URBAN DRAINAGE PLANNING OF PALEMBANG CITY - A CASE STUDY

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

WATER RESOURCES DEVELOPMENT

By

JONI RAHALSYAH PUTRA

DEPARTMENT OF WATER RESOURCES DEVELOPMENT AND MANAGEMENT
INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE -247 667 (INDIA)

JUNE, 2007

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the dissertation entitled " **URBAN DRAINAGE PLANNING OF PALEMBANG CITY-A CASE STUDY**" in partial fulfillment of the requirement for the award of degree of **MASTER OF TECHNOLOGY** in **WATER RESOURCES DEVELOPMENT** and submitted in the Department of Water Resources Development and Management of Indian Institute of Technology Roorkee is a record of my own work carried out during a period from July 2006 up to June 2007 under the supervision of **Dr. S.K. Mishra**, Assistant Professor and **Dr. M.L. Kansal**, Associate Professor, Water Resources Development and Management Department, Indian Institute of Technology Roorkee, Roorkee, India.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree .

Dated : June 13 , 2007 **JONI RAHALSYAH PUTRA**

This is to certify that the above mentioned statement made by the candidate is correct to the best of our knowledge.

Dr. M.L. KANSAL
Associate Professor,
Department of Water Resources
Development and Management
I.I.T. Roorkee
Roorkee-247667
INDIA

Dr. S.K. MISHRA
Assistant Professor,
Department of Water Resources
Development and Management
I.I.T. Roorkee
Roorkee-247667
INDIA

ACKNOWLEDGMENT

I would like to take this opportunity to express my sincere and profound gratitude to **Dr. S.K. Mishra,** Assistant Professor and **Dr. M.L. Kansal**, Associate Professor, Water Resources Development and Management Department, Indian Institute of Technology Roorkee, for their guidance, encouragement and suggestions at every stage of this dissertation, in spite of their busy schedule, without which it would been very difficult to complete this work in time.

I express my sincere gratitude also to **Dr. S.K. Tripathi**, Professor and Head of Department, all the Faculty members and staff of WRD&MD IIT-Roorkee for the facilities extended.

I am also thankful to all my co-trainee of 50th WRD and 26th IWM Batch, WRDMD, for their cooperation in the completion of the works.

I wish to express my wholehearted gratitude to the **Ministry of Public Works, Director General of Water Resources Development, Chief of Water Resources Development Office of South Sumatera Province**, which gave me opportunity to study at this premiere institution for M.Tech degree, for their support and encouragement in my study.

Ultimately, a special and sincerest thanks to my father **H. Alwie Hamry**, my mother **Hj. Rahmawati Rahim**, my wife **Tri Indiani** and my loving son **Atthallah Putra Ramadhan** for their persistent support, encouragement and prayers throughout the duration of my study at WRD&M . Also I am highly indebted to my big family for their praying and support, and all the help during my absence from Indonesia.

Roorkee,

Dated : June 13 , 2007

JONI RAHALSYAH PUTRA

ABSTRACT

The rainfall-runoff process in an undeveloped area is primarily determined by the natural surface detention, infiltration characteristics, and the drainage pattern formed by the natural flow paths. The type of the surface soil, the nature of vegetative cover, and the topography are the governing factors. The natural rainfall-runoff process is altered in urbanizing areas. Part of the land surface is covered by impervious material due to urbanization. The water courses are cleared, deepened, and straightened to improve their conveyance capacities.

New man-made drainage facilities are added to the drainage system. A typical urban land cover consisting of impervious rooftops, streets, and parking lots allows far less surface retention and infiltration than an undeveloped land. Moreover, stormwater runoff occurs over smooth, impervious surfaces, and in man-made or improved natural channels with increased velocity. As a result of these factors, urbanization increases the stormwater runoff volumes and rates, and possibly causes flooding of downstream areas. It can also accentuate downstream channel erosion.

In response to these critical problems, engineers and scientists have developed many innovative techniques to analyze urban hydrology. They have also designed many innovative structures to control urban flooding and improve stormwater quality. These analysis techniques and design structures rely heavily on numerical methods and computer models. Thus, desktop methods and empirical models are giving way to new, physically based techniques that are embedded in modern computer software.

Storm Water Management Model (SWMM) is a rainfall runoff simulation model which can be used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas.

Application of SWMM provided in this dissertation for simulating single event storm runoff. Study deals with existing drainage system of Sekanak System. Field survey and field data have been collected to provide properties of subcatchments and existing drainage as per requirement for use of SWMM. Even though it is combined sewer system in Sekanak Drainage System, only 50 year storm runoff drainage has been considered. It is found that existing layout and capacities are adequate to handle 50 year storm event. It is possible to make further improvement in simulation study of Sekanak Drainage System by increasing the number of subcatchments (through subdivision) and incorporating more accurate properties of subcatchments, channels based on field measurements.

Computer based ***Storm Water Management Model*** (SWMM) can be used in several practical applications. This study provides an understanding of urban hydrology, i.e. how to plan urban drainage. The latest version of SWMM 5 software developed by US EPA has been studied and applied in this study.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ANNEXURES

WATER RESOURCES DEVELOPMENT & MANAGEMENT
INDIAN INSTITUTE OF TECHNOLOGY – ROORKEE

No. WR/E-21/2007/ 508

Assistant Registrar (ACD)
I I T – Roorkee

Dated : June 21, 2007

Please find enclosed herewith a sealed envelope containing the marks of M. Tech Dissertation viva-voce examination held on 21st June 2007 of Mr. Joni Rahalsyah Putra, trainee officer of this department alongwith a copy of his M. Tech dissertation entitled "Urban Drainage Planning of Palembang City – A case study".

Encl : As mentioned

(S.K. Mishra)
Assistant Professor

CHAPTER I

INTRODUCTION

1.1. THE DEFINITION OF URBAN HYDROLOGY

Hydrology may be defined as the physical science which treats the waters of the Earth, their occurrence, circulation and distribution, their chemical and physical properties, and their reaction with the environment, including their relation to living things (UNESCO, 1979). These words serve to emphasize two particular aspects of the subject: its interdisciplinary nature, which embraces physical, chemical and biological as well as applied sciences; and its concern with the spatial and temporal distribution and movement of water in all its forms. The latter is implicit in the concept of the hydrological cycle, which illustrates the multifarious paths by which the water precipitated on to the land surface finds its way to the oceans, where evaporation provides the supply of moisture for the renewal of the process.

The hydrological cycle is commonly presented in pictorial form, of which *FIG. 1.1*, adapted from Todd (1959), provides a typical example.

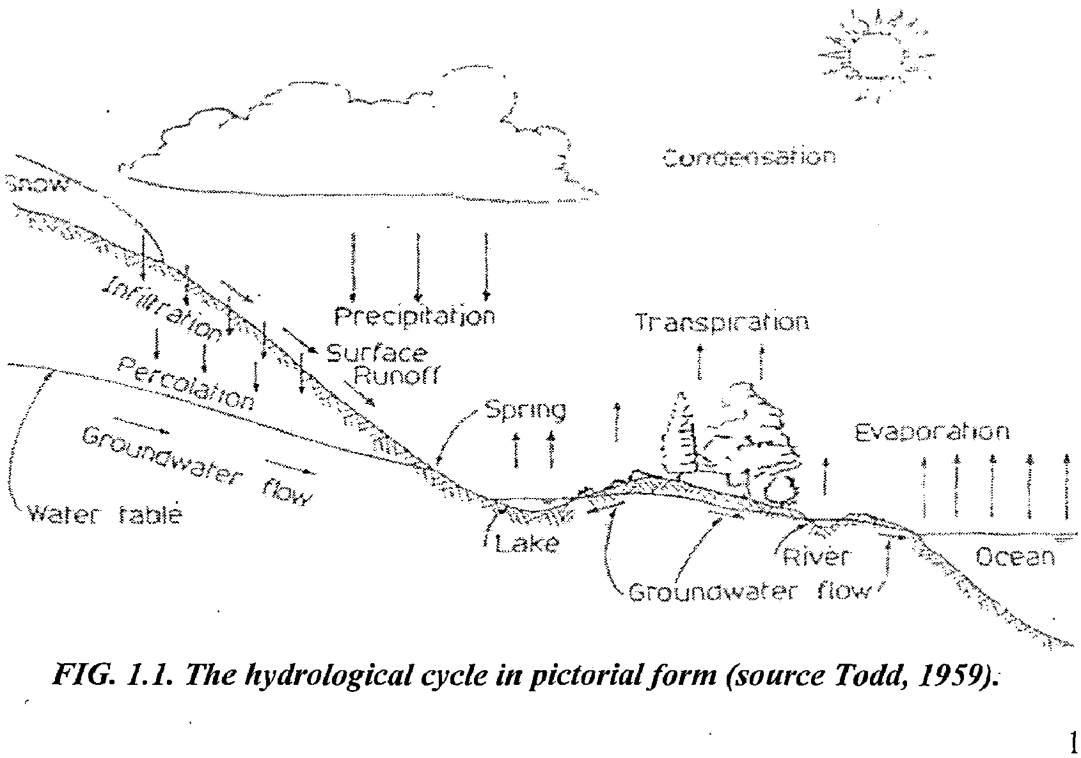


FIG. 1.1. The hydrological cycle in pictorial form (source Todd, 1959).

Although *FIG. 1.1* is useful in imparting the essential features of a water cycle driven by the excess of incoming over and outgoing radiant energy, this representation fails to provide an adequate framework that can be obtained by adopting the so-called system notation, in which the paths of water transport link the major sources of moisture storage, as presented by Dooge (1973) in *FIG. 1.2.*

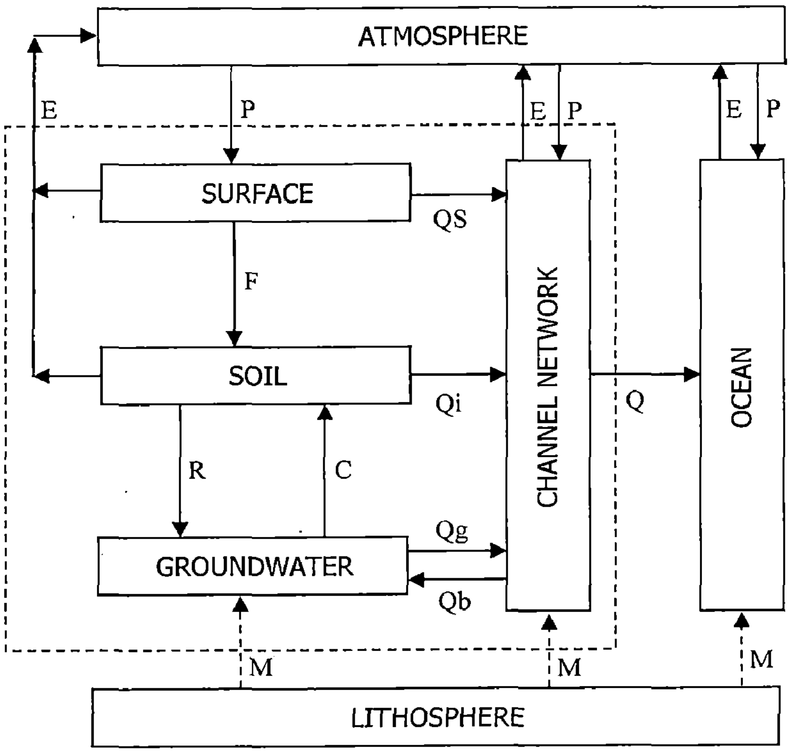


FIG. 1.2. The hydrological cycle in system notation (modified from Dooge, 1973).

A closer examination of *FIG. 1.2.* reveals that hydrologists do not in fact concern themselves with the whole of the hydrological cycle. The oceans are the province of the oceanographer, the atmosphere is studied by the meteorologist, and the lithosphere by the geologist. What remains is commonly referred to as the land phase of the hydrological cycle. This subsystem, whose limits are shown by the broken line in *FIG. 1.2.*, receives an input of precipitation, **P**, and produces outputs in the form of evaporation, **E**, and river flow, **Q**. Further subdivision is possible in order to demarcate the interests of other specialist groups. For example, the soil scientist may confine his interest to the upper soil horizons, which receive water by infiltration, **F**, or capillary rise, **C**, and lose water by evaporation, **E**, deep percolation, **R**, or throughflow, **Qi**. Nevertheless, despite the improvement in the level of comprehension afforded by ***FIG. 1.2.*** over ***FIG. 1.1.***, an additional important element is missing – that of the influence of man.

Since time immemorial, man has manipulated his environment, and therefore the land phase of the hydrological cycle, for his own purposes. Wildscape has been cleared for agriculture, forests have been cut, swamps have been drained and, most important of all, towns and cities with all their associated infrastructure have been created in what were once rural areas. Over the last 25 years, increased attention has been devoted to the hydrology of land use changes in general, but only the process of urbanization has given rise to a new and recognizable branch of the subject – urban hydrology.

Perhaps the most obvious definition of urban hydrology would be the study of the hydrological processes occurring within the urban environment. However, further consideration of the hydrological cycle of an urban area, as presented in ***FIG. 1.3.***, soon reveals the inadequacy of this simplistic conception. The natural drainage systems are both altered and supplemented by sewerage. The effects of flooding are mitigated by flood alleviation schemes or storage ponds. In the initial stages of urban development, septic tanks are employed for the disposal of domestic wastes. As the urban area grows, foul sewerage systems discharging to sewage treatment works are installed, and the treated effluent is returned to local watercourses or even the ocean. Initially, water supplies are drawn from local surface and groundwater sources at minimum cost, but as the population increases the demand for water rises, further supplies may only be obtainable from more remote locations. Both waste disposal and water supply therefore extend the influence of the urban area well beyond its immediate boundaries. Urban hydrology may consequently be defined in more appropriate terms as the study of hydrological processes both within and outside the urban environment that are affected by urbanization.

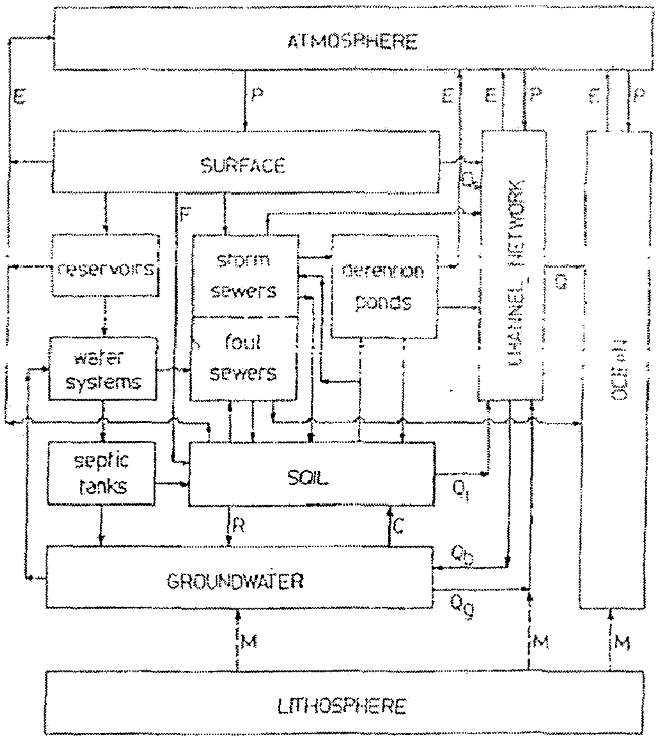


FIG. 1.3. The urban hydrological cycle.

1.2. THE SCOPE OF URBAN HYDROLOGY

Several authors, including (In : Akan et al., 2003) Savini and Kammerer (1961), Leopold (1968), Hall (1973) and Cordery (1976), have described the changes in flow regime which occur when an initially rural catchment area is subject to urbanization. The particular aspects of urbanization which exert the most obvious influence on hydrological processes are the increase in population density and the increase in building density within the urban area. The consequences of such changes are outlined diagrammatically in ***FIG. 1.4.***

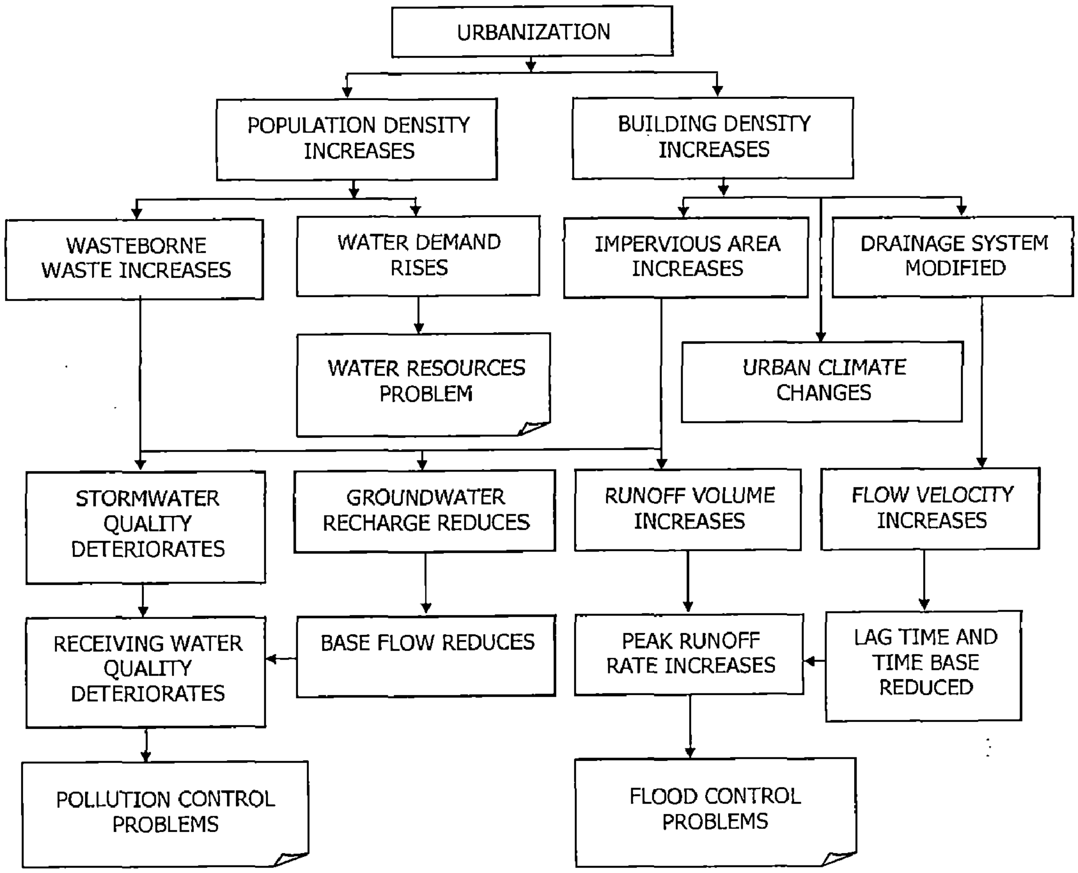


FIG. 1.4. The effects of urbanization on hydrological processes.

As the population increases, water demand begins to rise. This growth in demand is accelerated as standards of living are raised and compounds the problem of developing adequate water resources – the first of the major hydrological problems.

Once the initial stages of urbanization have passed and sewerage systems are installed for both domestic and surface water drainage, the amount of waterborne waste increases in response to the growth in population. However, the resultant water quality changes are intimately linked with the consequences of the increase in building density. As the latter rises, the extent of impervious area also increases, the natural drainage system is modified and the local microclimate changes. Owing to the larger impervious area, a greater proportion of the incident rainfall appears as runoff than was experienced when the catchment was in its rural state. Furthermore, the laying of storm sewers and the realignment and culverting of natural stream channels which takes place during urbanization result in water being transmitted to the drainage network more rapidly. This increase in inflow velocities directly affects the timing of the runoff hydrograph. Since a larger volume of runoff is discharged within a shorter time interval, peak rates of flow inevitably increase, giving rise to the second of the major hydrological problems of flood control.

