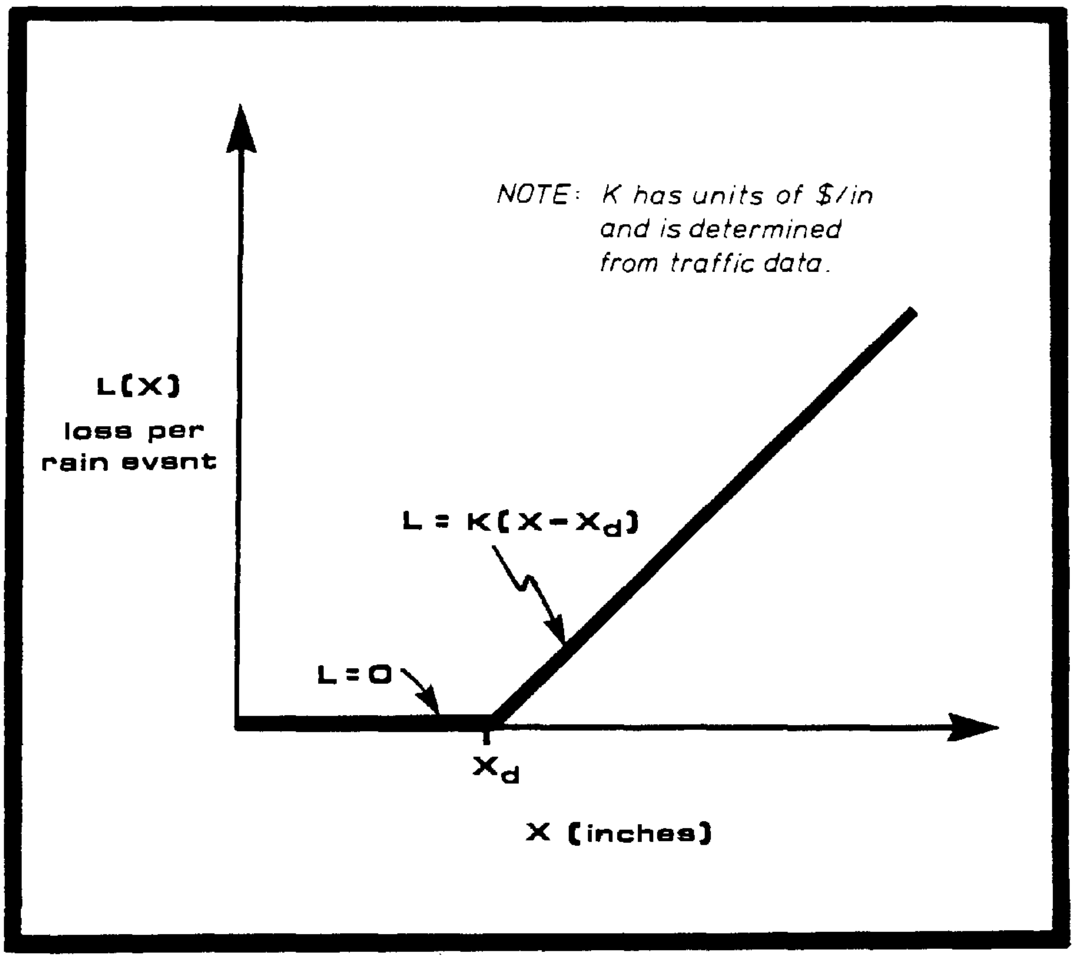


Figure 7. Loss function.

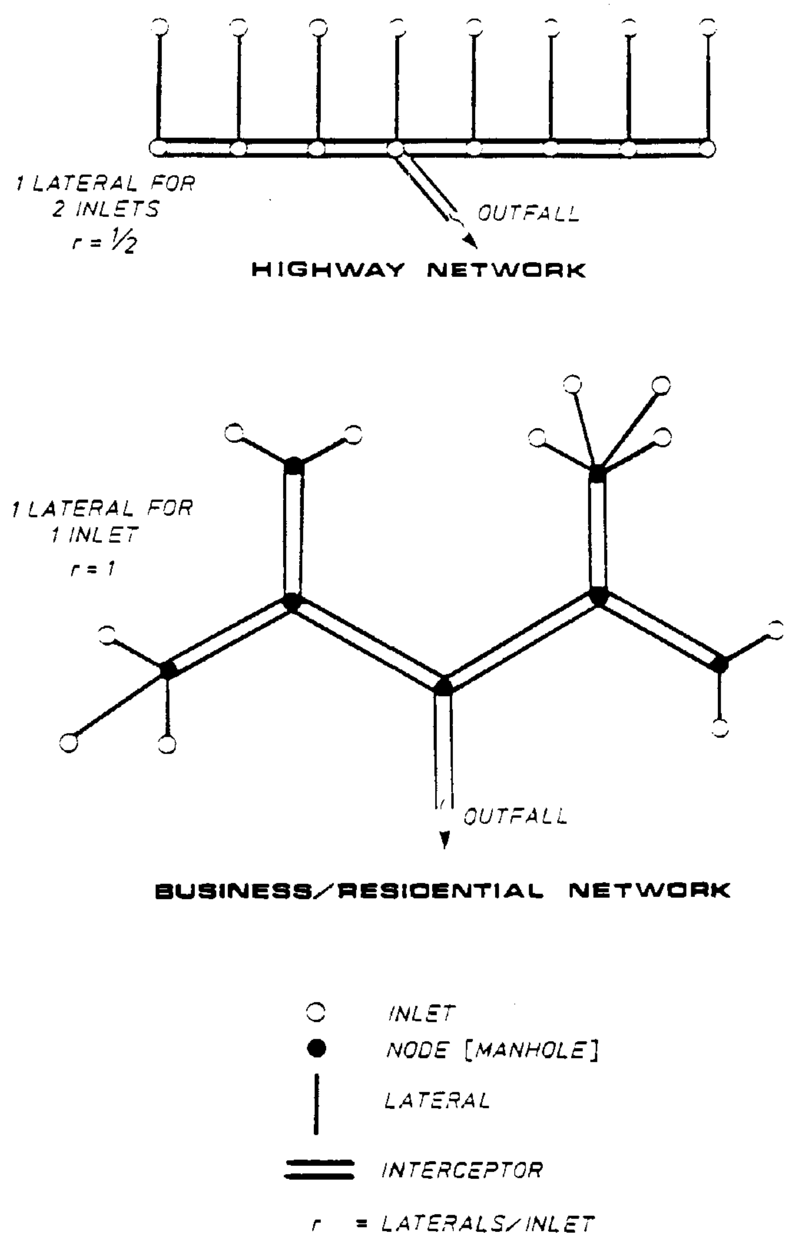


Figure 8. Typical drainage networks.

Intermediate Variables.- These variables are used as intermediate quantities in the derivation of model or are needed to calculate an independent variable, as in the case of gutter geometry.

C_o - the total cost of the subsurface pipe system consisting of interceptors and outfalls which is relatively fixed and independent of the design rain when a free outfall condition exists. ($/system)

f(x) - the probability function itself, shown in figure 6.

i - the rainfall intensity. (in/hr)

L(x) - the loss function itself, shown in figure 7. ($)

m - the number of inlets within A, each of which is assumed to handle a flow of q_T. (#)

n - the Manning's friction factor.

Q - the hydraulic capacity of the entire drainage system. This variable represents the sum of all the inlet flows at the inlet design capacity. (cfs)

S - the typical or average street grade. (ft/ft)

S_x - the typical or average gutter cross slope that is representative of all the gutters in the surface drainage system. (ft/ft)

T - the allowable gutter spread. This variable is established by the designer prior to analysis. It may also be set by administrative fiat. It is assumed that spread in excess of **T** will be disruptive to traffic function. The allowable spread could be the width of the shoulder or it could be the width of the shoulder plus the distance from the edge of traveled pavement to the outside tire track locus. (ft)

X - the total rain associated with a rainfall event, which is the area under the hyetograph. (inches)

The Cost Model. - The cost model is related to the number of inlets. The costs which are a function of the number of inlets are inlet costs and lateral costs. Laterals are related to inlets based on their ratio, **r**, which depends on the type of network. Typical networks are shown in figure 8.