The inadvertent changes in the microclimate which accompany the growth of urban areas may at first sight appear somewhat irrelevant in comparison to the changes in the hydrological cycle brought about by urbanization. Nevertheless, further consideration of the available evidence, as presented by Landsberg (1981a, b) In: Akan et al., 2003, for example, shows that, since all aspects of climate are affected to some extent by urban development, some attention should be devoted to the possible consequences of such changes in terms of infrastructure design. For example, in drainage design practice, particular importance is attached to the frequency of heavy rainfalls within predetermined durations. Changes in the relationship between rainfall depth, duration and frequency may therefore alter the degree of protection afforded by engineering works subsequent to their design and construction. Possible allowances for such changes are most conveniently treated as a supplementary aspect of the flood control problem.

As ***FIG. 1.4.*** demonstrates, the water quality aspects of the hydrological cycle are affected by both the rise in population and the increase in the extent of the impervious area. Since the volume of runoff becomes larger with the onset of development, the amount of soil moisture recharge is reduced. Consequently, less water is likely to percolate into any aquifer underlying the urban area. Between storm events, the baseflow within the natural drainage system is derived from such subsurface storages. Low flows may therefore be expected to decrease as the urbanization of an area proceeds. Unfortunately, this decrease occurs simultaneously with the increase in the volume of waterborne wastes referred to above and the deterioration in the quality of stormwater runoff as contaminants are washed from streets, roofs and paved areas. The disposal of both solid and waterborne wastes may also have an adverse effect upon groundwater quality. The degradation of the quality of the flows in both the drainage network serving the urban area and the underlying aquifers gives rise to the third of the major hydrological problems - pollution control.

In summary, the process of urbanization may be seen to create three major hydrological problems: the provision of water resources for the urban area that are adequate in both quantity and quality; the prevention of flooding within urban areas; and the disposal of waterborne wastes from urban areas without impairing the quality of local watercourses. Of these three problems, that of water supply forms part of the wider subject of water resources development, and is beyond the scope of this text. Nevertheless, two distinct attitudes to the development of water resources for rapidly growing urban areas may be identified in current practice.

1.3. OBJECTIVES AND SCOPE OF THE PRESENT STUDY

1. To understand the issues involved in the planning of urban drainage.
2. To understand basic theory of an operational model, and related software (Storm Water Management Model) for its potential use in simulation of an existing drainage system and in planning of a new drainage system.
3. Field survey and collection of field data of Sekanak Drainage System to identify subcatchments and existing drainage. Field data is converted to input data as per requirement of SWMM software.
4. To understand how to use SWMM software (input data as per requirement) and applied an various illustrative examples.
5. To understand and derive properties of subcatchments and drainage based on field information and literature for the Sekanak Drainage System site for use as input data in computer based SWMM.
6. Estimation of single event storm and simulation study of storm water drainage in Sekanak Drainage System using SWMM software (Chapter IV) and based on field information.

1.4. ORGANIZATION OF DISSERTATION

Chapter I	:	***Describes Urban Hydrology***, The Definition of Urban Hydrology, The Scope of Urban Hydrology, Objectives and Scope of Study, and Organization of Dissertation.
Chapter II	:	***Gives details about Desk-top Methods for Urban Drainage Design***; The Rational Method for Surface Drainage, Kinematic Time of Concentration Formulas, The Kinematic Rational Method, Storm Sewer Design by The Rational Method, and Calculation of Normal Depth or Uniform Flow Depth.
Chapter III	:	***Explains about Storm Water Management Model-software***; Introduction, SWMM'S Conceptual Model and Overview of Computational Methods.
Chapter IV	:	***Simulation Study of Sekanak Drainage System in Palembang City***; The Study Area, Design Storm, Input Data and Parameters for SWMM.
Chapter V	:	***Results and Discussion***; Design of Open Channel (Calculation of Normal Depth) by Pillai's Method, Computation of Design Storm, Computation of Infiltration, Computation of Runoff at the Outlets of Subwatersheds, Flow through Conduit, Computation of Flood Hydrograph, Computation of Water Surface Profile along the Drainage System, Computation of Flow Velocity in Conduit Flow, Checking of Water Balance and Sensitivity Analysis.
Chapter VI	:	***Summary & Conclusions***.

CHAPTER II

DESK-TOP METHODS FOR URBAN DRAINAGE DESIGN

2.1. THE RATIONAL METHOD FOR SURFACE DRAINAGE

The conventional Rational Method is essentially a peak discharge design method. The underlying principle is that within the context of a specified return period and under constant rate of rainfall, the maximum discharge from a drainage area will occur when the entire area is contributing to runoff. The entire area starts contributing to runoff when rainwater reaches the drainage outlet from the hydrologically most remote point in the drainage area. This time is called the time of concentration. The assumptions of the Rational Method are based on this rationale and can be summarized as follows :

1. The intensity of the design rainfall is constant and uniform.
2. The duration of the design rainfall, t_d, is equal to the time of concentration, T_c of the drainage area being considered.
3. The return period of the peak discharge resulting from a rainstorm is equal to that of the rainstorm.

In addition, the Rational Method assumes that the peak discharge is proportional to the rain intensity, or

$$Q_p = CiA \tag{2.1}$$

where

Q_p	=	peak discharge = design discharge
C	=	dimensionless runoff coefficient
i	=	rate of rainfall
A	=	surface area of the drainage basin

In most applications of the Rational Method, the runoff coefficient is treated merely as a function of the type of the land use or the surface cover. ***Tables 2.1*** and *2.2* present the runoff coefficients that are suggested for 5- to 10-year design-storms. Higher coefficients should be used for less frequent storms having higher return periods (ASCE, 1970). The runoff coefficient can be reasonably increased by 10%,20%, and 25% for 20-,50-, and 100-year rainstorms, respectively.

A key parameter in using the Rational Method is the time of concentration. There are several time of concentration formulae available in the literature. Among the empirical time of concentration formulae the Kirpich formula has found widespread use especially in the Rational Method applications. The Kirpich formula is given as

$$T_c = \frac{0.0078 L^{0.77}}{S^{0.385}} \tag{2.2}$$

where

T_c = time of concentration (min)

L = length of main channel from headwater to outlet (ft)

S = average watershed slope

TABLE 2.1. Runoff Coefficients for Different Land Use Types (source: ASCE. 1970).

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighborhood	0.50 to 0.70
Residential	
Single-family	0.30 to 0.50
Multi-units, detached	0.40 to 0.60
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Other	
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad yard	0.20 to 0.35
Unimproved	0.10 to 0.30

TABLE 2.2. Runoff Coefficients with Respect to Surface Type (source: ASCE. 1970).

Character of Surface	Runoff Coefficients
Pavement	
Asphalt and concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, Sandy Soil	
Flat, 2 percent	0.05 to 0.10
Average, 2 to 7 percent	0.10 to 0.15
Steep, 7 percent	0.15 to 0.20
Lawns, Heavy Soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

This formula was originally developed from runoff data for several rural basins with well-defined channels and steep slopes. Often, the Kirpich formula is used to determine the individual travel times for overland and channel flow segments along the main flow path. In urban watershed applications, it is suggested that T_c be multiplied by 0.4 for overland flow on concrete or asphalt surfaces, and by 0.2 for concrete channels. No adjustment is needed for overland flow on bare soil or flow in roadside ditches.

Example 2.1

A culvert is proposed under a roadway in Norfolk, Virginia. The flow path can be approximated by an overland flow on bare soil and a ditch. For the overland flow segment L = 300 ft and S = 0.03, and for the ditch segment, L = 320 ft and S = 0.001. The runoff coefficient is C = 0.50. We are to determine the 10-year design discharge for the culvert if the drainage area is 7 acres = 304,920 ft^2.

First, we need to calculate the time of concentration for the drainage basin. Using Equation (2.2),

$$T_c\ \text{(overland)} = 0.0078(300)^{0.77}/(0.03)^{0.385} = 2.43\ \text{min}$$

$$T_c\ \text{(ditch)} = 0.0078(320)^{0.77}/(0.001)^{0.385} = 9.46\ \text{min}$$

The combined time of concentration becomes 2.43 + 9.46 = 11.89 ≈ **12 minutes**. From the intensity-duration-return period curve for Norfolk *(FIG. 2.5.)*, for a return period of 10-years and a duration of 12 minutes, the average rainfall intensity is $i = 5.5$ in/hr = 1.27×10^{-4} ft/sec. Finally, from Equation (2.1),

$$Q_p = (0.50)(1.27 \times 10^{-4})(304{,}920) = 19.4 \text{ cfs} = \mathbf{0.55\ m^3/sec.}$$

Example 2.2

A drainage basin in Norfolk, Virginia is made of two distinct subareas. The upper subarea is undeveloped with a surface area of 10 acres = 435,600 ft^2 and a runoff coefficient of 0.20. The lower subarea has a drainage area of 14 acres = 609,840 ft^2 and a runoff coefficient of 0.60. For the combined basin, the time of concentration is 30 minutes, and for the lower subarea alone, the time of concentration is 10 minutes. We are to determine the 10-year peak discharge for this drainage basin.

First the combined drainage basin will be considered. The area weighted runoff coefficient is calculated as

$$C = [(0.20)(10) + (0.60)(14)]/(10 + 14) = 0.43$$

From ***FIG. 2.5***, for T_r = 10 years and $t_d = T_c$ = 30 minutes, we obtain $i = 3.5$ in/hr = $8.10 \text{ X } 10^{-5}$ ft/sec, and

$$Q_p = (0.43)(8.10 \times 10^{-5})(435{,}600 + 609{,}840) = 36.4 \text{ cfs} = \mathbf{1.03\ m^3/sec.}$$

However, in this problem we notice that the upper subarea has a *low* runoff coefficient and a relatively large flow time. In such a case, the rational peak discharge obtained for the lower subarea alone may become greater than that for the combined basin. Considering the lower subarea alone, from ***Fig. 2.5.***, we obtain $i = 6$ in/hr = 1.39×10^{-4} ft/sec for T_r = 10 years and $t_d = T_c$ = 10 minutes. Then, from Equation (2.1),

$$Q_p = (0.60)(1.39 \times 10^{-4})(609{,}840) = 50.9 \text{ cfs} = \mathbf{1.44\ m^3/sec.}$$

Since, **1.44 m^3/sec** > **1.03 m^3/sec**, we should use **1.44 m^3/sec** as the design discharge. This discharge is produced by a rainfall of 10 minutes duration and

6 inches/hour intensity. By excluding the upper basin from the rational formula in calculating ***Qp* =1.44 m^3/sec**, we are assuming that the upper basin has no contribution to the peak discharge under this design-storm. The rationale for this assumption is that it takes 30 minutes for rainwater to reach the outlet from the upper basin while the peak discharge occurs 10 minutes after the storm commences. Still, the Rational Method peak discharge from the lower basin alone is greater than the one calculated considering the entire basin in this particular problem.

This example shows that caution must be practiced in applying the Rational Method to composite basins especially where the downstream areas are developed and upstream areas are undeveloped.

2.2. KINEMATIC TIME OF CONCENTRATION FORMULAS

It is apparent from the preceding section that the time of concentration is a key parameter in Rational Method applications. Most of the formulas used commonly in the practice are empirical, and they have limitations. However, it is possible to derive a series of time of concentration formulas based on the kinematic-wave theory for a variety of basin configurations. Several assumptions are needed to obtain these formulas. First, the time of concentration is assumed to be equal to the time to equilibrium. If both overland and channel flows are involved the time of concentration is assumed to be equal to the sum of equilibrium times of the two components. Finally, the rate of rainfall excess is assumed to be constant and equal to the runoff coefficient times the rate of rainfall.

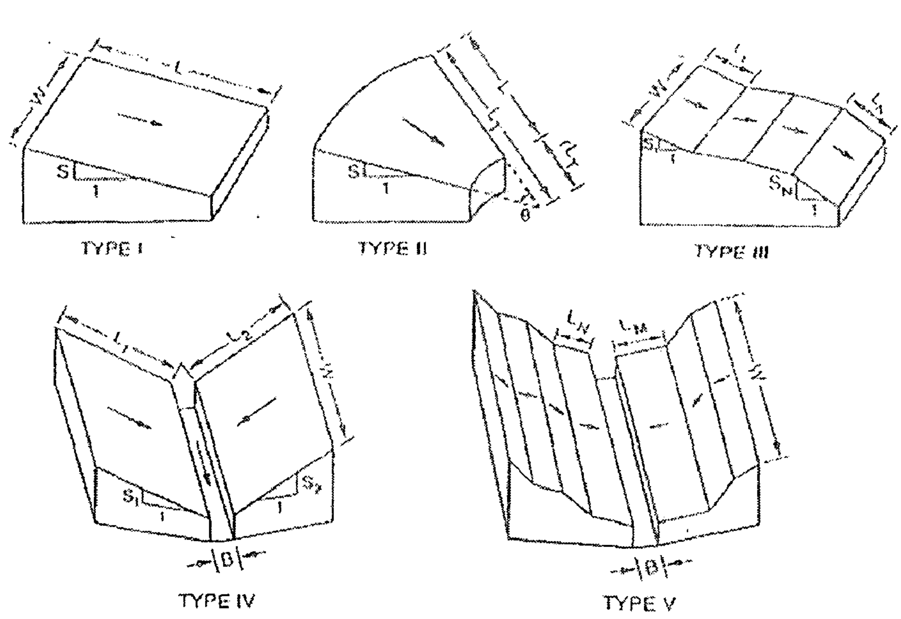


FIG. 2.1. Various drainage basin configurations [source: Akan, A. O. 1985].

The different drainage basin configurations considered are displayed in ***FIG. 2.1.*** Basin type I is a simple rectangular overland flow plane. For such a plane, the expression for time of concentration is

$$T_c = \frac{\left(\dfrac{Ln}{k\sqrt{S}}\right)^{0.6}}{(Ci)^{0.4}} \tag{2.3}$$

where

T_c = time of concentration

L = flow length

n = Manning roughness factor

k = 1.49 $ft^{1/3}$/sec = 1.0 $m^{1/3}$/sec

S = slope

C = runoff coefficient

i = rate of rainfall

Basin type II is a converging surface. An approximate expression is available for the time of concentration of such basins (Overton and Meadows, 1976).

$$T_c = \left[\frac{Ln}{k\sqrt{S}}\right]^{0.6}\left[\frac{(1-r)}{Ci}\right]^{0.4} \tag{2.4}$$

Where $(1 - r)$ is the convergence factor, and r is as defined in ***FIG. 2.1.*** For a cascade of planes, type III, the time of concentration can be expressed as (Overton and Meadows, 1976)

$$T_c = \frac{\sum_{j=1}^{N}\left(\frac{n_j C_j i}{k\sqrt{S_j}}\right)\left(Z_j^{1.6} - Z_{j-1}^{1.6}\right)}{\sum_{j=1}^{N} C_j i\left(Z_j - Z_{j-1}\right)} \tag{2.5}$$

where

j = index representing a plane in the cascade

N = total number of planes in the cascade

and

$$Z_j = \sum_{m=1}^{j} L_m \tag{2.6}$$

Basin type IV represents a rectangular channel receiving runoff from two overland flow planes. Assuming the channel is wide, the time of concentration formula for such a basin becomes (Akan, 1985) :

$$T_c = \frac{1}{i^{0.4}}\left[\frac{1}{C_0^{0.4}}\left(\frac{L_0 N_0}{k\sqrt{S_0}}\right)^{0.6} + \frac{B^{0.4}}{\left(BC_c + L_1C_1 + L_2C_2\right)^{0.4}}\left(\frac{Wn_c}{k\sqrt{S_c}}\right)^{0.6}\right] \tag{2.7}$$

where

B = width of the channel

W = length of the channel = width of overland flow planes

In Equation (2.7) the subscripts 1,2, and c stand for the first and second overland flow planes and the channel, respectively. Subscripts o refers to the overland plane that has the larger equilibrium time.

If a channel receives overland flow from two composite catchments made of cascades of M and N rectangular planes, such as basin type V, the expression for the time of concentration is (Akan, 1985) :

$$T_c = \frac{1}{i^{0.4}}\left\{\frac{\sum_{j=1}^{N}\left(\frac{n_j C_j}{k\sqrt{S_j}}\right)^{0.6}\left(Z_j^{1.6} - Z_{j-1}^{1.6}\right)}{\sum_{j=1}^{N} C_j\left(Z_j - Z_{j-1}\right)} + \frac{B^{0.4}\left(\frac{Wn_c}{k\sqrt{S_c}}\right)^{0.6}}{\left[BC_c + \sum_{j=1}^{N} C_j L_j + \sum_{m=1}^{M} C_m L_m\right]^{0.4}}\right\} \tag{2.8}$$

In Equation (2.8), N refers to the number of planes in the cascade that has the greater equilibrium time.

It should be noted that Equations (2.3) to (2.8) are dimensionally homogeneous, and they can be used with a consistent unit system. Only the value of the constant $k = 1.0\ m^{1/3}/sec = 1.49\ ft^{1/3}/sec$ depends on the unit system used.

2.3. THE KINEMATIC-RATIONAL METHODS

Although the kinematic time of concentration formulas have a theoretical basis, they have not found widespread use in the Rational Method applications. This is probably because even for the simplest configuration, basin type I, the time of concentration is a function of the rate of rainfall i [Equation (2.3)]. In other words, we can not determine the time of concentration without knowing the design rainfall intensity. On the other hand, we need to know the time of concentration to find the design-storm intensity from the intensity-duration-return period curves as explained in Examples 2.1 and 2.2. Therefore, a trial-and-error approach is needed to use the kinematic equations in the conventional Rational Method.

However, we can eliminate the trial-and-error procedure by adopting a mathematical expression from the intensity-duration relationship. The possible relationships are

$$i = \frac{X}{t_d^Y} \tag{2.9}$$

and

$$i = \frac{a}{t_d + b} \tag{2.10}$$

For instance, if Equation (2.9) is adopted, combining Equations (2.1), (2.9) and Equations (2.3) to (2.8) and setting $t_d = T_c$

$$Q_p = \eta \mu^f X^p \tag{2.11}$$

where

$$f = \frac{Y}{0.4Y - 1} \tag{2.12}$$

and

$$p = \frac{1}{1 - 0.4Y} \tag{2.13}$$

The parameters η and μ are evaluated differently as shown in ***Table 2.3*** for the different basin configurations. Because Equation (2.9) is not dimensionally homogeneous, we need to pay attention to the units of the variables in using Equations (2.11) to (2.13) and Table 2.3. If i is in inches per hour and t_d is in minutes in Equation (2.9) and all the lengths are in feet in Table 2.3, then k_o = **43,200** and k_1 = **0.94**, and ***Qp*** **will be in cfs.** If i is in millimeters/hour and t_d is in minutes in Equation (2.9), and all the lengths are in meters in ***Table 2.3***, then k_o = **3,600,000** and k_1 = **6.99**, and ***Qp*** **will be in m^3/sec.**

A similar procedure is available if the intensity-duration relationship is given in the form of Equation (2.10). In this procedure, first the parameters f_1 and f_2 are determined using ***Table 2.4*** for different drainage basin configurations. Next, the parameter Q_o is obtained from ***FIG. 2.2.*** Finally, from the expressions given for Q_o in Table 2.4, the design discharge, Q_p, is calculated.

If i is in inches/hour and t_d is in minutes in Equation (2.10) and all the lengths in ***Table 2.4*** are in feet, then k_o = **43,200** and k_1 = **0.94**, and Q_p **will be in cfs**. If i is in millimeters/hour and t_d is in minutes in Equation (2.10), and all the lengths are in meters in Table 2.4, then k_o = **3,600,000** and k_1 = **6.99**, and Q_p **will be in m^3/sec**

Table 2.3. Kinematic-Rational Method Parameters for Different Configurations [Source:Akan,1985].

Basin Type	η	μ
I	$\frac{CWL}{k_0}$	$\frac{k_1 (Ln)^{0.6}}{C^{0.4} S^{0.3}}$
II	$\frac{C\theta L^2 (1+r)}{2k_0 (1-r)}$	$\frac{k_1 (Ln)^{0.6} (1-r)^{0.4}}{C^{0.4} S^{0.3}}$
III	$\frac{W \sum_{j=1}^{N} C_j L_j}{k_0}$	$\frac{k_1 \sum_{j=1}^{N} \frac{(n_j C_j)^{0.6}}{S_j^{0.3}} \left(Z_j^{1.6} - Z_{j-1}^{1.6}\right)}{\sum_{j=1}^{N} C_j (Z_j - Z_{j-1})}$
IV	$\frac{W(C_c B + C_1 L_1 + C_2 L_2)}{k_0}$	$k_1 \left\{ \frac{(L_0 n_0)^{0.6}}{C_0^{0.4} S_0^{0.3}} + \frac{B^{0.4} (W n_c)^{0.6}}{S_c^{0.3} (C_c B + C_1 L_1 + C_2 L_2)^{0.4}} \right\}$
V	$\frac{W\left(C_c B + \sum_{j=1}^{N} C_j L_j + \sum_{m=1}^{M} C_m L_m\right)}{k_0}$	$k_1 \left\{ \frac{\sum_{j=1}^{N} \frac{(n_j C_j)^{0.6}}{S_j^{0.3}} \left(Z_j^{1.6} - Z_{j-1}^{1.6}\right)}{\sum_{j=1}^{N} C_j (Z_j - Z_{j-1})} + \frac{B^{0.4} (W n_c)^{0.6}}{S_c^{0.3} \left(C_c B + \sum_{j=1}^{N} C_j L_j + \sum_{m=1}^{M} C_m L_m\right)^{0.4}} \right\}$

Table 2.4. Definitions of Q_0, f_1 And f_2 For Different Basin Configurations.

Basin Type	Q_0	f_1	f_2
I	$\dfrac{k_0 Q_p}{CWL}$	$\dfrac{k_1 (Ln)^{0.6}}{bC^{0.4} S^{0.3}}$	$\dfrac{a}{b}$
II	$\dfrac{2k_0(1-r)}{\theta L^2 (1+r) C} Q_p$	$\dfrac{k_1 (Ln)^{0.6} (1-r)^{0.4}}{bC^{0.4} S^{0.3}}$	$\dfrac{a}{b}$
III	$\dfrac{k_0 Q_p}{W \sum_{j=1}^{N} C_j L_j}$	$\dfrac{k_1 \sum_{j=1}^{N} \dfrac{(n_j C_j)^{0.6}}{S_j^{0.3}} (Z_j^{1.6} - Z_{j-1}^{1.6})}{\sum_{j=1}^{N} C_j (Z_j - Z_{j-1})}$	$\dfrac{a}{b}$
IV	$\dfrac{k_0 Q_p}{W(C_c B + C_1 L_1 + C_2 L_2)}$	$\dfrac{k_1}{b} \left\{ \dfrac{(L_0 n_0)^{0.6}}{C_0 S_0^{0.4}} + \dfrac{B^{0.4} (W n_c)^{0.6}}{S_c^{0.3} (C_c B + C_1 L_1 + C_2 L_2)^{0.4}} \right\}$	$\dfrac{a}{b}$
V	$\dfrac{k_0 Q_p}{W \left(C_c B + \sum_{j=1}^{N} C_j L_j + \sum_{m=1}^{M} C_m L_m \right)}$	$\dfrac{k_1}{b} \left\{ \dfrac{\sum_{j=1}^{N} \dfrac{(n_j C_j)^{0.6}}{S_j^{0.3}} (Z_j^{1.6} - Z_{j-1}^{1.6})}{\sum_{j=1}^{N} C_j (Z_j - Z_{j-1}^{1.6})} + \dfrac{B^{0.4} (W n_c)^{0.6}}{S_c^{0.3} \left(C_c B + \sum_{j=1}^{N} C_j L_j + \sum_{m=1}^{M} C_m L_m \right)^{0.4}} \right\}$	$\dfrac{a}{b}$

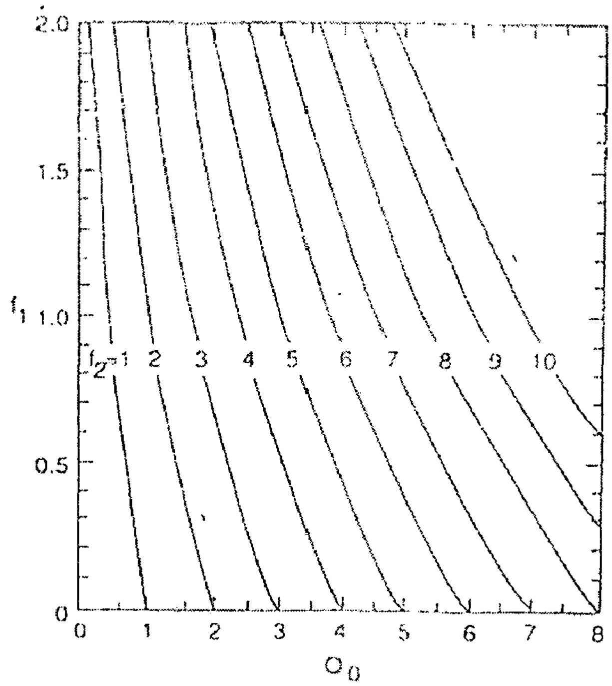


FIG. 2.2. Kinematic-Rational Method chart.

Example 2.3.

A drainage structure will be designed for a 10-year storm. The intensity duration relationship for this return period is $i = 7/t_d^{0.5}$ in which i is in inches per hour and t_d is in minutes. The drainage basin may be approximated by a converging surface with L = 2800 ft, n = 0.10, S = 0.04, C = 0.40, r = 0.20 and θ = 0.80 radians. We are to determine the design discharge.

From the intensity-duration relationship, X = 7 and Y = 0.5. Then, from Equations (2.12) and (2.13), respectively,

$$f = 0.5/[(0.4)(0.5)] -1.0] = -0.625$$

$$p = 1.0/[1 -(0.4)(0.5)] = 1.25$$

Next. using ***Table 2.3***,

$$\eta = \frac{(0.4)(0.8)(2800)^2(1+0.2)}{(2)(43{,}200)(1-0.2)} = 43.56$$

$$\mu = \frac{(0.94)[(2800)(0.1)]^{0.6}(1-0.2)^{0.4}}{(0.4)^{0.4}(0.04)^{0.3}} = 95.77$$

and finally from Equation (2.11),

$$Q_p = 43.56(95.77)^{-0.625}(7.0)^{1.25} = 28.65 \text{ cfs} = \mathbf{0.81\ m^3/sec.}$$

Example 2.4

A drainage structure will be designed for a 25-year storm. The intensity-duration relationship for this return period is $i = 240/(t_d + 40)$ in which i is in inches/hr and t_d is in minutes. The drainage basin can be represented by a cascade of two rectangular planes. For the upper plane, L_1 = 200 ft. S_1 = 0.01, n_1 = 0.1, and C_1 = 0.40. For the lower plane, L_2 = 300 ft, S_2 = 0.004, n_2 = 0.05, and C_2 = 0.60. The width of the catchment is W = 400 ft. We are to determine the design discharge.

From the problem statement a = 240 and b = 40. Also, the catchment is of type III as shown in ***FIG. 2.1.*** Using Equation (2.6),

$$Z_1 = L_1 = 200 \text{ ft}$$

and

$$Z_2 = L_1 + L_2 = 200 + 300 = 500 \text{ ft}$$

Next from Table 2.4.

$$f_1 = 0.94\left\{\frac{(0.1)^{0.6}(0.4)^{0.6}}{(0.01)^{0.3}}(200)^{1.6} + \left[\frac{(0.05)^{0.6}(0.60)^{0.6}}{(0.004)^{0.3}}\right]\left[(500)^{1.6} - (200)^{1.6}\right]\right\}$$

$$\div\{40[0.40(200) + 0.60(500 - 200)]\} = 1.18$$

$$f_2 = \frac{240}{40} = 6$$

Then, from ***FIG. 2.2.***, with f_1 = 1.18 and $f2$ = 6, we obtain Q_0 = 3.5. Finally, using the expression given for Q_0 in ***Table 2.4. for basin type III***,

$$Q_p = \frac{Q_0 W \Sigma C_j L_j}{k_0} = \frac{(3.5)(400)[0.40)(200) + (0.60)(300)}{43{,}200} = 8.43 \text{ cfs} = \mathbf{0.24\ m^3/sec}$$

2.4. STORM SEWER DESIGN BY THE RATIONAL METHOD

The Rational Method is probably the most commonly used method for storm sewer design. The assumptions of the Rational Method for surface drainage design are involved also in the design of storm sewers. Briefly, the Rational Method assumes that the return period of a peak discharge is equal to that of the rainfall that produces it, the duration of the design rainfall is equal to the time of concentration of the drainage system above the design point, and the rate of the design rainfall is constant over the duration. The use of the rational method is usually limited to urban areas smaller than 13 square kilometers.

2.4.1. HYDRAULICS OF STORM SEWERS

Flow in a storm sewer is normally nonsteady and nonuniform. However, for practical purposes, the sewer flow is usually assumed to be steady and uniform at the peak discharge. In a typical storm sewer design situation, given the design discharge and the sewer slope, we would need to determine the sewer diameter. Using the Manning formula,

$$D_r = \left[\frac{nQ_p}{0.31k\sqrt{S_0}} \right]^{3/8} \tag{2.14}$$

where

n = Manning roughness factor

Q_p = design discharge

k = 1.49 $ft^{1/3}/sec$ = 1.0 $m^{1/3}/sec$

S_0 = slope of the sewer

D_r = minimum required diameter to accommodate Q_p

The actual diameter selected for the sewer will be the next standard pipe size larger than D_r. This will ensure that the sewer will flow partially full at the design discharge. Typical values for the Manning roughness factor vary from 0.012 to 0.016 for storm sewers.

The average sewer flow velocity is also needed in the Rational Method. A storm sewer normally flows partially full. Strictly speaking the flow velocity should be calculated by dividing the discharge by the actual flow area. However, simpler approaches are used in the practice to estimate the velocity. If full flow condition is assumed,

$$V = \frac{4Q_p}{\pi D^2} \tag{2.15}$$

where

V = average velocity

D = sewer diameter

If we assume that the sewer is nearly full but the flow still has a free surface,

$$V = \frac{kD^{2/3}S_0^{1/2}}{2.52n} \tag{2.16}$$

2.4.2. TIME OF CONCENTRATION FOR STORM SEWER DESIGN

In a typical urban storm drainage system, the stormwater first flows over the ground to a surface inlet. The time required for stormwater to reach an inlet from the hydrologically most remote point is called the *inlet time.* Then it discharges into the sewer system and flows in the downstream direction under the effect of gravity. Considering such a flow path, the time of concentration for a sewer can be written as

$$T_c = t_o + t_f \tag{2.17}$$

where

T_c = time of concentration

t_o = inlet time

t_f = flow time in the sewers upstream of the design point

If there are N upstream sewers along the flow path

$$t_f = \sum_{j=1}^{N} \frac{L_j}{V_j} \tag{2.18}$$

where

L_j = length of *j-th* sewer

V_i = average velocity in the *j-th* sewer

The *inlet time* may include overland, gutter, and roadside ditch flow times, and it can be calculated using the methods discussed in Section 2.2. However, in most applications a constant value is assumed for the inlet time. In densely developed areas where impervious surfaces are directly connected to the drainage system, an inlet time of 5 minutes is used. In well developed districts with relatively flat slopes, an inlet time of 10 to 15 minutes is common. In flat residential areas with widely spaced street inlets, inlet times of 20 to 30 minutes are customary (ASCE, 1970).

2.4.3. STORM SEWER DESIGN DISCHARGE

To determine the design discharge for a storm sewer, first the time of concentration is determined using Equation (2.17). Next, for the specified return period, the intensity of the design rainfall is obtained from the intensity-duration-return period curves assuming the duration equals the time of concentration. Then the design discharge is found from the rational formula

$$Q_p = i\sum_{j=1}^{M} C_j A_j \qquad (2.19)$$

where

i = design rainfall intensity

M = number of subcatchments above the sewer

C_j = runoff coefficient of subcatchment j

A_j = drainage area of subcatchment j

Equation (2.19) can be used in conjunction with any consistent unit system. Alternatively, i can be in inches/hour, A in acres, and Q_p in cfs.

It is evident from this procedure that the time of concentration and the design rainfall intensity will differ from sewer to sewer within the same storm sewer system. In other words, different design-storms are used to size each sewer. Therefore, we can size different components of the same sewer system for different return periods depending on the importance of each component.

We must use Equation (2.19) cautiously. In a complex sewer system stormwater can reach a particular sewer from several different paths. Under normal conditions, the path with the largest time of concentration will determine the design discharge. In that event, all the subcatchments above the design point will be included in M of Equation (2.19). However, paths other than the longest path can also be critical in a composite basin where the runoff coefficients vary significantly. Therefore, it is good practice to check all the possible paths. When a particular path is considered, the parameter M in Equation (2.19) includes all the subcatchments along that particular path plus those along the other paths that have a shorter time of concentration.

Example 2.5

A simple storm drainage system is considered as shown in ***Fig. 2.3.*** The arrows indicate the drainage pattern. The subcatchment characteristics, and sewer lengths and slopes are given in Table 2.5. A constant inlet time of 15minutes is adopted. We are to determine the sewer diameters for a return period of 10 years using the intensity-duration-return period curves shown in ***FIG. 2.5.***

We can start with either sewer AB or CB. Sewer BD can be considered only after AB and CB are sized. There is only one path for stormwater to reach sewer AB. Along this path, stormwater from subcatchments 1 and 2 discharge directly to manhole A. Since there are no pipes further upstream, $t_f = 0$, and therefore $T_c = t_o$ = 15 minutes. From ***FIG. 2.5***, for T_r = 10 years and $t_d = T_c$ = 15 minutes, $\boldsymbol{i}$ = 5.0 in/hr = **127 mm/hr**. Then from Equation *(2.19)*, $\boldsymbol{Q_p}$ = 12.5 cfs = **0.35 m^3/sec,** and from Equation (2.14), $\boldsymbol{D_r}$ = 1.41 ft = **0.43 m**. The next larger standard size is selected as the diameter of sewer AB, that is $\boldsymbol{D}$ = 1.5 ft = **0.46 m**. Then, from Equation (2.16), V = 8.43 fps = **2.57 m/sec.** This represents a flow time.

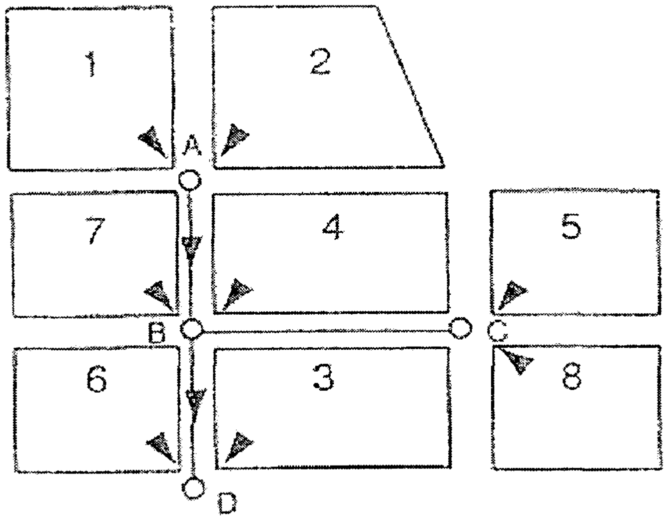


FIG. 2.3. Storm drainage system for Example 2.5.

Table 2.5. Basic Data For Example Basin

Subcatchment	Area (acres)	Runoff Coefficient	CA (Acres)
1	1.5	0.6	0.9
2	2.0	0.8	1.6
3	1.5	0.8	1.2
4	2.0	0.4	0.8
5	1.5	0.6	0.9
6	1.8	0.5	0.9
7	2.0	0.7	1.4
8	1.6	0.5	0.8

Sewer	Roughness Factor	Length (ft)	Upstream Invert Elevation (ft)	Downstream Invert Elevation (ft)	Slope
AB	0.013	200	108.0	104.0	0.020
CB	0.013	400	105.6	104.0	0.004
BD	0.013	180	104.0	99.5	0.025

of L/V = 24 seconds = 0.40 minutes from point A to B along the sewer AB. The calculations are summarized in ***Table 2.6.*** Sewer CB is sized in a similar manner as shown in ***Table 2.6.***

For pipe BD, there are three paths possible: surface runoff to manhole B, surface runoff to A then sewer flow to B, and surface runoff to C then sewer flow to B. The subcatchments along these paths are listed in column 3 of ***Table 2.6.*** For the first path, t_f = 0 and therefore T_c = t_o = 15 minutes. For the second path, t_f = flow time from A to B = 0.40 minutes. Therefore, T_c = 15.0 + 0.4 = 15.4 minutes. For the third path, t_f= 1.60 minutes, and T_c = 15.0 + 1.6 = 16.6 minutes. The time of concentration of the second path is greater than that of the first path. Thus to calculate the peak discharge for the second path the subcatchments along both the first and the second paths should be considered. In other words if a rainstorm that has a duration of 15.4 minutes occurs, the subcatchments 1, 2, 4 and 7 will contribute to the peak discharge, since for stormwater to reach point B it takes 15 minutes from subcatchments 1 and 2 and 15.4 minutes from 4 and 7. Because path 3 has the greatest time of concentration, all the subcatchments above point B, namely 1, 2, 4, 5, 7 and 8, should be included for this path. Using Equation (2.19), $\boldsymbol{Q_p}$ is obtained as being **11.0, 23.3 and 30.7 cfs** or **0.31, 0.66, 0.87 m^3/sec**, respectively, for the three paths. Obviously, the largest Q_p must be chosen as the design discharge. In this case, the largest value is 30.7 cfs, and therefore, sewer BD is sized using $\boldsymbol{Q_p}$ = 30.7 cfs = **0.87 m^3/sec**.