The costs are formulated as:

$$C(m) = C_o + (C_1 + r \cdot C_2) \cdot m \qquad (6)$$

In order to adapt this formulation to the decision variable, X_d, the number of inlets has to be computed as a function of Q/q_T. For the Yen and Chow hyetograph, the maximum rainfall intensity as a function of total design rain is:

$$i(X_d) = 2 \cdot X_d / \bar{t} \quad \text{(7)}$$

Using the rational method to determine the flow that the inlets must handle yields:

$$\mathbf{Q(X_d)} = C \cdot (2 \cdot X_d / \bar{t}) \cdot A \quad \text{(8)}$$

Dividing $\mathbf{Q(X_d)}$ by q_T gives the required number of inlets:

$$m = C \cdot (2 \cdot X_d / \bar{t}) \cdot A / q_T \ ,$$

or, upon reduction,

$$m(X_d) = \frac{2 \cdot C \cdot A}{q_T \ \bar{t}} \cdot X_d \quad \text{(9)}$$

Substitution of $m(\mathbf{X_d})$ into the expression for **C(m)**, equation (6), gives the capital cost model as a function of the decision variable, $\mathbf{X_d}$:

$$C(X_d) = C_0 + (C_1 + r \cdot C_2) \frac{2 \cdot C \cdot A}{q_T \ \bar{t}} \cdot X_d \quad \text{(10)}$$

The Risk Model.- The annual cost risk model is the weighted probability of rain and loss times the expected number of rains per year:

$$R(X_d) = N \cdot \int_{X_d}^{\infty} f(x) \cdot L(x) \, dx \quad \text{(11)}$$

Substituting the Gaussian error function and the loss function yields:

$$R(X_d) = N \cdot K \cdot \int_{X_d}^{\infty} \frac{(X - X_d) \cdot \exp(-0.5 \cdot X^2)}{\sqrt{2\pi}} \, dx \quad \text{(12)}$$

Equation (12) is numerically integrated and fit to an exponential approximation function. The following function was determined when the fit was performed over the range of 0.2 to 2.0:

$$R(X_d) = N \cdot K \cdot [\ 0.542 \cdot \exp(-1.9887 \cdot X_d)] \quad \text{(13)}$$

The value in brackets approximates the exact integral over the practical range of $\mathbf{X_d}$ as shown in table 4.

Table 4. **Numerically integrated vs. approximated evaluations of the risk model integral.**

X_d (inches)	Exact Integral	Exponential Approximation ($R^2 = 0.96$ for $0.2 \le X_d \le 2.0$)
0.0	0.400	---
0.2	0.307	0.364
0.4	0.230	0.245
0.6	0.169	0.164
0.8	0.120	0.110
1.0	0.083	0.074
1.2	0.056	0.050
1.4	0.037	0.033
1.6	0.023	0.022
1.8	0.014	0.015
2.0	0.008	0.010
2.2	0.005	0.007
2.4	0.003	0.005
2.6	0.001	0.003
2.8	0.001	0.002
3.0	0.000	0.001

Equation (13) is an approximation of the risk integral. However, its region of approximation of 0.2 to 2.0 is adequate to the analysis considering the other sources of parameter error; for $X_d > 2.0$, the exact integral has higher order contact with the abscissa than does the exponential approximation.

The Total Cost Model.- The total cost model is the sum of the annual risk costs plus the annualized construction costs. To analyze construction costs, a capital recovery factor (crf), also known as "(A|P)" in published compound interest tables is used which is a function of interest rate and time horizon.[12] For example, an interest rate of 8 percent and time horizon or economic life of 20 years gives a **crf** of 0.1 . The total cost model using the approximate exponential risk function is:

$$TC(X_d) = N \cdot K \cdot [\ 0.542 \cdot \exp(-1.9887 \cdot X_d)\ ] + \left[\ C_0 + (C_1 + r \cdot C_2)\ \frac{2 \cdot C \cdot A}{q_T\ \bar{t}} \cdot X_d\ \right] \cdot crf \qquad (14)$$

The Decision Model.- The lowest total economic cost occurs at the minimum of **$TC(X_d)$** where $dTC(X_d)/dX_d = 0$. Differentiating equation (14) and setting the expression equal to zero yields the LTEC design as a function of design rainfall:

$$X_d = -0.503 \cdot \ln \left[1.856 \cdot (1/N) \cdot 1/(q_T \cdot \bar{t}) \cdot (C \cdot A) \frac{(C_1 + r \cdot C_2) \cdot crf}{K}\right] \quad .....(15)$$

This equation is the decision model. The optimum value obtained for X_d is then used in the LTEC risk-based design. The optimum design intensity is calculated by equation (7) as $i(X_d) = 2 \cdot X_d / \bar{t}$ and is then used to generate optimum inlet spacing and locations using HEC-12 methods.

ESTIMATION OF LOSS FUNCTIONS

This section develops the method used to estimate the loss function slope, **K**. The approach is to assume that a system is adequate for a design rain of one inch and then to analyze the hydraulic and traffic effects of an additional inch of rain.

Finding the Spreadograph.- The question is: how much spread does an extra inch of rain cause? The previous sections provide the following relationships which support determination of spread versus time for various hyetographs:

1. The spread at an inlet is governed by the apex value of the triangular hyetograph.

2. Average rainfall duration in the mid-Atlantic region is 0.6 hours. Thus, a one inch rain generates a maximum instantaneous intensity of $1 \cdot 2 / 0.6 = 3.33$ inches/hour. For a 2-inch rain, the maximum instantaneous intensity would be 6.66 inches/hour.

3. Gutter flow is proportional to spread raised to the 2.67 power, as given in equation (1). Since gutter flow is also proportional to rainfall intensity, the rainfall intensity is thus proportional to spread to the 2.67 power.

4. When a gutter inlet exceeds its design capacity, the bypass flows rapidly accumulate at the sump.

From these relationships, the typical spreadograph associated with a 1-inch excess over a 1-inch storm (i.e. a 2-inch storm) can be developed if the following assumptions are made:

1. The typical surface drainage system, with an 18 to 24 inch outfall, has on the order of 20 inlets.

2. One half (i.e. 10) of the inlets drain to each side of the sump.

3. The system is designed to have 10 feet of allowable spread for a one inch storm having an intensity of 3.33 inches/hour. Traffic lanes in the system are 12 feet wide.

The analysis is performed using the following proportionality:

$$\frac{(\text{2-inch storm spread at sump})^{2.67}}{(\text{1-inch storm spread})^{2.67}} = \frac{(\text{2-inch storm intensity at sump})}{(\text{1-inch storm intensity})} \quad \ldots(18)$$

At the 1-inch design condition for a single inlet, the intensity of 3.33 inches/hour is proportional to the allowable spread of 10 feet raised to the 2.67 power. Beyond the design condition, at a 2-inch storm, the intensity at the sump is equal to the 2-inch maximum intensity of 6.66 inches/hour plus the excess intensities resulting from the gutter flows bypassed from the 10 inlets on one side of the sump. Thus, the above proportionality becomes:

$$\frac{T(\text{sump})^{2.67}}{10^{2.67}} = \frac{6.66 + (6.66 - 3.33)\cdot 10}{3.33} \quad \ldots\ldots(19)$$

or

$$T(\text{sump}) = 25.4 \text{ ft}$$

Thus, at 3.33 inches/hour the spread at the individual inlets will be 10 feet; at 6.66 inches/hour the spread at individual inlets will increase slightly, but the spread at the sump will increase to approximately 25 feet.

For the average storm duration of 0.6 hours with a peak at 1/3 of its duration, 0.2 hours, the approximate spreadographs, tabulated in table 5 and depicted in figure 9, will ensue for the one and 2-inch storms.

Table 5. Approximate spread for 1- and 2-inch storms.

Time (hours)	1-Inch Storm Spread (feet)	2-Inch Storm Spread (feet)
0	0	0
0.1	8	10
0.2	10	25
0.4	8	10
0.6	0	0

These spreadographs, idealized for the purpose of estimating the expected loss function, are actually slightly offset in time because of the time of concentration of 3 to 5 minutes for one inlet. The peaks are also attenuated slightly by the channel water storage within the gutters. Thus, the flooding condition represented in figure 9 is slightly more severe than what is actually expected.

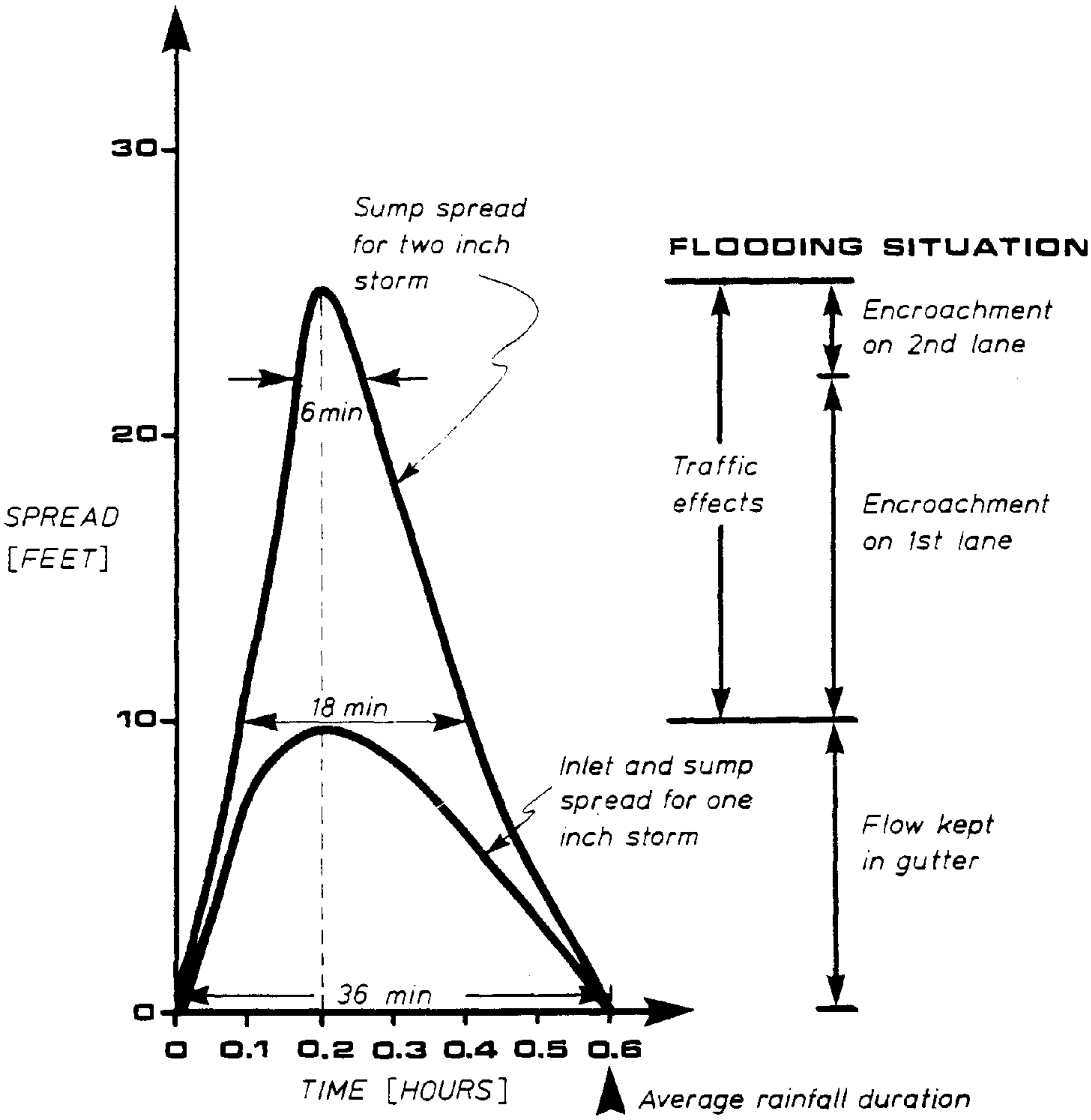


Figure 9. Idealized spreadograph.

Estimating the Loss Function Slope, K.- Losses associated with roadway flooding are attributable to increased accidents, increased operating costs (mainly due to idling), and delay costs associated with loss of time which would otherwise be used for other pursuits. Accidents are related to speed and wet or dry weather conditions. Accidents increase with speed and wet conditions independently of roadway drainage. Roadside flooding , however, reduces speed and mitigates the effects of wet conditions which are more the design responsibility of the pavement engineer than the drainage engineer.

The traffic engineering firm of Bellomo McGee, Inc., Vienna, VA (BMI) made sensitivity calculations of losses associated with an urban four-lane freeway with an Average Daily Traffic (ADT) of 60,000, assuming a traffic distribution of 10 percent of ADT in rush hours, 5 percent in midday, 3 percent in the evening, and 1 percent at night. The probability of rainfall in each of these periods is assumed to be uniform, although there is evidence that probabilities are higher in the late afternoon, a period coincident with the rush hour.[11] Furthermore, BMI used the AASHTO traffic economics methodology and recent delay time estimation methods documented by Morales.[1,10] The BMI computations indicated a loss distribution of 92 percent associated with time delay, 7 percent with increased operating costs, and 1 percent traffic accidents. This distribution is representative of heavy storms that cause traffic blockage on urban freeways.

Given that accident losses and increased operating costs are small in comparison to delay costs, the loss approach presented below focuses on delay costs. If one wishes to include accident and operating costs, the estimates of the **K** factor should be increased by 10 percent prior to the establishment of the economic ratio.

A sound delay time estimate can be found using the LOTUS 1-2-3 spreadsheet DELAY.[10] DELAY calculates the total delay, in vehicle hours, for an incident that obstructs or blocks traffic, such as flooded gutters. The program requires the analyst to know or estimate several traffic flows (vehicles/hour):

- Capacity of the facility.
- Demand flow at the time of the incident.
- Bottleneck flow rates. Note: typical values are given in the reference.[10]
- The total time the facility is completely blocked.

DELAY embodies expert knowledge of the traffic engineer. It represents a precise method of estimating traffic delays for the purpose of calculating losses based on the value of the vehicle occupants' time.

The concepts found in DELAY are simplified for approximate application in figure 10 for demonstration of LTEC. The independent variable is encroachment time which is intended to represent the period of time spread exceeds allowable spread. The dependent variable is the expected traffic delay time. Two conditions are modeled with figure 10: partial flooding and full flooding.

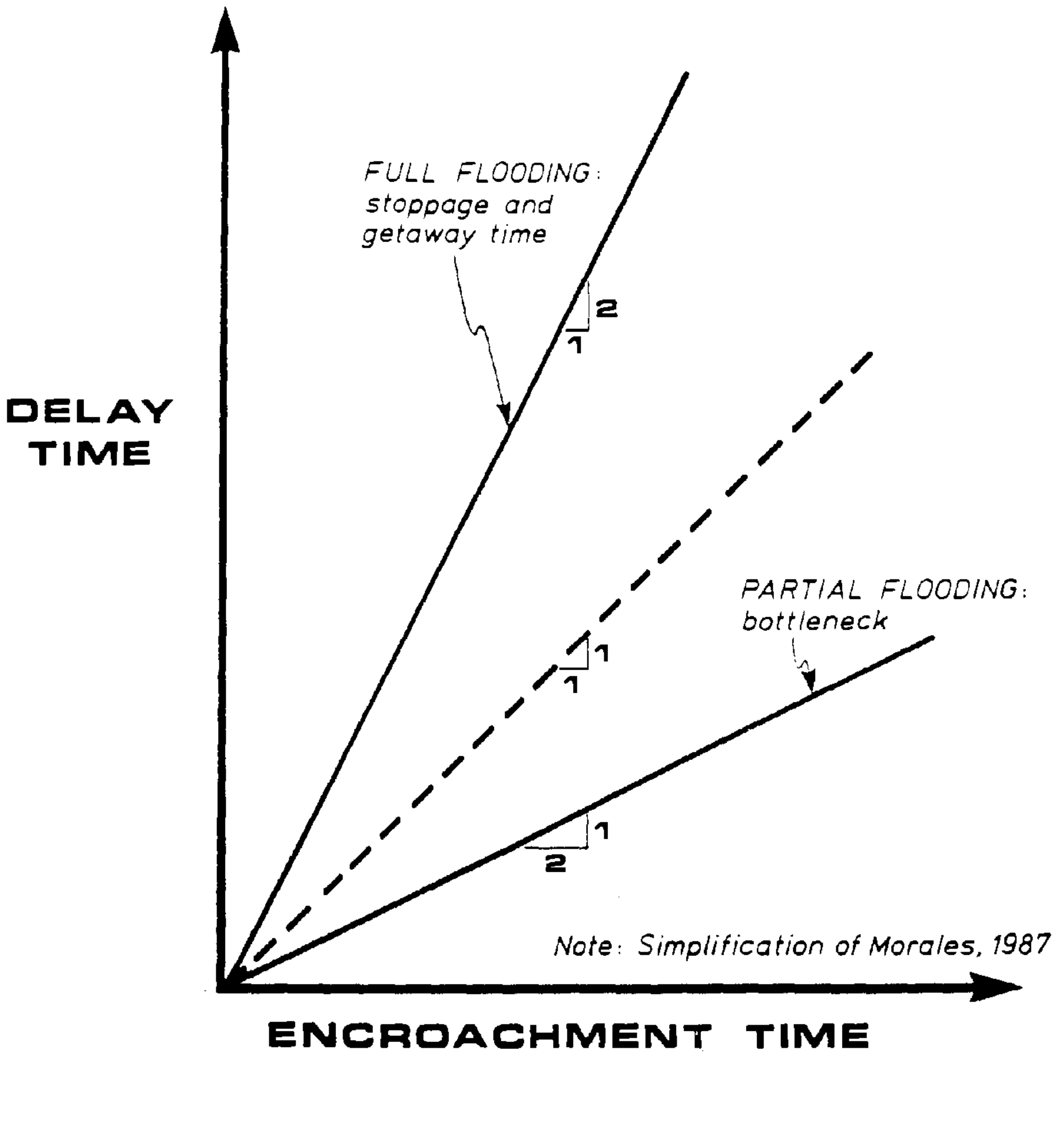


Figure 10. Simplified delay function.