Table 2.6. Sewer Calculations For Example 2.5. (In British Standard and International Standard)

Sewer	Path	Sub-catchment on Path	t_0 (min)	t_f (min)	T_c (min)	i (in/hr)	Sub-catchment for Equation (2.19)	$\sum CA$	Q_p (cfs)	S_0 (ft)	L (ft)	n	D_r (ft)	D (ft)	V (fps)	L/V (min)
AB	A	1,2	15.0	0	15.0	5.0	1,2	2.5	12.5	0.020	200	0.013	1.41	1.5	8.43	0.40
CB	C	5,8	15.0	0	15.0	5.0	5,8	1.7	8.5	0.004	400	0.013	1.65	1.75	4.18	1.60
BD	B	4,7	15.0	0	15.0	5.0	4,7	2.2	11.0							
	AB	1,2	15.0	0.4	15.4	4.95	1,2,4,7	4.7	23.3							
	CB	5,8	15.0	1.6	16.6	4.80	1,2,4,7,5,8	6.4	30.7	0.025	180	0.013	1.89	2.0	11.41	0.26
Sewer	Path	Sub-catchment on Path	t_0 (min)	t_f (min)	T_c (min)	i (mm/hr)	Sub-catchment for Equation (2.19)	$\sum CA$	Q_p (cms)	S_0 (m)	L (m)	n	D_r (m)	D (m)	V (mps)	L/V (min)
AB	A	1,2	15.0	0	15.0	127	1,2	2.5	0.354	0.0061	60.96	0.013	0.43	0.46	2.57	0.40
CB	C	5,8	15.0	0	15.0	127	5,8	1.7	0.241	0.0012	121.92	0.013	0.50	0.54	1.27	1.60
BD	B	4,7	15.0	0	15.0	127	4,7	2.2	0.312							
	AB	1,2	15.0	0.4	15.4	125.73	1,2,4,7	4.7	0.659							
	CB	5,8	15.0	1.6	16.6	121.92	1,2,4,7,5,8	6.4	0.869	0.0076	54.86	0.013	0.57	0.61	3.47	0.26

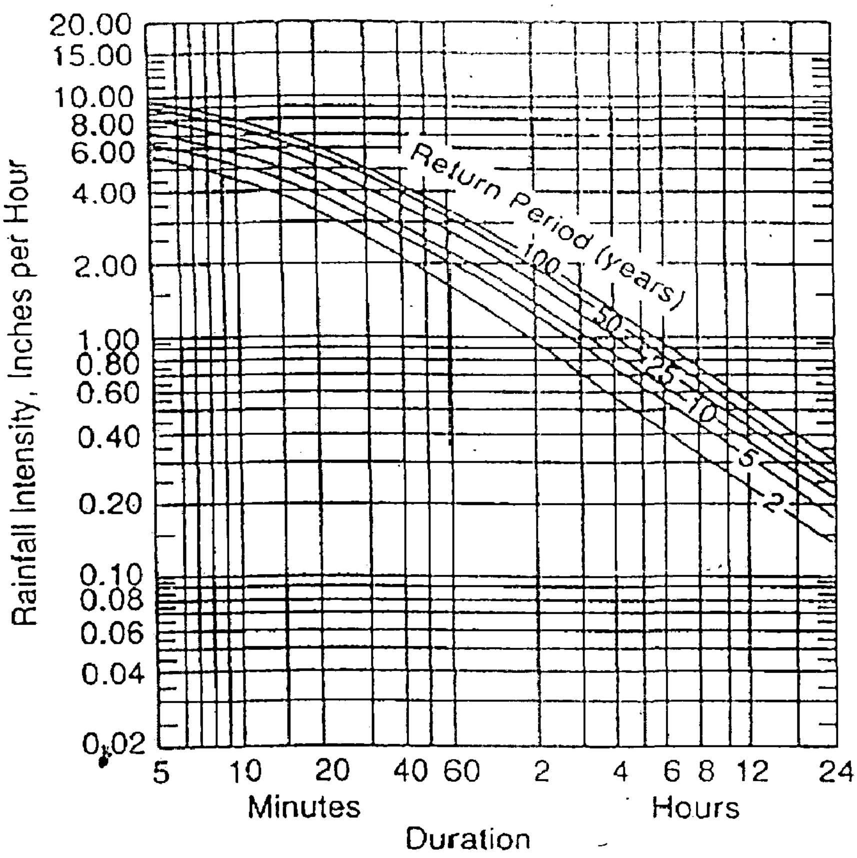


FIG. 2.4. IDF Curves for Norfolk, Virginia.

2.5. CALCULATION OF NORMAL DEPTH OR UNIFORM FLOW DEPTH

Given a prismatic channel of specified geometry and bed slope, one can find a depth of flow at which uniform flow is possible for a given discharge Q. It is known as the normal depth *yn*. For uniform flow, by Manning's formula,

$$Q = \frac{A}{N} R^{2/3} S_0^{\ 1/2}; \text{ hence } \frac{Q_n}{S_0^{\ 1/2}} = AR^{2/3}$$

In a given problem $\frac{Q_n}{S_0^{\ 1/2}}$ reduces to a constant, and a value of y is found which gives the value of $AR^{2/3} = \frac{Q_n}{S_0^{\ 1/2}}$.

It can be done by a trial and error procedure. A value of y_n can be assumed intially and $AR^{2/3}$ can be worked out. y_n is modified until the corrected value of $AR^{2/3}$ is equal to $\frac{Q_n}{S_0^{\ 1/2}}$.

Easy Iterative Procedures

(i) Rectangular channels

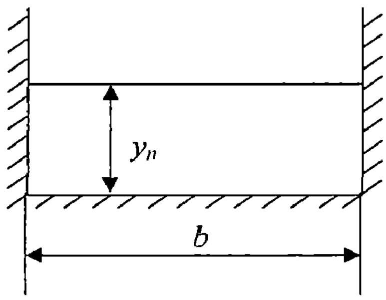


FIG. 2.5. Cross-section of a rectangular channel.

Let b the width of the rectangular channel (*FIG. 2.5*). A = b y_n *and* $P = b+2\,y_n$. From Manning's formula,

$$\frac{Q_n}{S_0^{\,1/2}} = AR^{2/3} = by_n\left(\frac{by_n}{b+2y_n}\right)^{2/3}$$

Pillai [(9)] suggests an iterative scheme for easy calculation of y_n, using a hand calculator.

$$y_n = \frac{Q_n}{b^{5/3}S_0^{\,1/2}}\,x\left(2+\frac{b}{y'_n}\right)^{2/3},$$

Where y'_n is an assumed approximate solution and y_n, a better solution.

(ii) Trapezoidal channels

Most man-made channels are trapezoidal. A typical section is shown in *FIG. 2.6.* A side slope *m*:1 (*m* horizontal to 1 vertical) is provided. The value of *m* can vary, usually between 1 to 2. Let Q be the discharge, S_0 the bed slope, *n* the coefficient of rugosity and y_n the normal depth of flow.

$A = (b + my_n)y_n.$

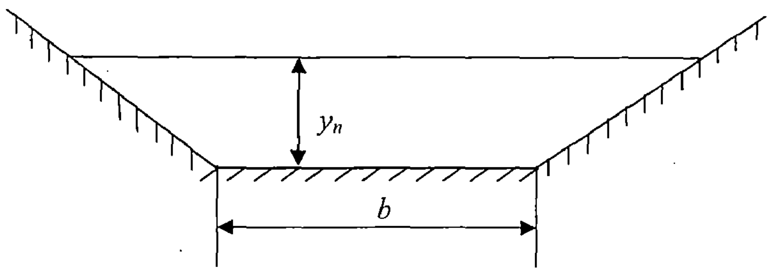


FIG. 2.6. Cross-section of a trapezoidal channel.

$P = b + 2\sqrt{1+m^2}\,y_n$.

$$Q = AV = A\frac{1}{n}R^{2/3}S_0^{\ 1/2} = \frac{A}{n}\left(\frac{A}{P}\right)^{2/3} S_0^{\ 1/2}$$

$$= y_n(b+my_n)\frac{[y_n(b+my_n)]^{2/3}}{(b+2\sqrt{1+m^2}\,y_n)^{2/3}}S_0^{\ 1/2}$$

Hence, $\dfrac{Q_n}{S_0^{\ 1/2}} = y_n^{\ 5/3}\dfrac{(b+my_n)^{5/3}}{(b+2\sqrt{1+m^2}\,y_n)^{2/3}}$.

An iteration solution is given by Pillai [9] as

$$y_n \approx \left(\frac{Q_n}{S_0^{\ 1/2}}\right)^{0.6}\frac{(b+2\sqrt{1+m^2}\,y'_n)^{0.4}}{(b+my'_n)},$$

Where, y'_n is an assumed solution and y_n a better approximation.

CHAPTER III

STORMWATER MANAGEMENT MODELS

3.1 INTRODUCTION

The EPA Storm Water Management Model (SWMM) is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of subcatchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each subcatchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps.

SWMM was first developed in 1971 and has undergone several major upgrades since then. It continues to be widely used throughout the world for planning, analysis and design related to storm water runoff, combined sewers, sanitary sewers, and other drainage system in urban areas, with many applications in non-urban areas as well. The current edition, Version 5, is a complete re-write of the previous release. Running under Windows, SWMM 5 provides an integrated environment for editing study area input data, running hydraulic, hydraulic and water quality simulations, and viewing the results in a variety of formats. These include color-coded drainage area and conveyance system maps, time series graphs and tables, profile plots, and statistical frequency analysis.

This latest re-write of SWMM was produced by the Water Supply and Water Resources Division of the U.S. Environmental Protection Agency's National Risk Management Research Laboratory with assistance from the consulting firm of CDM, Inc.

3.1.1 Modeling Capabilities

SWMM accounts for various hydrologic processes that produce runoff from urban areas. These include :

- time-varying rainfall
- evaporation of standing surface water
- snow accumulation and melting
- rainfall interception from depression storage
- infiltration of rainfall into unsaturated soil layers
- percolation of infiltrated water into groundwater layers
- interflow between groundwater and the drainage system
- nonlinear reservoir routing of overflow flow.

Spatial variability in all of these processes is achieved by dividing a study area into a collection of smaller, homogeneous subcatchment area, each containing its own fraction of pervious and impervious sub-areas. Overland flow can be routed between sub-areas, between subcatchment, or between entry points of a drainage system.

SWMM also contains a flexible set of hydraulic modeling capabilities used to route runoff and external inflows through the drainage system network of pipes, channels, storage/treatment units and diversion structures. These include the ability to :

- handle network of unlimited size
- use a wide variety of standard closed and open conduit shapes as well as natural channels.
- model special elements such as storage/treatment units, flow dividers, pump, weirs, and orifices.
- apply external flows and water quality inputs from surface runoff, groundwater interflow, rainfall-dependent infiltration/inflow, dry weather sanitary flow, and user-defined inflows.
- utilize either kinematic wave or full dynamic wave flow routing methods

- model various flow regimes, such as backwater, surcharging, reverse flow, and surface ponding.
- apply user-defined dynamic control rules to simulate the operation of pumps, orifice openings, and weir crest levels.

In addition to modeling the generation and transport of runoff flows, SWMM can also estimate the production of pollutant loads associated with this runoff. The following processes can be modeled for any number of user-defined water quality constituents :

- dry-weather pollutant buildup over different land uses.
- pollutant washoff from specific land uses during storm events
- direct contribution of rainfall deposition
- reduction in dry-weather buildup due to street cleaning
- reduction in washoff load due to BMPs
- entry of dry weather sanitary flows and user-specified external inflows at any point in the drainage system
- routing of water quality constituents through the drainage system
- reduction in constituent concentration though treatment in storage units or by natural processes in pipes and channels.

3.1.2 Potential Applications of SWMM

Since its inception, SWMM has been used in thousands of sewer and stormwater studies throughout the world. Typical applications include :

- design and sizing of drainage system components for flood control
- sizing of detention facilities and their appurtenances for flood control and water quality protection
- flood plain mapping of natural channel system
- designing control strategies for minimizing combined sewer overflows
- evaluating the impact of outflow and infiltration on sanitary sewer overflows

- generating non-point source pollutant loadings for waste load allocation studies
- evaluating the effectiveness of BMPs for reducing wet weather pollutant loadings.

3.2 SWMM's CONCEPTUAL MODEL

SWMM conceptualizes a drainage system as a series of water and material flows between several major environmental compartments. These compartments and the SWMM objects they contain include:

- The Atmosphere compartment, from which precipitation falls and pollutants are deposited onto the land surface compartment. SWMM uses **Raingage** objects to represent rainfall inputs to the system.
- The Land Surface compartment, which is represented through one or more **Subcatchment** objects. It receives precipitation from the Atmospheric compartment in the form of rain or snow; it sends outflow in the form of infiltration to the Groundwater compartment and also as surface runoff and pollutant loadings to the Transport compartment.
- The Groundwater compartment receives infiltration from the Land Surface compartment and transfers a portion of this inflow to the Transport compartment. This compartment is modeled using **Aquifer** objects.
- The Transport compartment contains a network of conveyance elements (channels, pipes, pumps, and regulators) and storage/treatment units that transport water to outfalls or to treatment facilities. Inflows to this compartment can come from surface runoff, groundwater interflow, sanitary dry weather flow, or from user-defined hydrographs. The components of the Transport compartment are modeled with **Node** and **Link** objects

Not all compartments need appear in a particular SWMM model. For example, one could model just the transport compartment, using pre-defined hydrographs as inputs.

FIG. 3.1. depicts how a collection of SWMM's visual objects might be arranged together to represent a stormwater drainage system. These objects can be displayed on a map in the SWMM workspace.

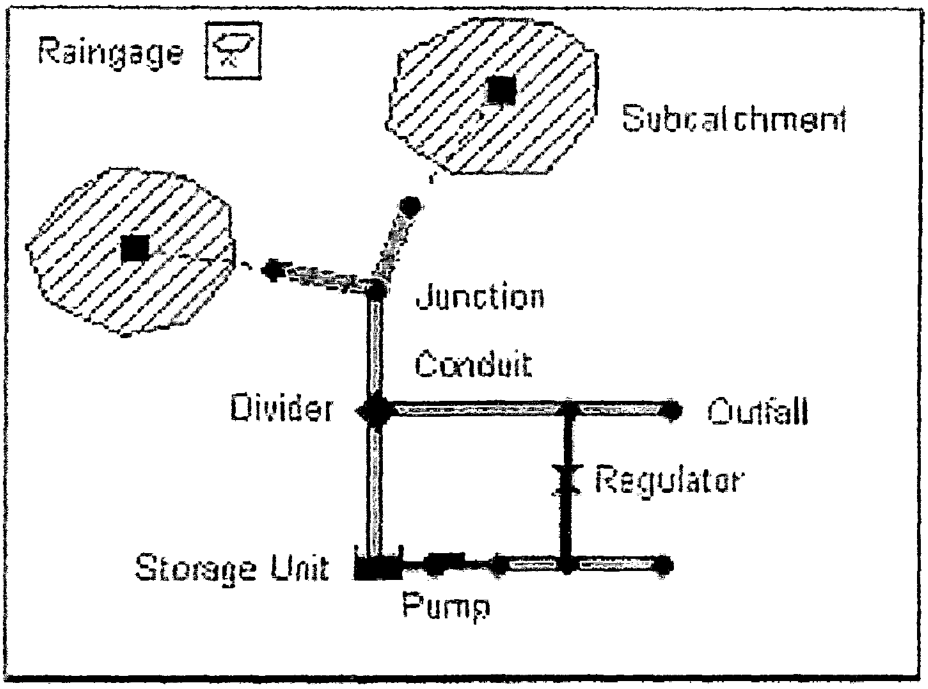


FIG. 3.1. Example of physical objects used to model a drainage system.

3.2.1 Raingages

Raingages supply precipitation data for one or more subcatchment areas in a study region. The rainfall data can be either a user-defined time series or come from an external file. Several different popular rainfall file formats currently in use are supported, as well as a standard user-defined format.

The principal input properties of raingages include:

- rainfall data type (e.g., intensity, volume, or cumulative volume)
- recording time interval (e.g., hourly, 15-minute, etc.)
- source of rainfall data (input time series or external file)
- name of rainfall data source

3.2.2 Subcatchments

Subcatchments are hydrologic units of land whose topography and drainage system elements direct surface runoff to a single discharge point. The user is responsible for dividing a study area into an appropriate number of subcatchments, and for identifying the outlet point of each subcatchment. Discharge outlet points can be either nodes of the drainage system or other subcatchments.

Subcatchments can be divided into pervious and impervious subareas. Surface runoff can infiltrate into the upper soil zone of the pervious subarea, but not through the impervious subarea. Impervious areas are themselves divided into two subareas - one that contains depression storage and another that does not. Runoff flow from one subarea in a subcatchment can be routed to the other subarea, or both subareas can drain to the subcatchment outlet.

Infiltration of rainfall from the pervious area of a subcatchment into the unsaturated upper soil zone can be described using three different models:

- Horton infiltration
- Green-Ampt infiltration
- SCS Curve Number infiltration

To model the accumulation, re-distribution, and melting of precipitation that falls as snow on a subcatchment, it must be assigned a Snow Pack object. To model groundwater flow between an aquifer underneath the subcatchment and a node of the drainage system, the subcatchment must be assigned a set of Groundwater parameters. Pollutant buildup and washoff from subcatchments are associated with the Land Uses assigned to the subcatchment.

The other principal input parameters for subcatchments include:

- assigned raingage
- outlet node or subcatchment
- assigned land uses

- tributary surface area
- imperviousness
- slope
- characteristic width of overland flow
- Manning's n for overland flow on both pervious and impervious areas
- depression storage in both pervious and impervious areas
- percent of impervious area with no depression storage.

3.2.3 Junction Nodes

Junctions are drainage system nodes where links join together. Physically they can represent the confluence of natural surface channels, manholes in a sewer system, or pipe connection fittings. External inflows can enter the system at junctions. Excess water at a junction can become partially pressurized while connecting conduits are surcharged and can either be lost from the system or be allowed to pond atop the junction and subsequently drain back into the junction.

The principal input parameters for a junction are:

- invert elevation
- height to ground surface
- ponded surface area when flooded (optional)
- external inflow data (optional).

3.2.4 Outfall Nodes

Outfalls are terminal nodes of the drainage system used to define final downstream boundaries under Dynamic Wave flow routing. For other types of flow routing they behave as a junction. Only a single link can be connected to an outfall node.

The boundary conditions at an outfall can be described by any one of the following stage relationships:

- the critical or normal flow depth in the connecting conduit
- a fixed stage elevation
- a tidal stage described in a table of tide height versus hour of the day

- a user-defined time series of stage versus time.

The principal input parameters for outfalls include:

- invert elevation
- boundary condition type and stage description
- presence of a flap gate to prevent backflow through the outfall.

3.2.5 Conduits

Conduits are pipes or channels that move water from one node to another in the conveyance system. Their cross-sectional shapes can be selected from a variety of standard open and closed geometries as listed in Table 3-1. Irregular natural cross-section shapes are also supported.

SWMM uses the Manning equation to express the relationship between flow rate (Q), cross-sectional area (A), hydraulic radius (R), and slope (S) in open channels and partially full closed conduits. For standard U.S. units,

$$Q = \frac{1.49}{n} A R^{2/3} \sqrt{S}$$

where n is the Manning roughness coefficient. For Steady Flow and Kinematic Wave flow routing, S is interpreted as the conduit slope. For Dynamic Wave flow routing it is the friction slope (i.e., head loss per unit length).

The principal input parameters for conduits are:

- names of the inlet and outlet nodes
- offset heights of the conduit above the inlet and outlet node inverts
- conduit length
- Manning's roughness
- cross-sectional geometry
- entrance/exit losses
- presence of a flap gate to prevent reverse flow.

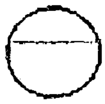
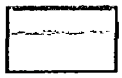
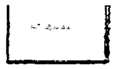
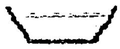
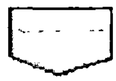
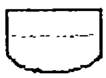
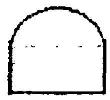
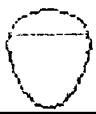
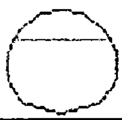
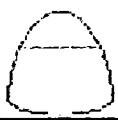
Name	Parameters	Shape	Name	Parameters	Shape
Circular	Depth		Filled Circular	Depth, Filled Depth	
Rectangular - Closed	Depth, Width		Rectangular – Open	Depth, Width	
Trapezoidal	Depth, Base Width, Side Slopes		Triangular	Depth, Top Width	
Horizontal Ellipse	Depth, Max., Width		Vertical Ellipse	Depth, Max. Width	
Arch	Depth, Max. Width		Parabolic	Depth, Top Width	
Power	Depth, Top Width, Exponent		Rectangular-Triangular	Depth, Top Width, Triangle Height	
Rectangular-Round	Depth, Top Width, Bottom Radius		Modified Baskethandle	Depth, Top Width	
Egg	Depth		Horseshoe	Depth	
Gothic	Depth		Catenary	Depth	
Semi-Elliptical	Depth		Baskethandle	Depth	
Semi-Circular	Depth				

FIG. 3.2. Available cross section shapes for conduits

3.3 OVERVIEW OF COMPUTATIONAL METHODS

SWMM is a physically based, discrete-time simulation model. It employs principles of conservation of mass, energy, and momentum wherever appropriate. This section briefly describes the methods SWMM uses to model stormwater runoff quantity and quality through the following physical processes :

1. Surface Runoff
2. Infiltration
3. Groundwater
4. Flow Routing
5. Surface Ponding

3.3.1 Surface Runoff

The conceptual view of surface runoff used by SWMM is illustrated in ***FIG. 3.3.*** below. Each subcatchment surface is treated as nonlinear reservoir. Inflow comes from precipitation and any designed upstream subcatchments. There are several outflows, including infiltration, evaporation, and surface runoff. The capacity of this "reservoir" is the maximum depression storage, which is the maximum surface storage provided by ponding, surface wetting, and interception. Surface runoff per unit area, Q, occurs only when the depth of water in the "reservoir" exceeds the maximum depression storage, dp, in which case the outflow is given by Manning's equation. Depth of water over the subcatchment (d in m) is continuously updated with time (t in seconds) by solving numerically a water balance equation over the subcatchment.