With partial flooding, a bottleneck is formed and delays are a function, in this case one half, of the duration of the partial flooding. With full flooding, traffic is completely stopped and must make a getaway once the blockage is removed; delays are a multiple, in this case of two, of the duration of full flooding. This delay function model addresses the conditions Morales describes: bottlenecks, stoppages, and getaways.

The factor **K** is estimated using the relationships given in figures 9 and 10. Factor **K**, the delay cost, is assumed to be a function of delay time due to flooding, ADT, vehicle occupancy rate (adults/vehicle), and the value of the passengers' time. Delay time is related to encroachment time by the simplified function depicted in figure 10. For an idealized storm, it is assumed from figure 9 that the water spread will be wide enough to have an effect on traffic for 18 minutes out of the storm's 36 minute duration, and be wide enough to encroach on two lanes of traffic for 6 of the 18 minutes.

Thus, for a two-lane road, the total delay time will be equal to the delay time due to 12 minutes of partial flooding plus that due to 6 minutes of full flooding (total blockage); or, using figure 10, delay time = $(1/2)\cdot 12$ minutes partial flooding + $(2)\cdot 6$ minutes full flooding = 18 minutes total delay time. For a four-to six-lane road, it is assumed that the water will not encroach past the second lane, and thus the 18 minutes of traffic effect can be interpreted as $(1/2)\cdot 18$ minutes partial flooding = 9 minutes total delay time. To summarize, for a two-lane road there is partial and full flooding, and for a four or more lane road there is only partial flooding associated with a 1-inch rainfall excess over design rain.

Values for ADT are typical.[5] Vehicle occupancy rate is assumed to be 1.56 adults/vehicle.[1] Passenger time is valued at $7.00/hr for cars and $13.47/hr for trucks in 1987 dollars.[1] Expected cost for a particular period of day (e.g. rush hours) is then expressed as:

Delay cost = delay time · fraction of ADT associated with the period of day · ADT · rainstorm probability associated with the period of day · number of adults per vehicle · (fraction of cars · value of car passenger time + fraction of trucks · value of truck passenger time) ....(20)

The traffic distributions during the day are assumed to be the same as those analyzed by BMI: 10 percent rush hours, 5 percent midday, 3 percent evening, 1 percent night as a percentage of ADT. The probability of rain is 25 percent at any of the four traffic intervals. The factor **K** is listed in table 6 for various road types and is the sum of the delay costs for each period of the day which can be written:

$$K = 0.001235\ (\text{delay time in minutes} \cdot \text{ADT}) \cdot [7 + 6.47 \cdot (\text{fraction of trucks})] \qquad (21)$$

Table 6. Estimated values for K.

Situation	Assumed ADT	K ($/inch)	
		0% Trucks	20% Trucks
2-lane local/ collector	2,000	311	369
2-lane local/ collector	4,000	622	738
2-lane rural arterial	9,000*	1,400	1,659
2-lane urban arterial	15,000*	2,334	2,766
4-lane rural freeway/arterial	40,000*	3,112	3,688
4-lane urban freeway/arterial	60,000*	4,668	5,531
6-lane urban freeway	90,000*	7,002	8,297

* at practical capacity assuming vehicles per hour are 10 percent ADT at rush hour.[5]

APPLICATIONS GUIDE

This section defines the required data for selecting a design rainstorm, presents a nomograph that serves as a convenient design tool and discusses the underlying assumptions.

Required Data.- The following are needed data.

1. N - the number of rains per year in excess of 0.2 inches. The expected value of this datum is 60 east of the Mississippi River, which is approximately one half of the total number of storms presented in table 2 for twenty cities.

2. $q_T\bar{t}$ - the gutter flow times the average rainfall duration. The gutter flow, q_T, is calculated with equation (1) which uses estimates of T, n, S_x, and S. Allowable spread, T, is selected by the analyst and should represent a value for which the gutters are flowing full but not interfering with vehicular traffic. Manning's n is usually taken as 0.016. The gutter cross slope, S_x, should be for the situation being analyzed -- typically 0.01 to 0.02. The value S is the average grade of the gutters: a realistic estimate is the drop from high point to outfall divided by the representative flow path taken along the gutters to the outfall. The average duration of rainfall, t is 0.6 hours for a large mid-Atlantic region; a site-specific value should be used if known.

3. CA - the impervious area. The runoff coefficient, C, varies between 0 and 1 and is a composite ratio of runoff to rainfall representative of the drainage area. Typically, C is 0.9 for paved areas and 0.2 for grassed areas. For the typical drainage area associated with a highway, C is a weighted average of 0.9 and 0.2. The drainage area, A, is the total acreage of the area being drained.

4. $\mathcal{E}$ - the economic ratio, defined as:

$$\mathcal{E} = (C_1 + r \cdot C_2) \cdot crf \,/\, K \qquad (16)$$

C_1 is the cost of an inlet and **C_2** is the cost of the typical lateral connecting the inlet to the larger interceptors. In 1987 dollars, **C_1** is approximately $2,000/unit. **$C_2$** can be computed by multiplying the lateral lengths by their diameters and then by multiplying the sum by the factor $34.81/ft/ft which is an installed, circular, concrete pipe cost factor. Both **C_1** and **C_2** reflect 1987 Fairfax County, VA public works experience with subdivision development. Local costs should be used when known.

The variable **r** is the ratio of laterals to inlets. For a highway drainage area which is oblong, such as shown in figure 2, **r** = 1/2. For a housing subdivision with a more circular drainage area, **r** could approach unity.

The capital recovery factor, **crf**, is a function of the interest rate and time horizon and converts dollars into dollars per year. For an interest rate of 8 percent and a time horizon of 20 years, **crf** = 0.1. For other rates and horizons, **crf** (also known as the engineering economics "(A|P)" factor) can be found in published compound interest tables.

The factor **K** is the slope of the loss function and is tabulated in table 6 for convenient application. The analyst must estimate ADT (average daily traffic), percentage of trucks, number of lanes, and class of service. The methodology described in the previous section, Estimation of Loss Functions, supports the tabulated values of **K** for 1987 dollars.

Nomograph.- Given the four factors described above, **N**, $q_T t$, **CA**, and ε, the nomograph shown in figure 11 graphically solves equation (15), the decision model, for the total design rainfall, X_d. The design rain intensity is then computed from equation (7), $i(X_d) = 2X_d/\bar{t}$.

Several practical issues associated with this application of LTEC are:

1. The solution for X_d is precise using equation (15) and approximate using the nomograph in figure 11.

2. The solution is based on an average representation of the system. For example, the analyst uses the average grade, **S**, the average cross slope, S_x, and an allowable spread that applies to all gutters in the system.

3. Once $i(X_d)$ is determined, it should be used to establish inlet spacings and subdrainage areas for individual system elements that might vary from the average.

4. A trial design approach can be utilized if individual elements cause the average representation to be distorted.

   a. First use an average set of system parameters to make a preliminary design based on $i(X_d)$.

   b. Then using the preliminary design, recalculate average **S** and S_x and refine the estimate of $i(X_d)$; then use the refined intensity to make a final design.

The LTEC approach is examined and analyzed in three existing site case studies in appendix I: a four-lane freeway near Washington, D.C., a main road arterial through Manassas, VA, and a subdivision local street in Norfolk, VA.

Discussion.- This LTEC analysis is based on several key assumptions: The nature of the losses, the shape of the hyetograph, the distribution of rainfall and the condition of the outfall.

The major economic loss associated with surface drainage failure, represented by gutter flooding into the travelled portion of the street, road, or highway, is loss of transportation function and delays. Although accidents are certainly a concern, the cost of delay is the principal loss of interest to the drainage system designer: rain tends to increase accidents at a given speed, but flooding tends to decrease traffic speeds to safer levels. Reduction of accidents attributable to rain is thus best addressed through pavement design rather than drainage system design. In addition, slow-downs

EXAMPLE

- N = 60 (Eastern U.S.)
- q_T = 3 cfs
- $\hat{t}$ = 0.6 hr (mid-Atlantic U.S.)
- C = 0.85
- A = 20 acres
- C_1 = $2,000
- C_2 = $5,200
- r = 1/2
- crf = 0.1
- K = $10,000
- X_d = 2.17 in; i_d = 7.23 iph

$$\varepsilon = \frac{(C_1 + rC_2)\,\text{crf}}{K}$$

i_d = design rain intensity (in/hr)

$$= \frac{2X_d}{\hat{t}}$$

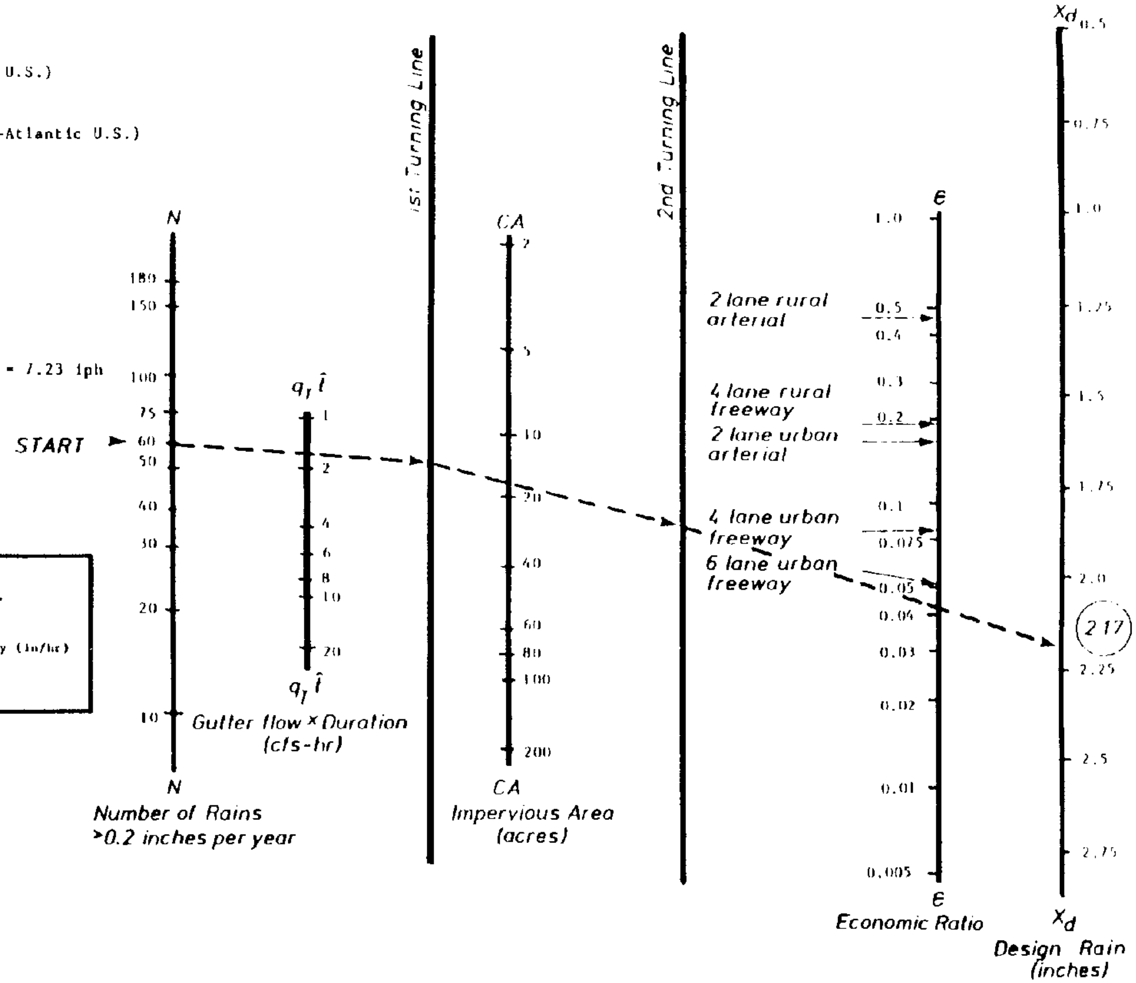


Figure 11. LTEC nomograph for road drainage.

and stops cause some increased operating costs (due to idling, mainly), which are considered a secondary loss. The order of significance of losses (to the drainage system designer) is delays over operating costs over accidents, with the latter two having a combined total contribution of less than ten percent.

A triangular hyetograph of average duration can be used to represent an expected rainstorm. The maximum instantaneous rain intensity is conveniently represented and corresponds with short duration IDF curve estimates which in turn set inlet spacing.

All rainfall can be represented as a daily total, with right-hand tail distributed normally, occurring as one, triangular storm per day with an expected duration. A more complete statistical representation would involve a bivariate distribution of rainfall and duration; this would probably eliminate the ability to use a convenient nomograph.

A free outfall exists that is not backed up by downstream flooding or does not need pumping to evacuate. With this assumption, the simulation experiments show that the interceptors and outfalls, if designed for a ten-year storm, can rapidly reduce ponding with pressure flow for higher return period storms. If this is not the case, the $\mathbf{C_o}$ cost factor, for the underground system, is also a function of $\mathbf{X_d}$ instead of being relatively constant as assumed in this analysis. If all three cost factors, $\mathbf{C_o}$, $\mathbf{C_1}$, and $\mathbf{C_2}$, are functions of $\mathbf{X_d}$ the decision model may be invalid and the LTEC solution must be sought using additional relationships; the concept remains the same but the realization of a solution is more complicated.

Two questions are of particular interest to the potential user of the decision model: what is the sensitivity of the model? and, how can selection of the economic factors be simplified? The sensitivity of the model can be demonstrated by individually increasing each of the example parameters on the nomograph in figure 11 by 10 percent and evaluating the effect on the response. The responses are tabulated in table 7.

Table 7. Sensitivity of the decision model.

Factor	Change in Factor	Change in Design Rain
N: # of rains > 0.2" per year	+ 10%	+ 2.21%
$q_T\bar{t}$: gutter flow times duration	+ 10%	+ 2.21%
CA: impervious area	+ 10%	- 2.21%
ε : economic ratio	+ 10%	- 2.21%

Thus, if there are a higher number of rains or higher gutter flows, the design rain increases. The design rain decreases with higher impervious areas or higher economic ratios. Using economic jargon, the response is inelastic: a 10 percent input change only causes a 2 percent output change. Note that the economic ratio has cost divided by risk factors; thus, if cost goes up, design rain goes down, and if risk goes up, design rain goes up.

The selection of the economic ratio is simplified as follows. Using a **crf** of 0.1, **r** = 0.5 for urban systems and **r** = 1.0 for rural systems, C_1 = \$2,000, C_2 = \$5,200, and the **K** values in table 6, yields the values in table 8.

Table 8. Economic ratios for various road situations.

Situation	0% Trucks	20% Trucks
2-lane local/ collector	1*	1*
2-lane local/ collector	1*	1*
2-lane rural arterial	0.514	0.434
2-lane urban arterial	0.197	0.166
4-lane rural freeway/arterial	0.231	0.195
4-lane urban freeway/arterial	0.099	0.083
6-lane urban freeway	0.066	0.055

* The practical upper limit of the economic ratio is unity, assuming that all facilities will be designed to handle the 2-year storm, regardless of LTEC.

The relationship between the economic ratio and the design rain is inverse, so the rural and two-lane facilities having a higher economic ratio, require a lower design rain for the LTEC design.

One of the case studies presented in appendix I, Stockley Gardens, revealed a practical constraint on LTEC. The very low traffic, local roads generate a very small **K** value; this is because these roads are amenities to residential developments and do not generate sufficient traffic to for there to be significant delay cost under flooded conditions. The main purpose of such roads is not to facilitate throughput traffic movements, but to provide access to residences.

For this low traffic volume case, the LTEC design for avoiding traffic delays breaks down; the optimum design rain is too small to satisfy land owners who want to get in and out of their improved properties and who essentially pay development costs within the prices of their homes. In this case, flooding of improved properties may replace traffic delay as a dominant economic factor. Given that LTEC calls for very low design rains in low traffic cases, a reasonable constraint on LTEC is that a 2-year rain, at least, should be used. This translates to a maximum economic ratio of approximately unity. Thus, $\mathcal{E} = 1$ is a practical upper limit and the design rain should equal or exceed the 2-year storm.

ASSUMPTIONS AND LIMITATIONS

Assumptions.- This LTEC method of design for road surface drainage system was derived with certain asumptions summarized as follows:

1. The rational method is valid for this application.

   The rational method is used to determine the total flow to be handled by the inlets (page 20, equation (8)). In using the rational formula, average values are deemed adequate for describing a complex system.

2. A triangular hyetograph is appropriate for representing an expected storm.

   This assumption provides a simple relationship between design rainfall and maximum intensity, where maximum intensity determines inlet spacing.

3. Rainfall is assumed to have a standard normal distribution.

   The risk model of equation (11) relies heavily upon the rainfall distribution. The assumptions associated with using the standard normal distribution in the risk model are as follows:

   - Precipitation data for 1983 from 20 cities east of the Mississippi adequately represent conditions of other years and other locations in the United States east of the Mississippi.

   - The data, having an average median rainfall of 0.2 inches and average standard deviation of 0.94, has a distribution which closely approximates the right hand tail of a normal curve with mean of 0 and standard deviation of 1.

   - Rainfall for a single day can be represented using a single duration.