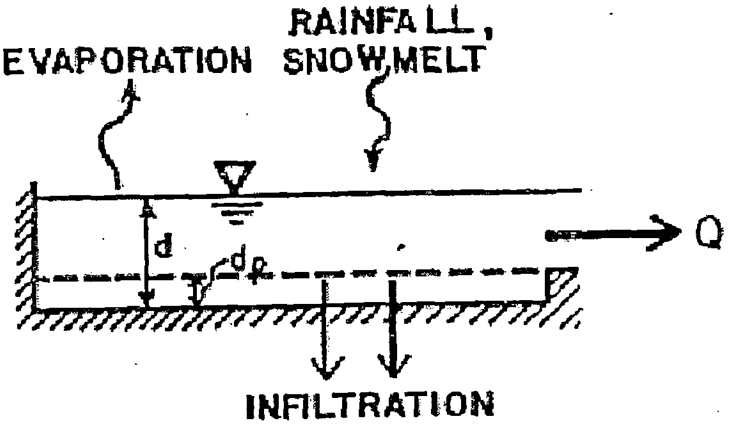


FIG 3.3. Conceptual View of Surface Runoff.

3.3.2 Infiltration

Infiltration is the process of rainfall penetrating the ground surface into the unsaturated soil zone of pervious subcatchment areas. SWMM offers three choices for modeling infiltration :

Horton's Equation

This method is based on empirical observations showing that infiltration decrease exponentially from an initial maximum rate to some minimum rate over the course of a long rainfall event. Input parameters required by this method include the maximum and minimum infiltration rates, a decay coefficient that describes how fast the rate decreases over time, and a time it takes a fully saturated soil to completely dry.

Green-Ampt Method

This method for modeling infiltration assumes that sharp wetting front exist in the soil column, separating soil with some initial moisture content below from saturated soil above. The input parameters required are the initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front.

Curve Number Method

This approach is adopted from the NRCS (SCS) Curve Number Method for estimating runoff. It assumes that the total infiltration capacity of a soil can be found from the soil's tabulated Curve Number. During a rain event this capacity is depleted as a function of a cumulative rainfall and remaining capacity. The input parameters for this method are the curve number, the soil's hydraulic conductivity (used to estimate a minimum separation time for distinct rain events), and a time it takes a fully saturated soil to completely dry.

3.3.3 Groundwater

FIG 3.4. is a definitional sketch of the two-zone groundwater model that is used in SWMM. The upper zone is unsaturated with a variable moisture content of θ.

The lower zone is fully saturated and therefore its moisture content is fixed at the soil porosity Ø.

The fluxes shown in the figure, expressed as a volume per unit area per unit time, consist of the following :

- **f_I** infiltration from the surface
- **f_{EU}** evapotranspiration from the upper zone which is a fixed fraction of the un-used surface evaporation
- **f_U** percolation from the upper to lower zone which depends on the upper zone moisture content θ and depth d_U
- **f_{EL}** evapotranspiration from the lower zone, which is a function of the depth of the upper zone depth d_U
- **f_L** percolation from the lower zone to deep groundwater which depends on the lower zone depth d_L
- **f_G** lateral groundwater interflow to the drainage system, which depends on the lower zone depth d_L as well as the depth in the receiving channel or node.

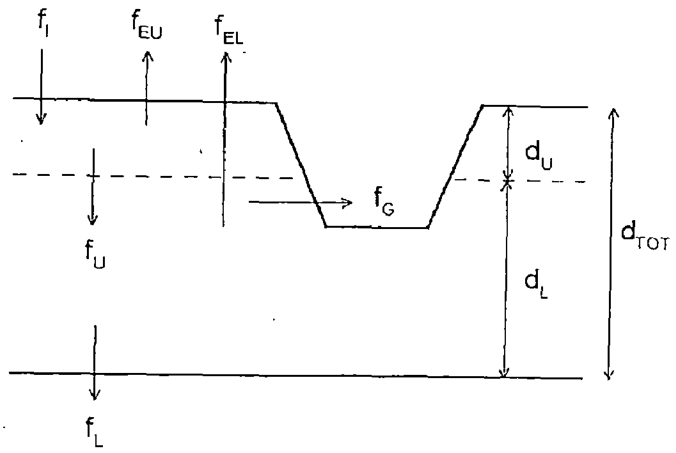


FIG. 3.4. Two-zone Groundwater Model

After computing the water fluxes that exist during a given time step, a mass balance is written for the change in water volume stored in each zone so that anew water table depth and unsaturated zone moisture content can be computed for the next time step.

3.3.4 Flow Routing

Flow routing within a conduit link in SWMM is governed by the conservation of mass and momentum equations for gradually varied, unsteady flow (i.e., the Saint Venant flow equation). The SWMM user has a choice on the level of sophistication used to solve these equations :

- Steady Flow Routing
- Kinematic Wave Routing
- Dynamic Wave Routing

Steady Flow Routing

Steady flow routing represents the simplest type of routing possible (actually no routing) by assuming that within each computational time step flow is uniform and steady. Thus it simply translates inflow hydrographs at the upstream end of the conduit to the downstream end, with in delay or change in shape. The Manning equation is used to relate flow rate to flow area (or depth).

This type routing cannot account for channel storage, backwater effects, entrance/exit losses, flow reversal or pressurized flow. It can only be used with dendritic conveyance networks, where each node has only a single outflow link (unless the node is a divider in which case two outflow links are required). This form of routing in insensitive to the time step employed and is really on appropriate for preliminary analysis using long-term continuous simulations.

Kinematic Wave Routing

This routing method solves the continuity equation along with a simplified form of the momentum equation in each conduit. The latter requires that the slope of the water surface equals the slope of the conduit.

The maximum flow that can be conveyed through conduit is the full-flow Manning equation value. Any flow in excess of this entering the inlet node is either lost from the system or can pond atop the inlet node and be re-introduced into the conduit as capacity becomes available.
Kinematic wave routing allows flow and area to vary both spatially and temporally within a conduit. This can result in attenuated and delayed outflow hydrographs as inflow is routed through the channel. However this form of routing cannot account for backwater effects, entrance/exit losses, flow reversal, or pressurized flow, and is also restricted to dendritic network layouts. It can usually maintain numerical stability with moderately large time steps, on the order of 5 to 15 minutes. If the aforementioned effects are not expected to be significant then this alternative can be an accurate and efficient routing method, especially for long-term simulations.

Dynamic Wave Routing

Dynamic Wave Routing solves the complete one-dimensional Saint Venant flow equations and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes.

With this form of routing it is possible to represent pressurized flow when a closed conduit becomes full, such that flows can exceed the full-flow Manning equation value. Flooding occurs when the water depth at a node exceeds the maximum available depth, and the excess flow is either lost either lost from the system or can pond atop the node and re-enter the drainage system.

Dynamic wave routing can account for channel storage, backwater, entrance/exit losses, for reversal, and pressurized flow. Because it couples together the solution for both water levels at nodes and flow in conduits it can be applied to any general network layout, even those containing multiple downstream diversions and loops. It is the method of choice for systems subjected to significant backwater effects due to downstream flow restrictions

and with flow regulation via weirs and orifices. This generally comes at a price of having to use much smaller time steps, on the order of a minute or less (SWMM will automatically reduce the user-defined maximum time step as needed to maintain numerical stability).

3.3.5 Surface Ponding

Normally in flow routing, when the flow into a junction exceeds the capacity of the system to transport it further downstream, the excess volume overflows the system and is lost. An option exists to have instead the excess volume be stored atop the junction, in a ponded fashion, and be reintroduced into the system as capacity permits. Under Steady and Kinematic Wave flow routing, the ponded water is stored simply as an excess volume. For Dynamic Wave routing, which is influenced by the water depths maintained at nodes, the excess volume is assumed to pond over the node with a constant surface area. This amount of surface area is an input parameter supplied for the junction. Alternatively, the user may wish to represent the surface overflow system explicitly. In open channel systems this can include road overflows at bridges or culvert crossings as well as additional floodplain storage areas. In closed conduit systems, surface overflows may be conveyed down streets, alleys, or other surface routes to the next available stormwater inlet or open channel. Overflows may also be impounded in surfaces depressions such as parking lots, back yards or other areas.

CHAPTER IV

SIMULATION STUDY OF SEKANAK DRAINAGE SYSTEM IN PALEMBANG CITY

4.1 THE STUDY AREA

4.1.1 Palembang City

Palembang City is the capital of South Sumatra Province. In the year of 2002, the city had a population of 1.5 millions. Palembang City lies on the low elevation ranging +2 to +4 meters above mean sea level (m, M.S.L.), and has a total area of 403 km^2 of which almost half is in the swampy areas located in the low-lying topography.

Palembang is located along the Musi River approximately at 85 km inland from the sea. At Palembang the Musi River is about 350 m in width and is affected by tides. The range of tidal variation is about 2.5 m at this section. During the rainy season, flood water level of the Musi River rises by about 1 m above that of the dry season.

Ground elevation of the lower basin of the Musi River is very flat and low ranging +2 m to +5 m,M.S.L. Because of the flatness of the land, drainage conditions are bad as a whole and many areas are frequently inundated after rainfall. The swamp areas are found along the main Musi, Ogan, Komering, Keramasan rivers, and other tributaries and branches.

The storm water from the city area is finally drained to the Musi River through 19 major drainage systems. To mitigate the inundation damages, drainage improvement works have been carried by improving drainage channels and constructing detention pond.

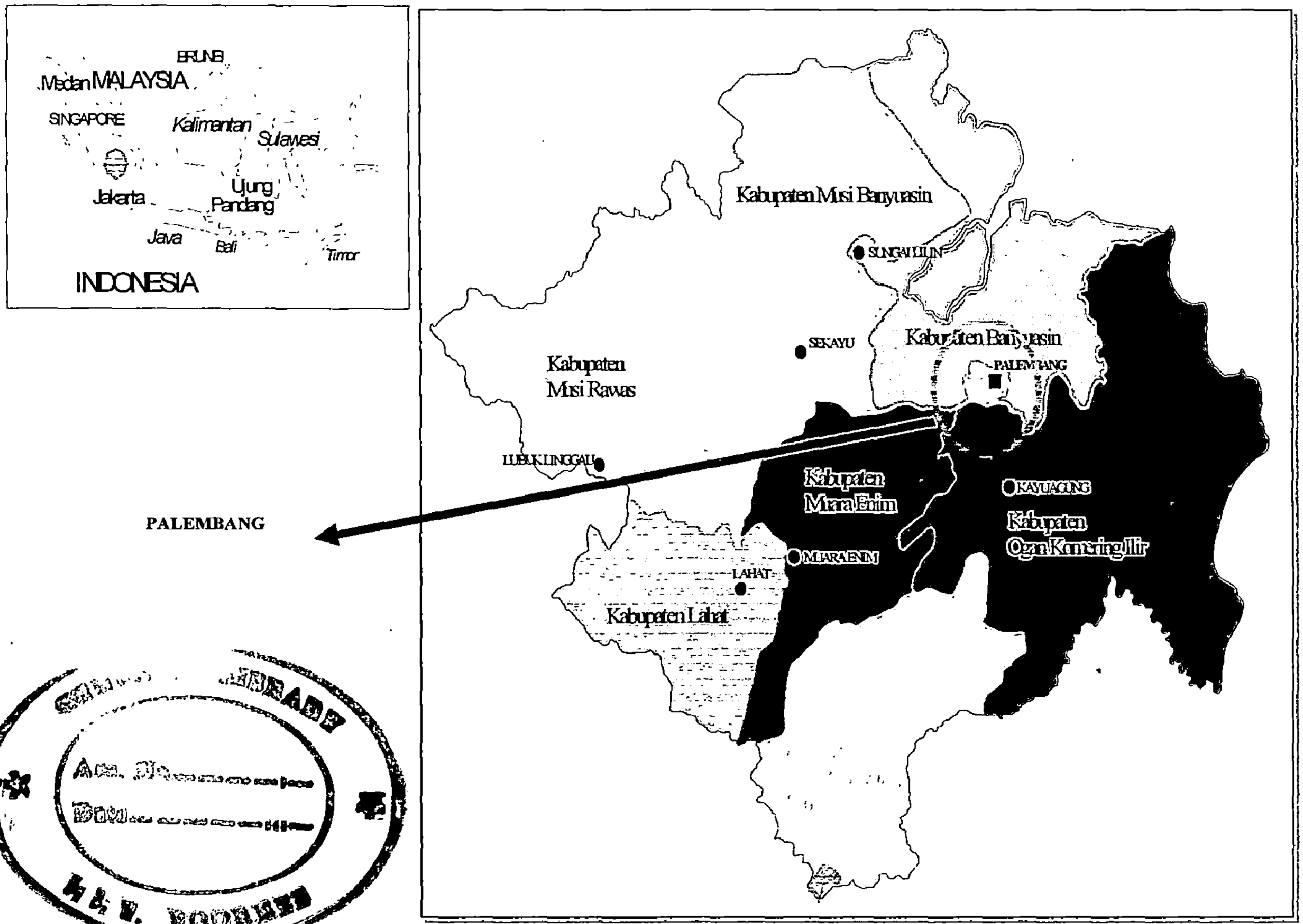


FIG. 4.1. Location of The Study Area

4.1.2 Drainage Systems in Palembang City

The drainage in Palembang City is divided into 19 drainage systems with a total area of 403 km^2. The drainage system is shown in ***Annexure 1.***

4.1.3. Sekanak Drainage System

Sekanak Drainage System lies in center of Palembang city. The Drainage System consists of primary channels, secondary channels, and tertiary channels and the storm water from the city area is finally drained to the Musi River.

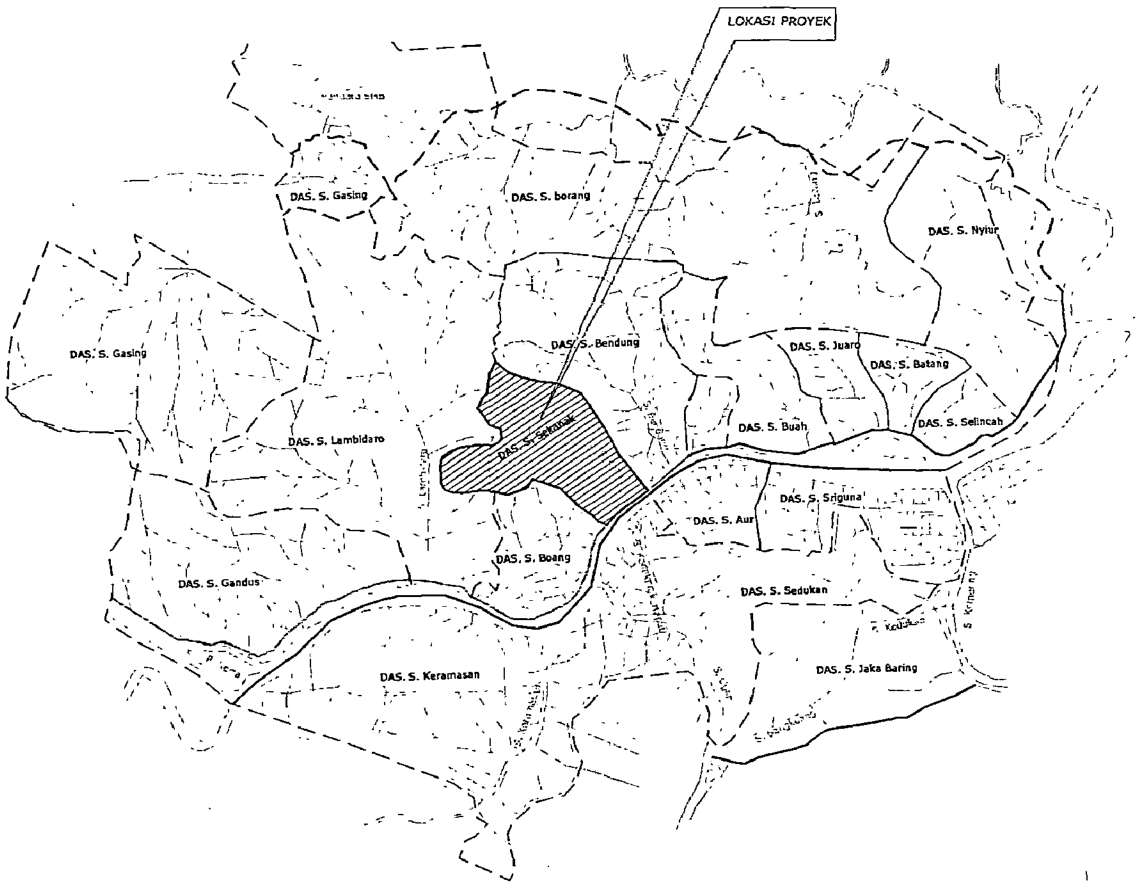


FIG. 4.2. Sekanak Drainage System

4.2 DESIGN STORM

Twenty six years of daily rainfall data (1976 to 2001) at Palembang City in the South Sumatera Province (Indonesia) has been used for the estimation of storm rainfall of various return periods. ***Table 4.1*** shows maximum daily rainfall for twenty six years from 1976 to 2001. Gumble's probability distribution has been applied for obtaining T year 24 hr rainfall values. ***Table 4.2, Table 4.3, Table 4.4*** are the calculation sheet for T year 24 hr rainfall values :

- 25 year 24 hr rainfall 144.23 mm
- 50 year 24 hr rainfall 163.49 mm
- 100 year 24 hr rainfall 182.60 mm

Time distribution coefficient of cumulative hourly rainfall for 24 hr rainfall values (***Table 4.5***) used to obtain hourly distribution of 25 year, 50 year and 100 year storm rainfall as shown in ***Table 4.6***.