4. The loss function is assumed to be linear.

   The risk model of equation (11) is also a function of the loss due to traffic delays caused by street flooding. Cost of delay time is considered to be the major loss, although the user can incorporate losses due to accidents, increased vehicle maintenance, and property damage. The factor K, the slope of the loss function, is calculated as a linear function of delay time, which in turn, is assumed to be a linear function of the time that water encroaches upon the traveled areas of the road. To simplify the calculation of K, the user is encouraged to assume uniform probability of rainfall throughout the day, an average daily traffic flow, an average vehicle occupancy rate, and average values of the worth of passengers' time.

5. Average values of drainage system features are adequate for determining LTEC.

   The user is required to calculate an average gutter flow (the average flow received by each inlet) based on an average allowable spread, average street grade, and average gutter cross slope. Average rainfall duration and runoff coefficient are also required by the decision model.

6. A free outfall condition exists for the drainage system.

   The outfalls and interceptors of the system are assumed to be hydraulically overdesigned for durability and preventing clogging. Thus, the cost of these components is not a function of design rain in the cost model of equation (10), which simplifies the decision model of equation (15).

Limitations.- In order to apply this LTEC method of design properly, a clear understanding of its limitations is important.

1. The decision model may be less accurate for design rains greater than 2.0 inches.

   The risk model integral of equation (11) was approximated using an exponential function. Outside the range of 0.2 to 2.0 inches, the exponential approximation may be inappropriate.

2. The model may not be applicable to locations west of the Mississippi River.

   Rainfall distribution selection was based on data collected for 20 cities located east of the Mississippi River.

3. Modifications to the loss function are necessary to address systems with low traffic but that drain properties of high value.

   The loss function of this study assumes negligible losses due to flooded property. Thus, the LTEC design rain is impractically small for residential streets with low traffic where delay time losses are minimal.

CONCLUSIONS AND RECOMMENDATIONS

1. A method for Lowest Total Economic Cost (LTEC) design of surface drainage elements composed of gutters, inlets, and storm sewers is practical and feasible. It should be refined and implemented as a means to optimize street and drainage infrastructure development or rehabilitation. It is applicable to freeways, arterials, and major collectors; it may not apply to local streets.

2. The method should be tested to find institutional means for using LTEC rather than fixed return period criteria. Or, as an alternative, the fixed criteria should be checked against LTEC results to get criteria into general conformance with known LTEC optimums for general classes of facilities.

3. The LTEC method is nomographic and requires as input:

   - Expected number of yearly rains with rainfall in excess of 0.2 inches.
   - Gutter flow that the average inlet is expected to or capture for entry into the storm sewers.
   - Duration of the average rainfall.
   - Runoff coefficient and drainage area.
   - Cost of a typical inlet.
   - Cost of a typical lateral that connects the inlet to an interceptor.
   - Appropriate capital recovery factor.
   - Traffic-related cost associated with that extreme event having one inch of rain in excess of the normal rain that fills, but does not overflow, gutters.

4. The traffic related costs, tabulated for convenience of application, are mainly attributable to delay costs.

5. The geographic regions for which known values exist for the factors used in these methods are:

   - Normal distribution: USA, east of Mississippi River
   - Number of storms: USA, east of Mississippi River
   - Rainfall duration: USA, mid-Atlantic region
   - Inlet and lateral costs: USA, Fairfax County, VA
   - Traffic losses: USA

   Application outside these regions requires extrapolation or additional site-specific estimates.

6. The hydraulic characteristics of surface drainage systems, as revealed by simulation experiments, are:

   - Inlet spacing is determined by the maximum intensity at the apex of the triangular hyetograph.
   - At the upper ends of storm sewer systems, the laterals are oversized and the inlets control the flows.
   - At the lower ends of storm sewer systems, slight pressure enables interceptors and outfall pipes to rapidly eliminate ponding, provided a free outfall condition exists.
   - When the allowable spread condition at the inlets is exceeded, bypasses flows rapidly move to the low point or sump.

7. Logical additional LTEC developments are:

   - Confirm the truncated Gaussian total rain distribution west of the Mississippi River or other regions of interest.
   - Widen the region of known storms per year and storm duration.
   - Explore the feasibility of using bivariate rain distributions of amount and duration as a refinement.
   - Investigate rainfall data from the perspective of generalizations that will support LTEC, such as development of a national data file for support of LTEC estimates, similar to the national HYDRO data file.
   - Confirm the linearity of the loss function and refine tabulations of loss function slope.

- Extend loss function estimation to include property losses to make the method applicable to local streets with low traffic but high property value.

- Integrate the DELAY methods directly into the analysis.

- Conduct additional case studies.

- Continue simulation experiments using HYDRA.

8. The insights necessary to define the decision model and its nomograph are the direct result of running dynamic routing experiments with the HYDRA model on a personal computer. The logical process involves exploring how the physical system works with multiple experiments using the HYDRA model, and then depicting the interactions with simplifications that can be linked to get a closed solution. The overall findings are indicative of the utility of personal computers in the engineering workplace.

9. It is also concluded that small computer experiments and simulations on the previous LTEC target problems of culverts and bridges should be conducted. These previous analyses should be revisited to seek insights and simplifications that would parallel the development of the closed solutions presented in this paper.

APPENDIX I.- Case Studies

Objective and Method of Analysis.- The objective is to investigate three actual sites and evaluate the impact of an LTEC design had it been invoked as part of their surface drainage system design. The method is to collect the site-specific, "as built" data that would be used in the decision model. From these data, the design rainfall intensity is inferred which is in turn used to estimate the inferred return period from site-specific IDF curves. The same data is then used, along with best estimates of existing traffic and costs, to determine the LTEC rainfall, rainfall intensity, and return period.

The Three Sites.- The three case studies represent a freeway, an urban four-lane road, and a residential, local street. All three sites have or had real or perceived flooding problems.

The freeway is represented by the George Washington Memorial Parkway. The section under study, located along the western shore of the Potomac River across from the District of Columbia, consists of five lanes of traffic passing under the Pennsylvania Central Railroad bridge, a METRO line bridge, and I-395's 14th Street bridge.

Grant Avenue in Manassas, VA represents an urban four lane road. The system of interest drains a portion of roadway passing through a business district of town. The Southern Railroad, which contributes some runoff to the drainage system, passes over the low point of Grant Avenue.

Stockley Gardens Road is a residential street in an older neighborhood of Norfolk, VA. Its drainage system was recently upgraded with additional inlets and the main interceptor is sized to handle flow from the immediate vicinity as well as from 119.41 acres of urban area upland of Stockley Gardens.

Analysis and Results.- The site-specific data used in the analysis for each location is presented in table 9.

Table 9. Site-specific data.

	GW Parkway	Grant Avenue	Stockley Gardens
Location	38°50′ N 77°03′ W	38°45′ N 77°28′ W	36°53′ N 76°15′ W
Drainage area (ac)	3.68[1]	18.42	88.68
Runoff coefficient	0.73	0.73	0.59
No. of inlets	12[1]	24	56
Laterals/inlet, r	0.25	0.58	0.55
Avg. gutter length (ft)	306	202	655
Number of lanes	5	4	2
Gutter cross slope (ft/ft)	0.015	0.038	0.038
Grade (ft/ft)	0.005	0.031	0.003
Gutter Manning's n	0.015	0.014	0.012
Allowable spread (ft)	6	6	8
Estimated time of concentration (min)	<5	<5	10.5
Gutter flow (cfs)	0.28	3.58	2.80
Average daily traffic	58,000[2]	16,000[3]	1,300[4]

[1]Areas not directly contributing runoff to the roadway and their associated drainage system elements were excluded from the analysis.

[2]Estimated from traffic counts by BMI.

[3]Washington Council of Governments estimate.

[4]Estimated by counting dwelling units and assuming seven trips per dwelling unit.

For each case, the inferred design rainfall intensity is determined by first solving for the gutter flow, q_T, in equation (1) and then applying the rational formula such that,

$$i(X_d) = q_T / (C \cdot a) \qquad \text{(A1)}$$

Inferred design rainfall is determined from the triangular hyetograph relationship given in equation (5), $X_d = i(X_d) \cdot t / 2$, where average duration t is assumed to be 0.6 hours. The inferred design return period is interpolated using site-specific IDF curves (such as can be generated by the software package HYDRO) by finding the intersection of the inferred design intensity and the time of concentration (estimated using HEC-12 methods). The information in table 9, along with the cost information in table 10, is then used with the LTEC decision model [equation (15)] to determine the LTEC design rainfall.

Table 10. Cost data.

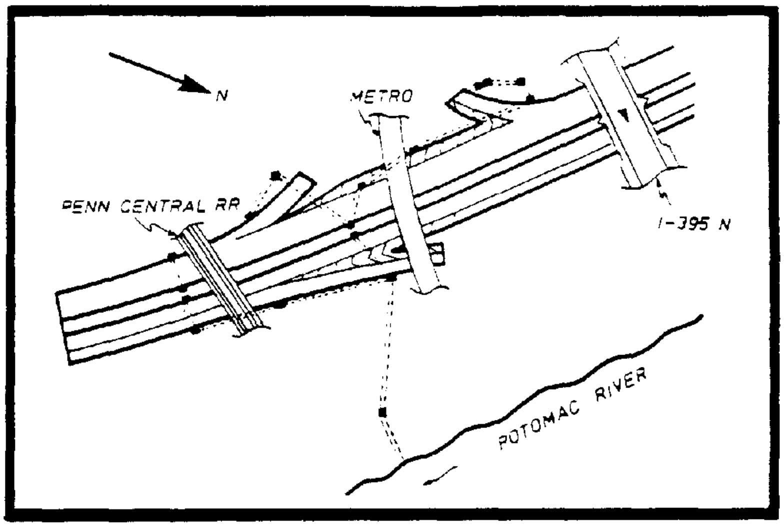
	GW Parkway	Grant Avenue	Stockley Gardens
Cost per inlet, C_1^*	$2000	$2000	$2000
Cost per lateral, C_2^*	$3100	$1800	$2600
Loss function slope, K ($/inch)	4513	1475	204
Capital recovery factor, crf	0.1	0.1	0.1

* from Fairfax County, VA current estimates

LTEC design intensity and return period are found using, respectively, the triangular hyetograph relationship and the IDF curves for the particular location. Table 11 presents both the inferred and LTEC results for the three cases; the method seems inappropriate for Stockley Gardens. Figures 12, 13, and 14 illustrate return period estimation using IDF curves for the three sites.

Table 11. Results from inferred and LTEC analyses.

	GW Parkway		Grant Avenue		Stockley Gardens	
	Inferred	LTEC	Inferred	LTEC	Inferred	LTEC
Design rain, X_d (inches)	0.38	2.20	1.91	1.72	2.25	0.02
Design intensity, $i(X_d)$ (in/hr)	1.27	7.32	6.37	5.72	7.52	0.063
Design return period (years)	<1	19	6	3	100	<1



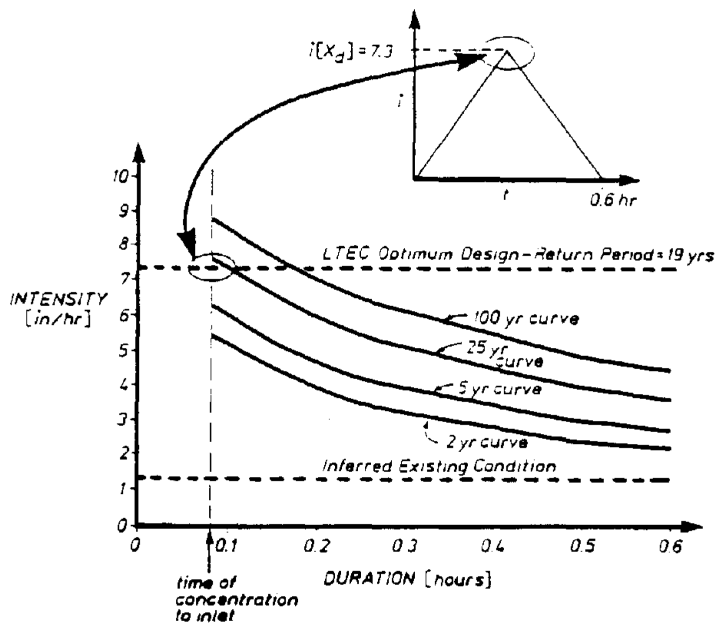


Figure 12. Site sketch and IDF curves for the George Washington Memorial Parkway.

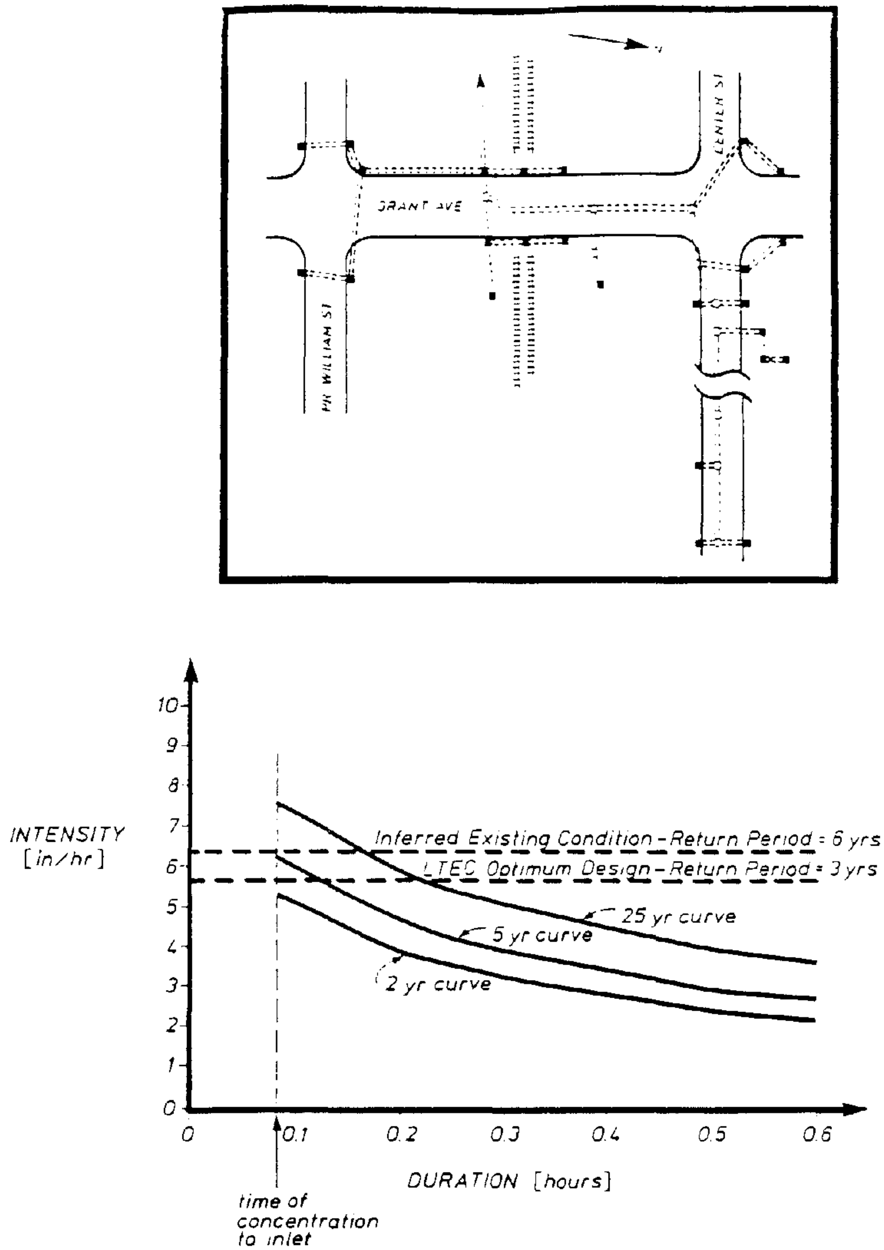
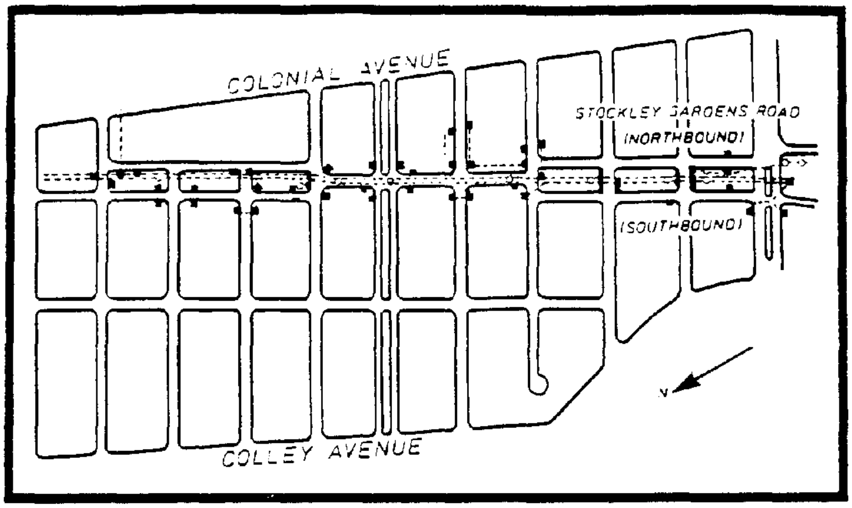


Figure 13. Site sketch and IDF curves for Grant Avenue.