Resume of Maximum Yearly Rainfall Data

	Year	Maximum Rainfall
1	1976	71
2	1977	139
3	1978	122
4	1979	101
5	1980	137
6	1981	94
7	1982	91
8	1983	72
9	1984	118
10	1985	107
11	1986	109
12	1987	66
13	1988	88
14	1989	133
15	1990	97
16	1991	125
17	1992	107
18	1993	145
19	1994	90
20	1995	157
21	1996	122
22	1997	105
23	1998	115
24	1999	87
25	2000	99
26	2001	110
		2807

TABLE 4.1. DATA FROM KENTEN STATION MAXIMUM DAILY RAINFALL (mm)

Year	January	February	March	April	May	June	July	August	September	October	November	December
1976	13	40	39	71	0	0	0	0	0	0	0	0
1977	0	0	0	61	0	43	16	20	31	33	139	97
1978	122	79	73	93	41	35	0	0	0	0	0	32
1979	0	0	0	99	10	0	0	68	43	59	101	71
1980	17	37	53	70	23	35	43	35	133	137	58	53
1981	48	53	77	68	85	51	43	23	54	72	94	71
1982	24	38	91	67	61	81	69	11	40	61	25	75
1983	72	70	61	18	67	71	33	36	6	39	66	65
1984	118	35	45	66	59	28	81	65	88	67	73	43
1985	49	58	47	64	31	43	107	22	36	70	55	39
1986	95	68	109	65	25	35	24	44	57	47	62	72
1987	26	21	56	54	46	8	18	59	2	16	27	66
1988	50	42	88	73	27	24	13	33	29	63	31	73
1989	54	59	51	36	46	10	133	90	29	95	28	72
1990	38	66	50	35	85	97	55	64	11	48	58	53
1991	42	40	93	44	33	17	10	0	33	16	125	108
1992	60	75	107	57	43	23	57	47	49	83	65	68
1993	60	53	70	80	35	35	114	16	27	27	72	145
1994	55	28	69	90	37	26	88	0	10	16	36	67
1995	76	91	48	48	51	157	16	34	23	73	62	128
1996	41	69	60	54	25	58	49	26	33	79	122	60
1997	33	44	66	70	105	37	5	4	0	6	43	90
1998	40	41	115	96	45	57	44	26	55	90	48	81
1999	68	31	85	84	31	85	29	27	13	50	87	57
2000	53	31	63	99	37	62	15	51	24	82	76	58
2001	99	57	91	71	56	71	45	46	52	99	85	110
Median	49.5	43	64.5	67.5	39	36	38	30	30	60	62	69.5

Table 4.2. Gumble Probability Distribution of 24 hr Rainfall at Kenten Station

ORDER NUMBER m	MAX. RAINFALL (mm)	DESC.RAINFALL (mm)	T_r=(N+1)/m (years)	$(X - X_{avr})^2$
1	0	145	27.00	5,430.56
2	97	128	13.50	3,214.02
3	32	110	9.00	1,497.09
4	71	108	6.75	1,346.33
5	53	97	5.40	660.09
6	71	90	4.50	349.40
7	75	81	3.86	93.94
8	65	75	3.38	13.63
9	43	73	3.00	2.86
10	39	72	2.70	0.48
11	72	72	2.45	0.48
12	66	71	2.25	0.09
13	73	71	2.08	0.09
14	72	68	1.93	10.94
15	53	67	1.80	18.56
16	108	66	1.69	28.17
17	68	65	1.59	39.79
18	145	60	1.50	127.86
19	67	58	1.42	177.09
20	128	57	1.35	204.71
21	60	53	1.29	335.17
22	90	53	1.23	335.17
23	81	43	1.17	801.33
24	57	39	1.13	1,043.79
25	58	32	1.08	1,545.09
26	110	0	1.04	5,084.79
$\sum$	1,854.00	71.31		22,361.54

X_{avr} = 71,31 mm $\delta_{n-1} = \sqrt{((\sum(X-X_{avr})^2)/(N-1))}$=29.91 mm

From Table 4.5 and 4.6, for N = 26, Y_n =0.53476 and S_n = 1.09256

T Return Periode	$Y_T = -[\ln.\ln(T/(T-1))]$	$K = (Y_T-Y_n)/S_n$	$X_T = X_{avr} + K\,\delta_{n-1}$
T = 25 year	3.1985	2.44	**144.23 mm**
T = 50 year	3.9019	3.08	**163.49 mm**
T = 100 year	4.6001	3.72	**182.60 mm**

Table 4.3. Reduced Mean Y_n in Gumble's Extreme Value Distribution

N	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.5070	0.5100	0.5128	0.5157	0.5181	0.5202	0.5220
20	**0.5326**	0.5252	0.5268	0.5283	0.5296	0.5309	0.5320	0.5332	0.5343	0.5353
30	**0.5362**	0.5371	0.5380	0.5388	0.5396	0.5402	0.5410	0.5418	0.5424	0.5430
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.5530	0.4433	0.5535	0.5538	0.5540	0.5543	0.5545
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.5570	0.5572	0.5574	0.5576	0.5578	0.5580	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.5600									

Source : K Subramaya, Engineering Hydrology

Table 4.4 : Reduced Standard Deviation S_n in Gumble's Extreme Value Distribution

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	**1.0628**	1.0696	1.0754	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	**1.1124**	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.1770	1.1782	1.793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1906	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1,1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1,2049	1.2055	1.2060
100	1.2065									

Source : K Subramaya, Engineering Hydrology

Note:
1. From interpolation N = 26 ; Yn = 0.53476
2. From interpolation N = 26 ; Sn = 1.09256

TABLE 4.5 TIME DISTRIBUTION COEFFICIENT OF CUMULATIVE HOURLY RAINFALL

DESIGN STORM DURATION (HOURS)																								INTER-MEDIATE HOURS
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	
1.00	0.85	0.73	0.62	0.56	0.52	0.46	0.43	0.41	0.38	0.37	0.34	0.32	0.30	0.28	0.26	0.25	0.23	0.21	0.20	0.19	0.18	0.17	0.17	1
	1.00	0.92	0.82	0.75	0.69	0.63	0.60	0.57	0.54	0.52	0.49	0.44	0.43	0.41	0.40	0.39	0.37	0.33	0.32	0.31	0.30	0.28	0.27	2
		1.00	0.94	0.87	0.82	0.75	0.71	0.68	0.65	0.62	0.60	0.55	0.53	0.51	0.50	0.48	0.46	0.43	0.41	0.39	0.38	0.37	0.36	3
			1.00	0.96	0.91	0.85	0.81	0.76	0.73	0.71	0.67	0.63	0.61	0.59	0.57	0.56	0.54	0.50	0.48	0.46	0.45	0.44	0.43	4
				1.00	0.97	0.92	0.88	0.84	0.81	0.77	0.74	0.70	0.68	0.66	0.64	0.62	0.60	0.56	0.54	0.52	0.51	0.50	0.48	5
					1.00	0.97	0.94	0.90	0.87	0.83	0.81	0.77	0.74	0.72	0.69	0.68	0.66	0.61	0.59	0.57	0.56	0.55	0.53	6
						1.00	0.97	0.95	0.92	0.88	0.86	0.82	0.79	0.77	0.74	0.73	0.71	0.66	0.64	0.62	0.61	0.60	0.58	7
							1.00	0.98	0.95	0.93	0.90	0.86	0.84	0.81	0.79	0.77	0.75	0.71	0.69	0.67	0.66	0.65	0.63	8
								1.00	0.98	0.96	0.94	0.90	0.87	0.85	0.83	0.81	0.79	0.75	0.73	0.71	0.70	0.69	0.67	9
									1.00	0.98	0.96	0.93	0.90	0.88	0.87	0.85	0.83	0.79	0.77	0.74	0.73	0.72	0.70	10
										1.00	0.96	0.96	0.93	0.91	0.90	0.88	0.86	0.82	0.80	0.77	0.76	0.75	0.73	11
											1.00	0.98	0.96	0.94	0.92	0.90	0.88	0.85	0.83	0.80	0.79	0.78	0.76	12
												1.00	0.98	0.96	0.94	0.92	0.90	0.88	0.86	0.83	0.82	0.81	0.79	13
													1.00	0.98	0.96	0.94	0.92	0.90	0.89	0.86	0.85	0.83	0.82	14
														1.00	0.98	0.96	0.94	0.92	0.91	0.89	0.87	0.85	0.84	15
															1.00	0.98	0.96	0.94	0.93	0.91	0.89	0.87	0.86	16
																1.00	0.98	0.96	0.95	0.93	0.91	0.89	0.88	17
																	1.00	0.98	0.97	0.95	0.93	0.91	0.90	18
																		1.00	0.99	0.97	0.95	0.93	0.92	19
																			1.00	0.99	0.97	0.95	0.94	20
																				1.00	0.99	0.97	0.96	21
																					1.00	0.99	0.98	22
																						1.00	0.99	23
																							1.00	24

Source : Central Water Commission CWC 1994

Table 4.6. Hourly Distribution of 24 Hr Storm Rainfall

Time	Storm 24 hr Distribution Coefficient	24 hr Rainfall of T year Return Period: 25 yr Return Period, Depth : 144.23 mm, Duration : 24 hr — Cum. rf mm	Incr. rf mm	50 yr Return Period, Depth : 163.49 mm, Duration : 24 hr — Cum. rf mm	Incr. rf mm	100 yr Return Period, Depth : 182.60 mm, Duration : 24 hr — Cum. rf mm	Incr. rf mm
1	0.17	24.52	24.52	27.79	**27.79**	31.04	31.04
2	0.27	38.94	14.42	44.14	**16.35**	49.30	18.26
3	0.36	51.92	12.98	58.86	**14.71**	65.74	16.43
4	0.43	62.02	10.10	70.30	**11.44**	78.52	12.78
5	0.48	69.23	7.21	78.47	**8.17**	87.65	9.13
6	0.53	76.44	7.21	86.65	**8.17**	96.78	9.13
7	0.58	83.65	7.21	94.82	**8.17**	105.91	9.13
8	0.63	90.87	7.21	103.00	**8.17**	115.04	9.13
9	0.67	96.64	5.77	109.54	**6.54**	122.34	7.30
10	0.70	100.96	4.33	114.44	**4.90**	127.82	5.48
11	0.73	105.29	4.33	119.35	**4.90**	133.30	5.48
12	0.76	109.62	4.33	124.25	**4.90**	138.78	5.48
13	0.79	113.94	4.33	129.16	**4.90**	144.26	5.48
14	0.82	118.27	4.33	134.06	**4.90**	149.73	5.48
15	0.84	121.16	2.88	137.33	**3.27**	153.39	3.65
16	0.86	124.04	2.88	140.60	**3.27**	157.04	3.65
17	0.88	126.92	2.88	143.87	**3.27**	160.69	3.65
18	0.90	129.81	2.88	147.14	**3.27**	164.34	3.65
19	0.92	132.69	2.88	150.41	**3.27**	167.99	3.65
20	0.94	135.58	2.88	153.68	**3.27**	171.65	3.65
21	0.96	138.46	2.88	156.95	**3.27**	175.30	3.65
22	0.98	141.35	2.88	160.22	**3.27**	178.95	3.65
23	0.99	142.79	1.44	161.85	**1.63**	180.78	1.83
24	1.00	144.23	1.44	163.49	**1.63**	182.60	1.83

4.3 INPUT DATA AND PARAMETERS FOR SWMM

Annexure 3 is the computer print out of input data and parameters used in simulation study of Sekanak Drainage System. Input data and parameters are explained below.

4.3.1 Rain Gages

Rain Gages supply precipitation data for one or more subcatchment areas in a study region. The rainfall data can be either a user-defined time series or come from an external file. Several different popular rainfall file formats currently in use are supported, by a standard user-defined format.

The principal input properties of rain gauges include :

- rainfall data type (e.g., intensity, volume)
- recording time interval (e.g., hourly, 15-minute, etc.)
- source of rainfall data (input time series or external file)
- name of rainfall data source

Rain Gage Properties

Name	User-assigned rain gage name. It is ***Gage1***
Time Series -Series Name	Name of time series with rainfall data if Data Source selection was TIMESERIES TS1. The 50 year return period 24 hour in ***Table 4.6***. is used for input.

4.3.2 Subcatchments

Subcatchments are hydrologic units of land whose topography and drainage system elements direct surface runoff to a single discharge point. The Sekanak drainage system is divided into an appropriate number of subcatchments, and for identifying the outlet point of each subcatchment. Discharge outlet points can be either nodes of the drainage system or other subcatchments.

Subcatchment can be divided into pervious and impervious subareas. Surface runoff can infiltrate into the upper soil zone of the pervious subarea, but not through the impervious subarea. Impervious areas areas themselves divided into two subareas – one that contains depression storage and another that does not. Runoff flow from one subarea in a subcatchment can be routed to the other subarea, or both subareas can drain to the subcatchment outlet. Infiltration of rainfall from the pervious area of a subcatchment into the unsaturated upper soil zone can be described using Green-Ampt Infiltration model.

The Sekanak drainage system has been divided into 14 subcatchments keeping in view the existing drainage channels. Subcatchment areas and percent imperviousness of the subcatchment areas are given in ***Table 4.7*** below :

TABLE 4.7. Subcatchment Properties of Sekanak Drainage System

Notation	Identification Mark for Subcatchment	Area (Ha)	Elevation Range (m)	Imperviousness (%)	Width (m)
1	2	3	4	5	6
S1	Sekanak A.1 – A.18	53.59	104.51 - 103.36	45	1,317.68
S2	Bukit Lama 18/14 – A.18	58.78	108.16 - 103.36	45	1,365.06
S3	Sekanak A.18 – A.22	53.07	103.36 - 103.12	45	1,592.96
S4	Kampus 22/10 – A.22	47.50	103.81 - 103.12	50	1,210.26
S5	Sekanak A.22 – A.24	64.05	103.12 - 102.95	50	1,803.25
S6	Khodijah P.31 – P.11	50.15	103.59 - 103.29	45	1,438.62
S7	Khodijah P.11 – A.24	77.53	103.29 - 102.95	50	2,129.89
S8	Sekanak A.24 – A.35	139.07	102.95 - 102.19	65	1,453.85
S9	Baung 35/31 – 35/12a	107.04	104.18 - 103.43	45	2,420.70
S10	Baung 35/12a – 35/6	90.89	103.43 - 102.98	65	1,822.51
S11	Tridinanti S.31 – 35/6	82.20	104.71 - 102.98	65	2,305.23
S12	Baung 35/6 – A.35	88.65	102.98 - 102.19	70	1,564.77
S13	Sekanak A.35 – A.44	138.30	102.19 - 101.56	65	2,173.63
S14	Sekanak A.44 – A.52	88.68	101.56 - 100.99	65	1,992.05
	TOTAL	**1,139.50**			

Subcatchment Properties

Name	User-assigned subcatchment name as given in ***Figure 5.1.***
Rain Gage	Name of the rain gage associated with the subcatchment. Gage 1 for all subcatchment.
Outlet	Name of the node or subcatchment which receives the subcatchment's runoff as given in ***FIG. 5.1.***
Area	Area of the subcatchment (hectares).
Width	Characteristic width of overland flow path for sheet flow runoff (meters). An initial estimate of the characteristic width is given by the subcatchment area divided by the average maximum overland flow length. The maximum overland flow is the length of the flow path from the inlet to the farthest drainage point of the subcatchment. These paths should reflect slow flow, such as over pervious surfaces, more than rapid flow over pavement.
% Slope	Average percent slope of the subcatchment.
% Imperv	Percent of land area which is impervious. It ranges from 10% to 70%.
N-Imperv	Manning's n for overland flow over the impervious portion of the subcatchment . It is taken as **0.014.**
N-Perv	Manning's n for overland flow over the pervious portion of the subcatchment. It is taken as **0.13.**
Dstore-Imperv	Depth of depression storage on the impervious portion of the subcatchment. It is taken as zero.
Dstore-Perv	Manning's n for overland flow over the pervious portion of the subcatchment. It is taken as 0.1 inches = **2.54 mm.**
%Zero-Imperv	Percent of the impervious area with no depression storage.Assumed to be 100%.
Subarea Routing	Runoff from both area flows directly to outlet.
Percent Routed	Percent of runoff routed between subareas. Assumed to be 100%.
Infiltration	The option controls how infiltration of rainfall into the upper zone of subcatchment is modeled. Green Ampt parameters such as **suction head (109.98 mm), hydraulic conductivity (10.92 mm/hr). Initial moisture deficient (equal to difference of porosity and field capacity =0.263)** corresponding to **sandy loam** for all subcatchment have been taken.

Result of the textural of soil profile in field of Sekanak System is shown below :

Table 4.8. Soil Type in Sekanak System

Depth	Soil Profile	Textural Analysis			USDA Class
		Sand %	Silt %	Clay %	
0.00		66.00	12.00	22.00	**Sandy Loam**
30.00		72.10	12.60	15.30	**Sandy Loam**
60.00	....	85.00	4.60	10.60	Loamy sand
75.00		94.05	1.75	4.20	Sand
105.00	--------------------	62.10	1.80	36.10	Clay
120.00	!!!!!!!!!!!!!!!!!!!!!	77.10	12.30	10.60	Loamy Sand
150.00					
	\-\-\-\-\-\-\-\-\-\-\-\-\-				

Source : Drainage Master Plan and Detail Design Sekanak Drainage System, Palembang City(2003)

Properties of Sandy Loam soil as obtained from literature are given below :

Table 4.9. Soil Characteristics

Soil Texture Class	**K**	**Ψ**	**Φ**	**FC**	**WP**
Sand	4.74	1.93	0.437	0.062	0.024
Loamy Sand	1.18	2.40	0.437	0.105	0.047
Sandy Loam	**0.43**	**4.33**	**0.453**	**0.190**	**0.085**
Loam	0.13	3.50	0.463	0.232	0.116
Silt Loam	0.26	6.69	0.501	0.284	0.135
Sandy Clay Loam	0.06	8.66	0.398	0.244	0.136
Clay Loam	0.04	8.27	0.464	0.310	0.187
Silty Clay Loam	0.04	10.63	0.471	0.342	0.210
Sandy Clay	0.02	9.45	0.430	0.321	0.221
Silty Clay	0.02	11.42	0.479	0.371	0.251
Clay	0.01	12.60	0.475	0.378	0.265

Source : Rawls, W.J. et al., (1983). J.Hyd..Engr., 109:1316

Where, K = saturated hydraulic conductivity, in/hr
Ψ = suction head, in
Φ = porosity, fraction
FC = field capacity, fraction
WP = wilting point, fraction

4.3.3 Junction Nodes

Junctions are drainage system nodes where links join together. Physically they can represent the confluence of natural surface channels, manholes in a sewer system, or pipe connection fittings. External inflows can enter the system at junctions. Excess water at a junction can become partially pressurized while connecting conduits are surcharged and can either be lost from the system or be allowed to pond atop the junction and subsequently drain back into the junction.

Junction Properties

Name	User-assigned junction name as shown in ***FIG. 5.1.***
Inflows	Assign time series, dry weather to the junction. Assumed to be zero.
Invert El.	Invert elevation of the junction (meters).
Max.Depth	Maximum depth of the junction (i.e.,from ground surface to invert) (meters).
Initial Depth	Depth of water at the junction at the start of the simulation (meters). Assumed to be zero.
Surcharge Depth	Additional depth of water beyond the maximum depth that is allowed before the junction floods (feet or meters). This parameter can be used to simulate bolted manhole covers. Assumed to be zero as no flooding is allowed.
Ponded Area	Area occupied by ponded water atop the junction after flooding occurs). It is taken to be zero.

The Sekanak Drainage System has been divided into 14 junctions with the properties are given in ***Table 4.10.*** below :

Table 4.10. Junction Node of Sekanak Drainage System

Junction	J1	J2	J3	J4	J5	J6
Invert El.(m)	104.51	103.36	105.16	103.12	103.81	103.59

Junction	J7	J8	J9	J10	J11	J12
Invert El.(m)	103.29	102.95	104.18	103.43	104.71	102.98

Junction	J13	J14
Invert El.(m)	102.19	101.56

4.3.4 Conduits

Conduits are pipes or channels that transport water from one node to another in the conveyance system. Their cross-sectional shapes can be selected from a variety of standard open and closed geometries. Irregular natural channels cross-section shapes are also supported.

The principal input parameters for conduits are :

- names of the inlet and outlet nodes
- conduit length
- Manning's roughness
- cross-sectional geometries
- presence of a flap gate to prevent reverse flow.