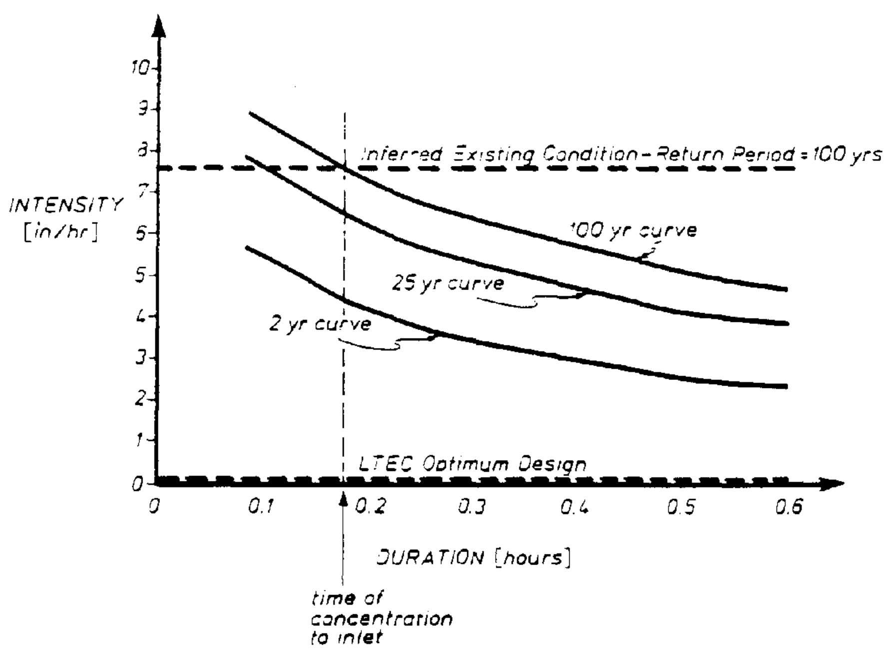


Figure 14. Site sketch and IDF curves for Stockley Gardens.

Figure 15 presents the relationship between design rainfall and the annualized risk cost, construction cost, and total cost for Grant Avenue. The risk curve is a plot of equation (13), while the construction cost curve is a plot of the annualized form of equation (10). The total cost curve is the sum of the risk and construction costs and represents equation (14). Since the design model, equation (15), is derived by setting the derivative of the right hand side of equation (14) equal to zero, the LTEC design rainfall occurs at the minimum of the total cost curve.

Conclusions.- Generally, the LTEC method and actual practice do not agree. Specific findings from these case studies are:

- The George Washington Parkway is very underdesigned, which is in agreement with local experience with the site.
- The system for Grant Avenue was designed close to the LTEC optimum.
- LTEC is not appropriate for applying to the residential area of Stockley Gardens. If frequent flooding is unacceptable in low traffic areas, then economic justifications other than avoiding traffic delay costs should govern design for local streets.
- The economics of LTEC should be expanded to include property damage to cover locations with low traffic and high property values.

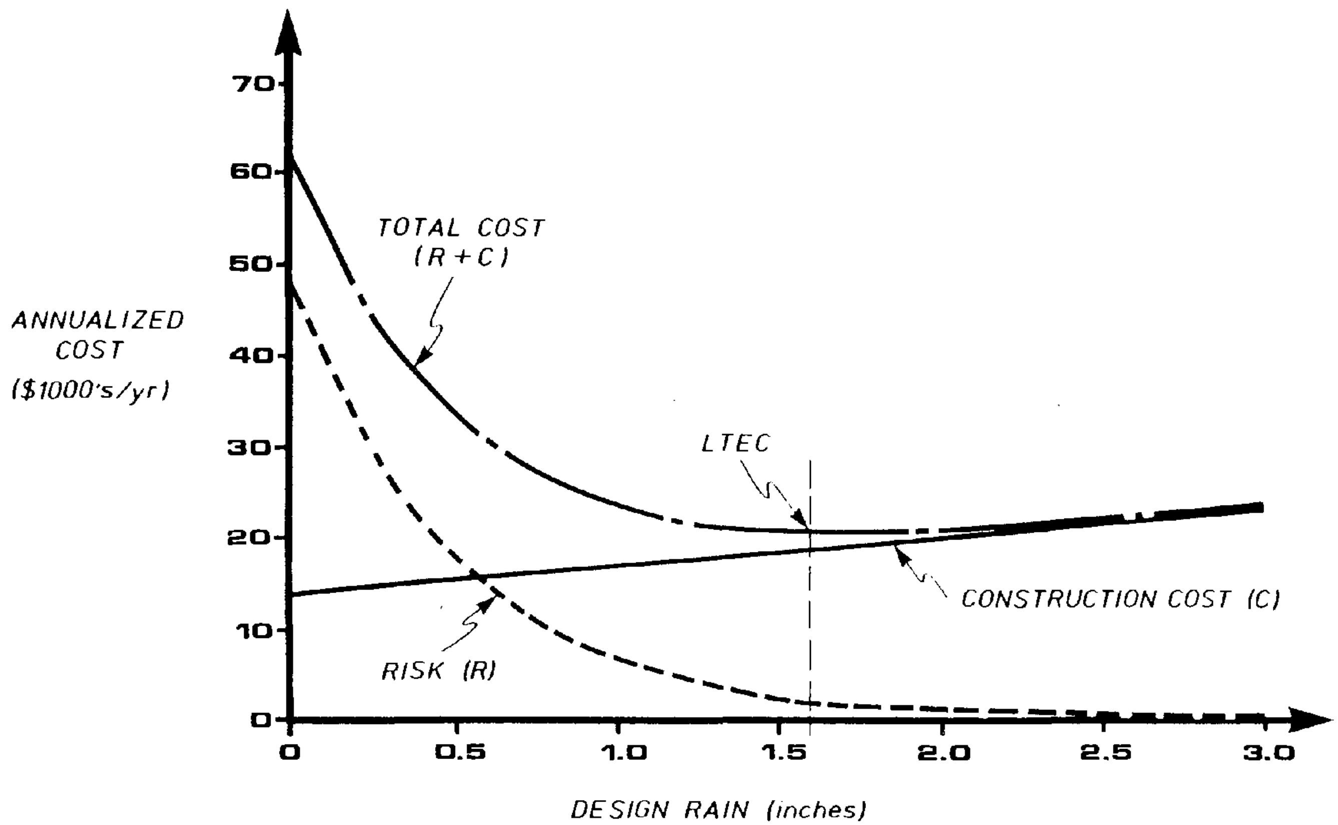


Figure 15. Road drainage system cost response curve for Grant Avenue.

APPENDIX II. Notation

A - the area of the surface drainage system. (acres)

a - the area contributing runoff to one inlet. (acres)

α - the ratio of the time to peak intensity to the average total storm duration for a triangular hyetograph.

C - the runoff coefficient as used in the rational method. (dimensionless)

C_o - the total cost of the subsurface pipe system consisting of interceptors and outfalls which is relatively fixed and independent of the design rain. ($/system)

C_1 - the cost of an inlet and its structure. ($/inlet)

C_2 - the cost of a lateral (the pipe connecting each inlet to an interceptor). ($/lateral)

crf - capital recovery factor

ε - the economic ratio defined as:
$\varepsilon = (C_1 + rC_2)crf / K$

$f(x)$ - the probability function itself, shown in figure 6.

i - the rainfall intensity. (in/hr)

K - the slope of the loss function. The assumed loss function is shown in figure 7. ($/inch)

L - inlet spacing (ft)

$L(x)$ - the loss function itself, shown in figure 7.

m - the number of inlets within A, each of which is assumed to handle a flow of q_T. (#)

N - the number of events per year with greater than 0.2 inches of rain. (#)

n - the Manning's friction factor.

Q - the hydraulic capacity of the entire drainage system. This variable represents the sum of all the inlet flows at the inlet design capacity. (cfs)

q_T - the gutter flow, for spread T, calculated using equation (1). This flow is assumed to be the flow received by one inlet. (cfs)

$R(X_d)$ - risk cost ($)

r - the ratio of laterals to inlets. Figure 8 shows a schematic of typical drainage networks with r = 1/2 for highway networks (oblong drainage areas) and r = 1 for business and residential networks (circular drainage areas).

S - the typical or average street grade. (ft/ft)

S_x - the typical or average gutter cross slope that is representative of all the gutters in the surface drainage system. (ft/ft)

T - the allowable gutter spread. (ft)

$\bar{t}$ - the average duration of rainstorms in the vicinity of the drainage system. (hours)

t_c - time of concentration to one inlet. (hours)

w - width of pavement contributing runoff. (ft)

X - the total rain associated with a rainfall event, which is the area under the hyetograph. (inches)

X_d - the design rain which, when utilized in design, will yield the LTEC design. (inches)

APPENDIX III. Unit conversions

To convert from:	To:	Multiply by:
ft	m	0.305
in	cm	2.54
in/hr	cm/hr	2.54
ac	m^2	4047
	ha	0.405
ft^3/s	m^3/s	0.0283

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