Conduit Properties

Name	User-assigned conduit name as shown in ***FIG. 5.1***
Inlet Node	Name of node on the inlet end of the conduit (which is normally the end at higher elevation).
Outlet Node	Name of node on the outlet end of the conduit (which is normally the end at lower elevation).
Shape	The geometric properties of the conduit's cross section. Open Rectangular and Trapezoidal channel section is taken side slope is 1.25 : 1 (H:V)
Length	Conduit length as shown in ***Table 4.11***
Roughness	Manning's roughness coefficient. It is taken as **0.014** for Brick Lined Channels and Concrete Open Channels.
Inlet Offset	Height of the conduit invert above the node invert at the upstream end of the conduit (meters).
Outlet Offset	Height of the conduit invert above the node invert at the downstream end of the conduit (meters).
Initial Flow	Initial flow in the conduit. It is zero.
Maximum Flow	Maximum flow allowed in the conduit under dynamic wave routing (flow units) – use 0 or leave blank if not applicable

Following are the properties of the conduits of Sekanak Drainage System in *Table 4.11* below.

Table 4.11. Conduits Properties of Sekanak Drainage System

Conduits	Shape	Max Depth (m)	Bottom Width (m)	Length (m)	Manning's n
1	2	3	4	5	6
C1	Trapezoidal (1.25 H:1V)	1.50	1.50	1,942.53	0.014
C2	Rectangular open	1.50	1.50	791.91	0.014
C3	Trapezoidal (1.25 H:1V)	1.75	2.50	401.28	0.014
C4	Rectangular open	1.50	2.00	821.72	0.014
C5	Trapezoidal (1.25 H:1V)	1.75	4.00	291.44	0.014
C6	Rectangular open	1.50	2.50	1,855.41	0.014
C7	Rectangular open	2.00	3.00	1,133.06	0.014
C8	Trapezoidal (1.25 H:1V)	2.00	6.00	1,266.19	0.014
C9	Rectangular open	2.00	2.50	1,400.76	0.014
C10	Rectangular open	2.25	3.50	735.17	0.014
C11	Rectangular open	1.75	2.50	1,471.38	0.014
C12	Trapezoidal (1.25 H:1V)	1.75	6.00	418.37	0.014
C13	Trapezoidal (1.25 H:1V)	2.75	8.00	1,068.38	0.014
C14	Trapezoidal (1.25 H:1V)	2.75	10.00	968.02	0.014

Note : Figures in bracket indicate side slope which is 1.25 H : 1V

4.3.5 Outfall Nodes

Outfalls are terminal nodes of the drainage system used to define final downstream boundaries. The principal input parameters for outfalls include :

- invert elevation
- Presence of a flap gate to prevent backflow through the outfall

Outfall Properties

Name	User-assigned outfall name as shown on ***Figure 5.1.***
Inflows	Assign time series, dry weather to the outfall.
Invert El.	Invert elevation of the outfall (meters).
Tide Gate	No tide gate present.
Type	Type of outfall boundary condition : FREE : outfall stage determined by minimum or critical

The invert elevation of outfall is 100.99 m

CHAPTER V

RESULTS AND DISCUSSION

As described in detail in Chapter II, Calculation of Normal Depth by Pillai's Method was carried out to examine economical dimension of open channel in Sekanak Drainage System, as shown in ***Table 5.1.*** All dimension of open channel in Sekanak Drainage System are satisfied and economical.

These sections are adopted and checked by SWMM for any flooding of the area. In order to simulate the Sekanak Drainage System *(FIG. 5.1.)*. The watershed was divided into 14 sub-watersheds, and these are designated as S1 through S14.

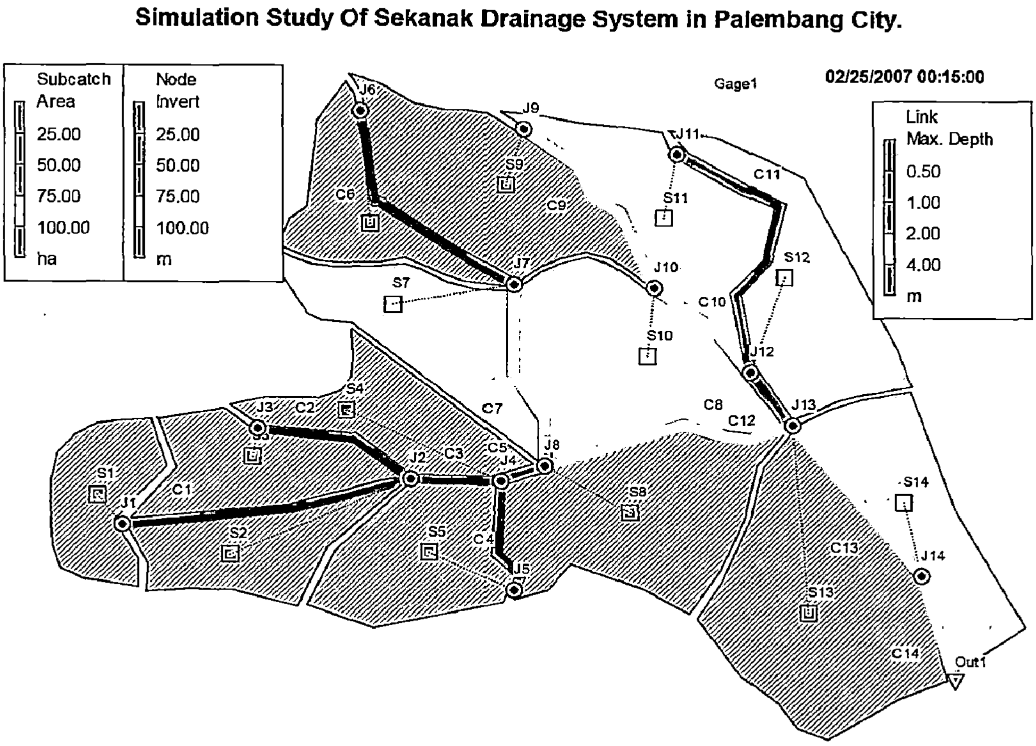


FIG. 5.1. Simulation Study of Sekanak Drainage System in Palembang City.

Table 5.1. Design of Open Channel (Calculation of Normal Depth) by Pillai's Method [9]

Channel Shapes	Channel No.	Q (m^3/sec)	So	Manning's n	Bed Width, b, (m)	Side Slope (m) for Trap. Ch.	Assumed yn (m)	Calculated yn (m)	Freeboard (m)	Max. Depth (m)	Adopted Max. Depth (m)	Velocity (m/sec)	Fr. No.
1	2	3	4	5	6	7	8	9	10	11	12	13	14
Rectangular Channels	2	2.06	0.0023	0.014	1.50		0.77	0.77	0.50	1.27	1.50	1.79	0.65
	4	1.86	0.0008	0.014	2.00		0.79	0.79	0.50	1.29	1.50	1.17	0.42
	6	1.68	0.0002	0.014	2.50		0.99	0.99	0.50	1.49	1.50	0.68	0.22
	7	4.21	0.0003	0.014	3.00		1.41	1.40	0.60	2.00	2.00	1.00	0.27
	9	3.80	0.0006	0.014	2.50		1.20	1.20	0.60	1.80	2.00	1.26	0.37
	10	8.15	0.0006	0.014	3.50		1.53	1.53	0.60	2.13	2.25	1.53	0.39
	11	4.03	0.0012	0.014	2.50		0.97	0.97	0.60	1.57	1.75	1.66	0.54
Trapezoidal Channels	1	1.79	0.0006	0.014	1.50	1.25	0.72	0.72	0.50	1.22	1.50	1.03	0.39
	3	5.47	0.0006	0.014	2.50	1.25	1.05	1.05	0.60	1.65	1.75	1.36	0.42
	5	9.66	0.0006	0.014	4.00	1.25	1.16	1.16	0.60	1.76	1.75	1.53	0.45
	8	19.03	0.0013	0.014	6.00	1.25	1.11	1.11	0.75	1.86	2.00	2.32	0.70
	12	15.63	0.0019	0.014	6.00	1.25	0.89	0.89	0.75	1.64	1.75	2.48	0.84
	13	40.27	0.0006	0.014	8.00	1.25	1.83	1.83	0.75	2.58	2.75	2.14	0.51
	14	43.65	0.0006	0.014	10.00	1.25	1.70	1.70	0.75	2.45	2.75	2.12	0.52

Minimal permissible velocity
0.6-0.9 m/s (2-3 ft/s) to prevent sedimentation
0.75 m/s (2.5 ft/s) to prevent vegetation growth

maximum permissible velocities
V < 2.1 m/s unreinforced channel
V < 5.5 m/s reinforced channels

Fr. No.
< 0.8

Note: as described in Chapter II (Calculation of Normal Depth or Uniform Flow Depth)
Ref. Principal of Fluid Mechanics and Fluid Machines by N. Narayana Pillai & C.R. Ramakhrisna (2006), Second Edition, Universities Press, Engineering.

FIG. 5.1. shows the junction nodes (designated as J1, J2, J3 etc.) and links (designated as C1, C2, C3, etc.) for SWMM application. As also described in Chapter III, SWMM accounts for various hydrologic processes that produce runoff from urban areas. These include: time-varying rainfall, evaporation of standing surface water, snow accumulation and melting, rainfall interception from depression storage, infiltration of rainfall into unsaturated soil layers, percolation of infiltrated water into groundwater layers, interflow between groundwater and the drainage system, nonlinear reservoir routing of overflow flow. Since its inception, SWMM has been used in thousands of sewer and stormwater studies throughout the world. Typical applications include: design and sizing of drainage system components for flood control, sizing of detention facilities and their appurtenances for flood control and water quality protection, flood plain mapping of natural channel system, designing control strategies for minimizing combined sewer overflows, evaluating the impact of outflow and infiltration on sanitary sewer overflows, generating non-point source pollutant loadings for waste load allocation studies, evaluating the effectiveness of BMPs for reducing wet weather pollutant loadings.

The SWMM is basically a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of subcatchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each subcatchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps. Among the above described potentials of the SWMM model, its following capabilities have been utilized: time-varying rainfall, rainfall interception from depression storage, infiltration of rainfall into unsaturated soil layers. These were sufficient for generation of runoff from the design storm using SWMM.

Computation of Design Storm

For the computation of design storm, as also described in detail in Chapter IV, 26 years of 24-hr annual maximum rainfall data were collected for the drainage basin (area = 11.4 km^2) and Gumbel Extreme Value (GEV) was employed to compute the 25-yr, 50-yr, and 100-yr return period storm rainfalls. These were 144.23 mm, 163.49 mm, and 182.60 mm, respectively. Consistent with the work of Kartika, Endra (2006) on Urban Flood Drainage Planning, the design storm corresponding to 50-yr was chosen for the study. This rainfall was then distributed in time (hourly) using ***Table 4.6.*** The resulting hyetograph shown in ***FIG. 5.2.***. Since the area of the drainage system (= 11.4 km^2) was small, the resulting was treated as to be uniform on whole drainage watershed.

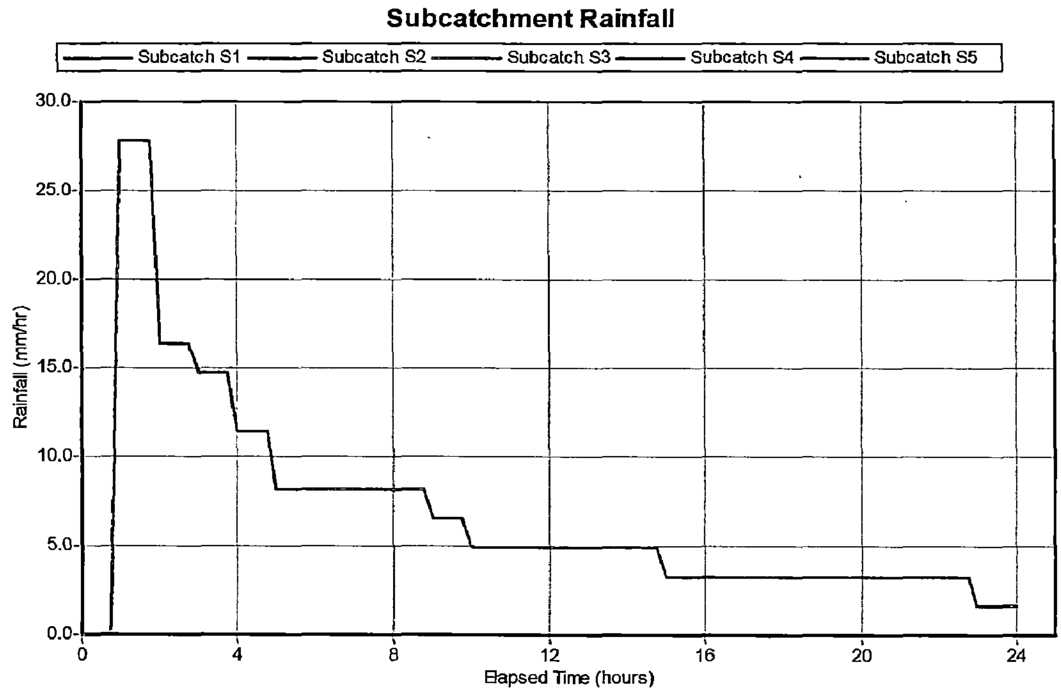


FIG. 5.2. Subcatchment Rainfall

Computation of Infiltration

For computation of infiltration, the physically based Green-Ampt model was employed. Here it is to note that the other losses such as the evaporation or evapotranspiration loss was not accounted for in the analysis for the reason that it was insignificant in computation of floods, and therefore, is usually ignored.

Among all other losses, in flood studies, it is of common experience that the infiltration loss forms to the most significant and major loss for accounting in runoff generation.

As described in Chapter III, in SWMM modeling, the Sekanak drainage system is divided into 14 number of subcatchments (***Fig. 5.1.***). For computational purpose, each subcatchment is further subdivided into pervious and impervious subareas. Surface runoff can infiltrate into the upper soil zone of the pervious subarea, but not through the impervious subarea. Impervious areas are themselves divided into two subareas – one that contains depression storage and another that does not. Runoff flow from one subarea in a subcatchment can be routed to the other subarea, or both subareas can drain to the subcatchment outlet. Infiltration of rainfall from the pervious area of a subcatchment into the unsaturated upper soil zone can be described using Green-Ampt Infiltration model.

As indicated in Chapter IV, the three model parameters, namely hydraulic conductivity, suction head, and initial soil moisture content were taken as: 10.92 mm/hr, 109.98 mm, 0.263 m/m, respectively. These values were derived from the data given in ***Table 4.8.*** The infiltration losses occurring in sample subwatersheds (S1 to S5) due to the occurrence of the above described design storm are shown in ***FIG. 5.3.*** As seen, the shape of the graphs resemble with that of the design storm, implying that the larger the amount of rainfall intensity, the larger the infiltration losses, and vice versa.

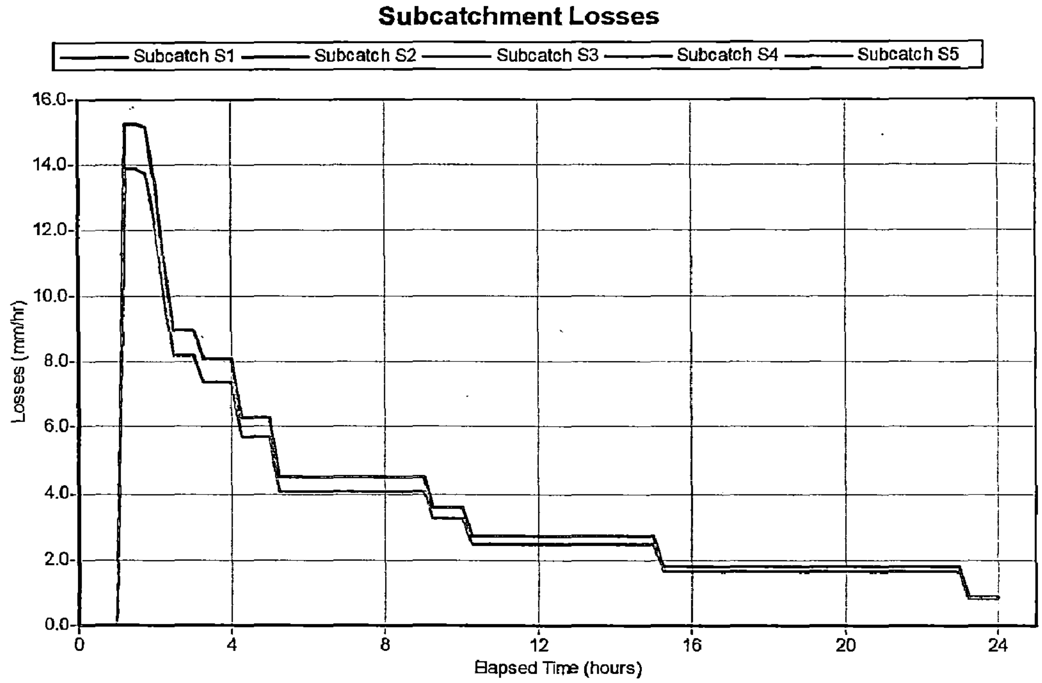


FIG. 5.3. Subcatchment Losses

Computation of Runoff at the Outlets of Subwatersheds

For the computation of runoff from each subwatershed, the infiltration losses were first subtracted from the rainfall to obtain the rainfall-excess. This rainfall excess was then subjected to overland flow by kinematic wave approach as described in Chapter III. The relevant data for employing the kinematic wave approach are shown in ***Table 4.7.*** The runoff hydrographs resulting at the outlets of the subcatchments (S1-S5) are shown in ***FIG. 5.4.*** As an example, the peak of discharge hydrograph appearing at the outlet of S4 subwatershed is of the order of 2.5 m^3/s (or CMS). To compare, this quantity of runoff would correspond to 0.79 mm/hr. When compared with the peak rainfall intensity, which is of the order of 27.79 mm/hr, the resulting runoff hydrograph in mm/hr is only 0.79, which is too low, indicating most water losing as infiltration. ***FIG. 5.3.*** is indicative of the amount of infiltration loss (i.e. of the order of 15 mm/hr) at the peak rainfall intensity. The corresponding depths and heads of runoff water at the outlets of the S1-S5 subwatersheds are shown in ***FIG. 5.5. and FIG. 5.6.***, respectively. The difference in the shapes of the hydrographs for a watershed, which is the same, is largely due to the addition of significantly varying invert elevations of the associated junctions and their plotting on the same graph.

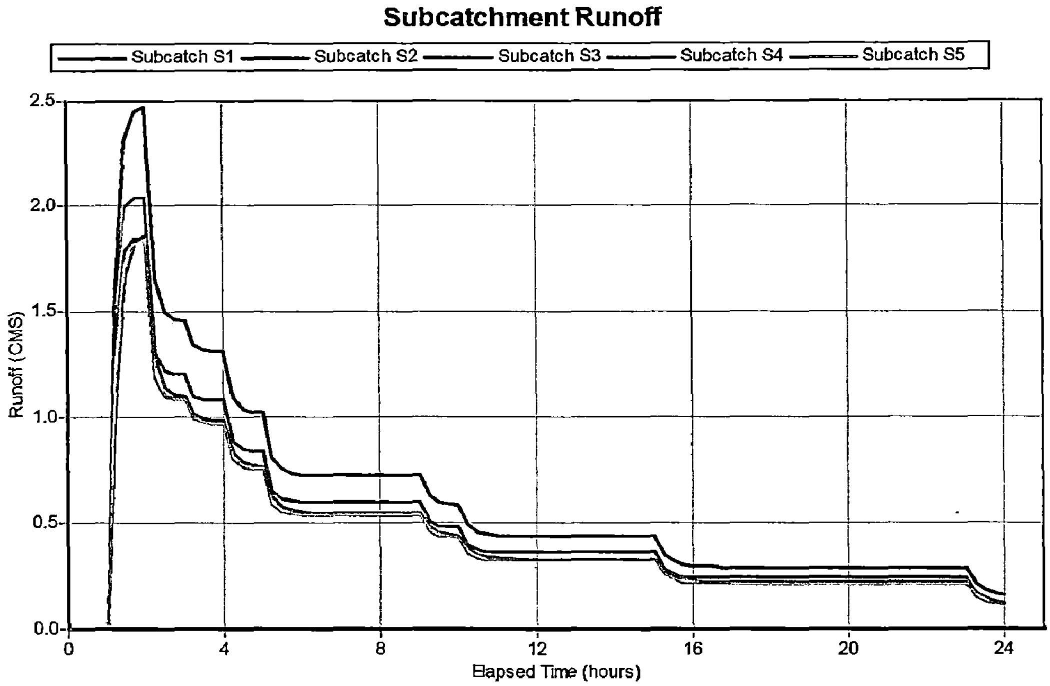


FIG. 5.4. Subcatchment Runoff

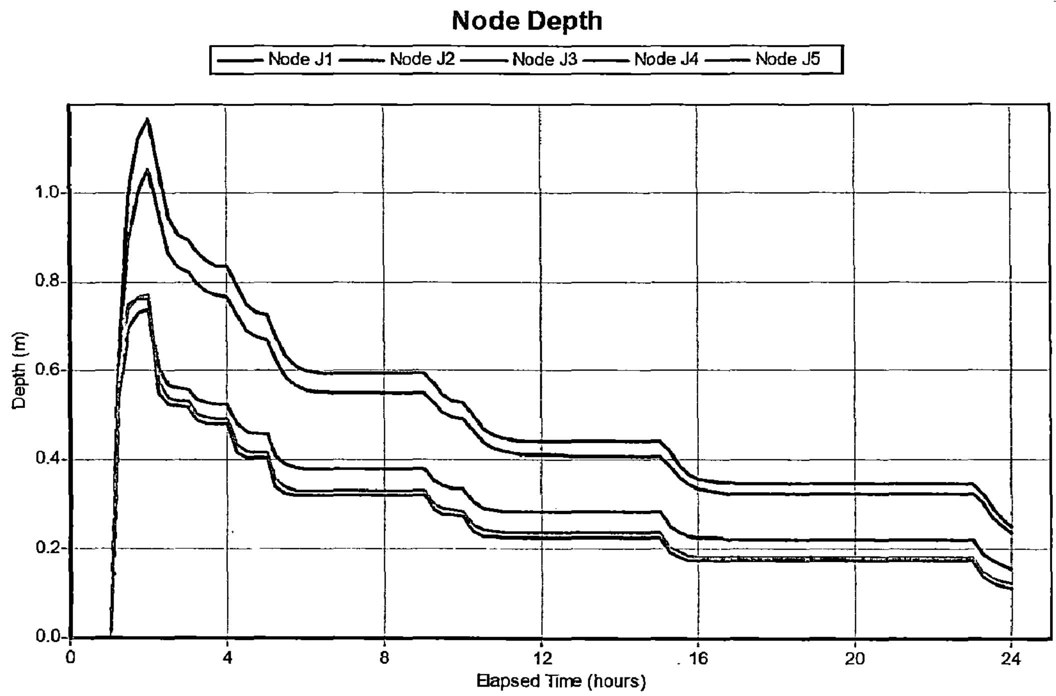


FIG. 5.5. Node Depth

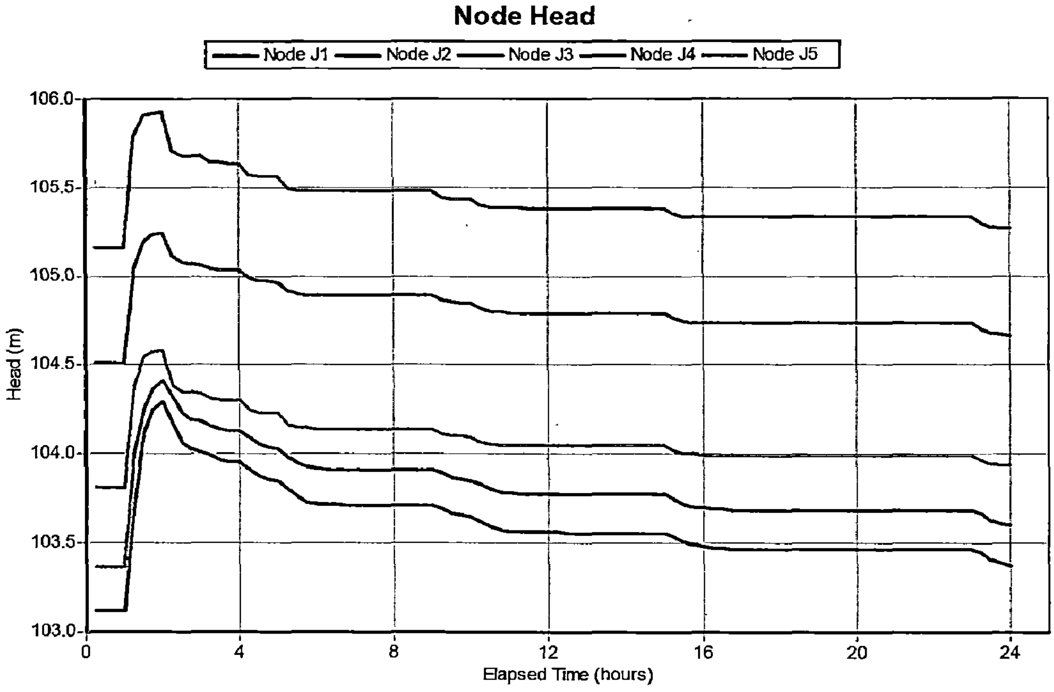


FIG. 5.6. Node Head

Flow through Conduit

The flow through conduit here actually refers to the surface drains, and not to the pipes. The sizes and shapes of these conduits are presented in ***Table 4.11.*** For computation of the final hydrograph resulting from the whole Sekanak Drainage System (***FIG 5.1.***), the flows appearing at various junctions were routed using the kinematic wave approach. Here, it is noted that if the flows at a junction are joining from different subwatersheds, these were summed up to obtain the total outflow from the junction, as shown in ***FIG. 5.7.***, for routing downstream using the kinematic wave approach.

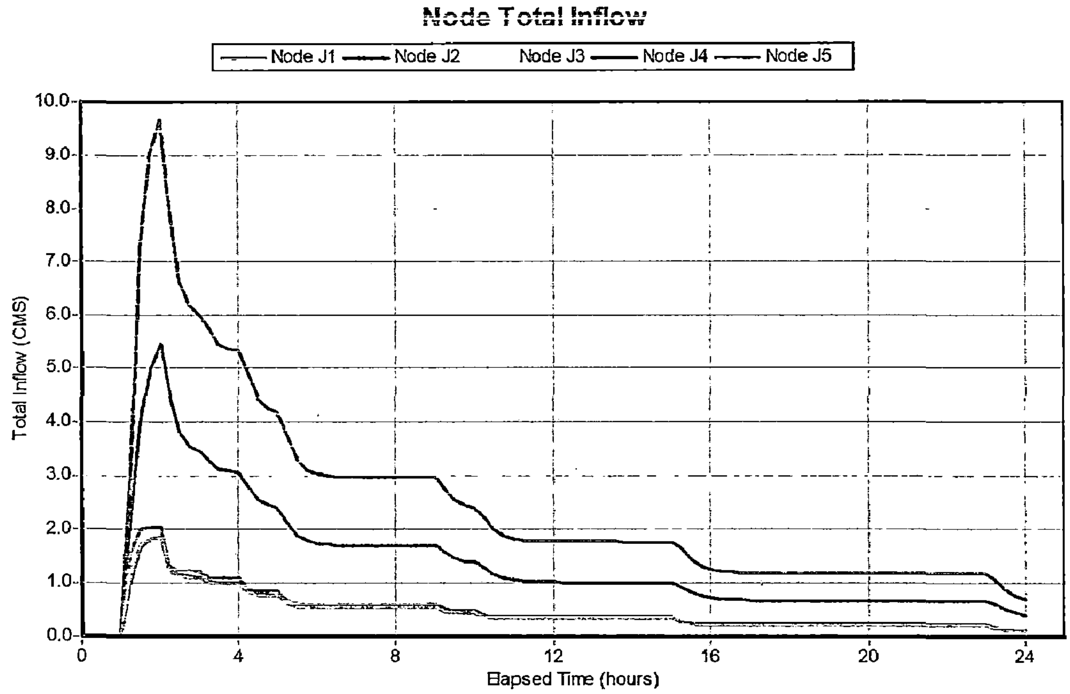


FIG. 5.7. Node Total Inflow

Computation of Flood Hydrograph

Employing the above scheme, the total or final flood hydrograph was computed at Conduit 14 and it is shown in ***FIG. 5.8.*** The corresponding stage hydrograph is shown in ***FIG. 5.9.***

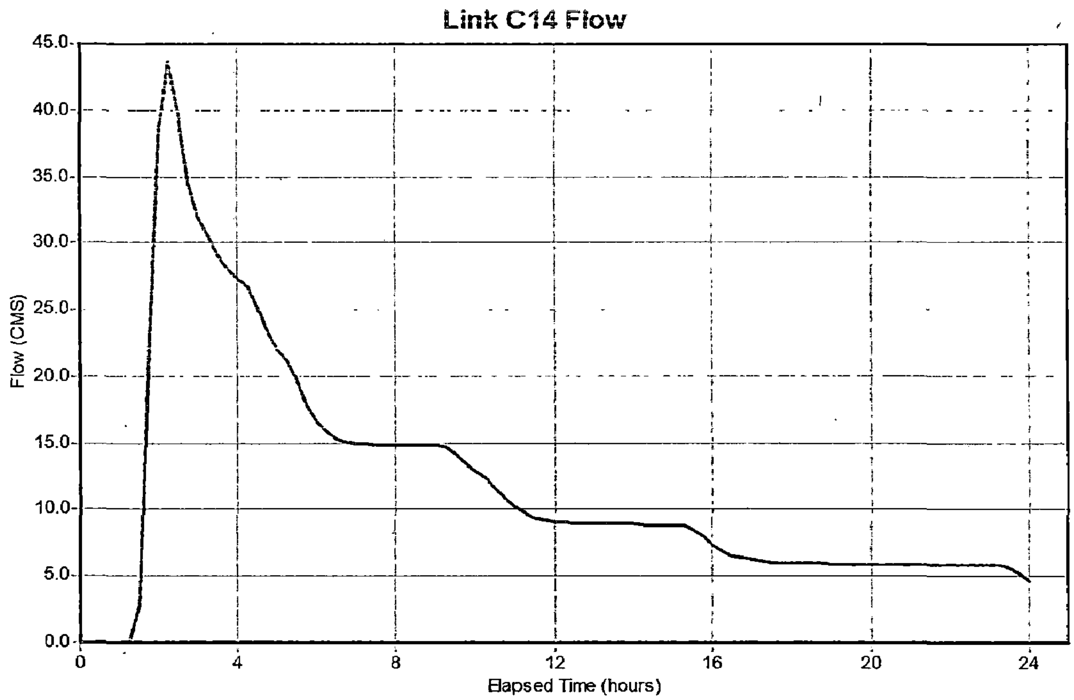


FIG. 5.8. Final Flood Hydrograph

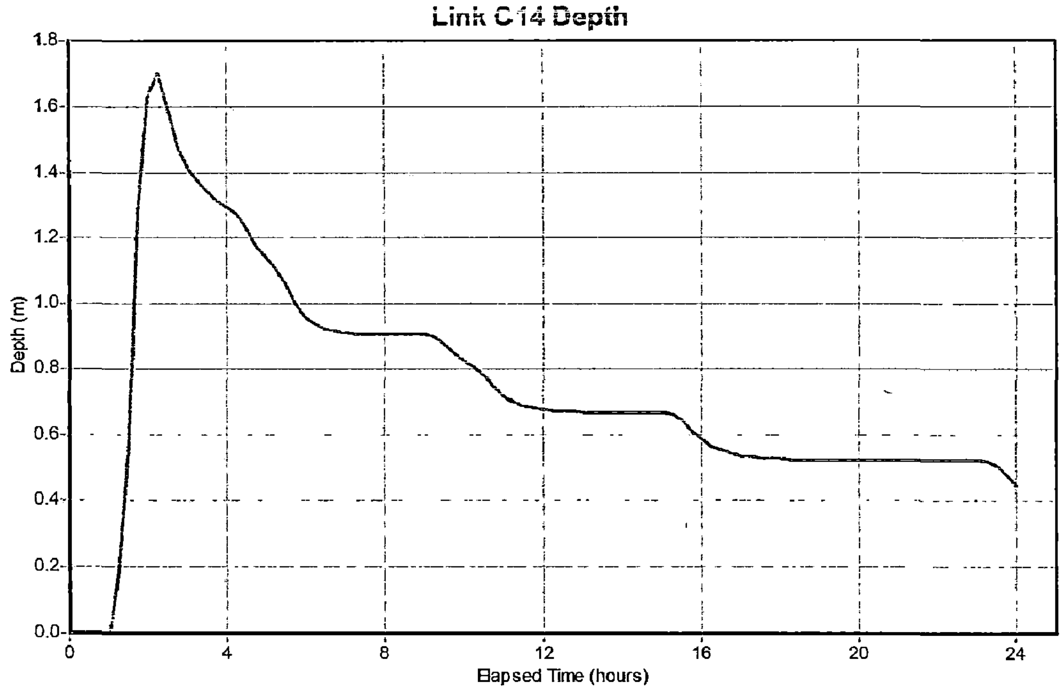


FIG. 5.9. Stage Hydrograph

Computation of Water Surface Profile along the Drainage System

For an assessment that the runoff at any stage overflows the banks of conduits, leading to flooding conditions in the drainage basin, water surface profile for the drainage path was computed and it is shown ***FIG. 5.10.*** In this figure, maximum depths available for water storage in the conduit at different junctions is plotted along with the water surface profile. At the outlet of the watershed, i.e. out1, it is seen that the depth abruptly changes to the maximum depth. It is due to the provision made in the model to opt for water accumulation and for water loss if the runoff stage exceeds the maximum water depth provided. Here, the water gets accumulated at out1 because there is no further outlet for release of water, leading to rise in the depth of flow at out1. Since the accumulated water exceeded the maximum water depth at out1, the water profile assumed the maximum water depth. Thus, the water surface profile being much below the maximum water depth for the considered 50-yr return period flood is indicative of the ample drainage capacity to handle the considered design flood. The foregoing analysis is to show the capacity of the existing drainage system to handle the level of return period flood and it is analyzed through a sensitivity analysis discussed later.

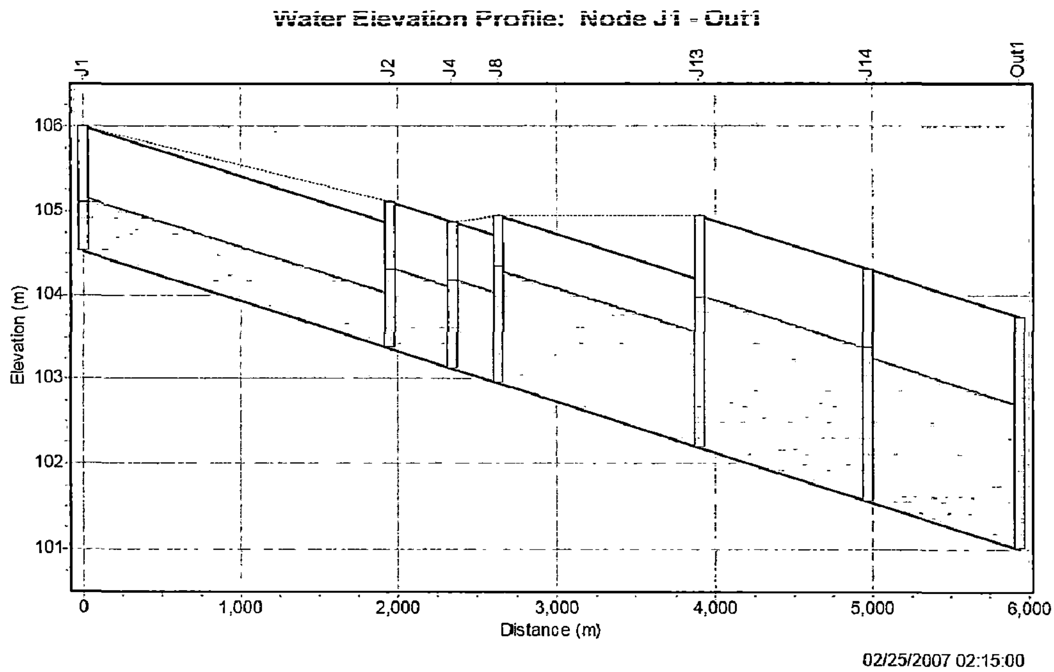


FIG. 5.10. Water Elevation Profile

Computation of Flow Velocity in Conduit Flow

Computation of flow velocity in SWMM is important for ascertaining the maximum and minimum flow velocity in storm sewer design, for avoiding, respectively, the bed scour and deposition of the matter being transported. Here, it has of significance in the sense to ascertain the maximum scour velocity. As seen from ***FIG. 5.11,*** the velocities in conduits C1-C5 range from 0 – 1.8 m/s, which are less than 3 m/s, and reasonable for transport of water without any damage to drainage works.

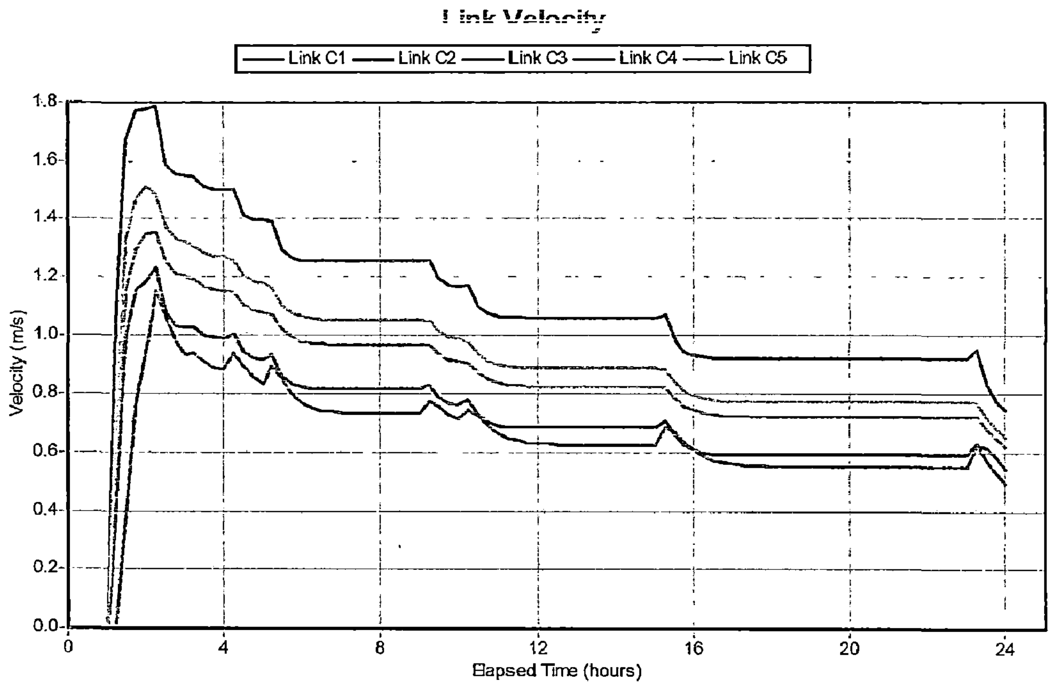


FIG. 5.11. Link Velocity

Checking of Water Balance

To check the model workability for the water balance, computations were made for the total precipitation, infiltration loss, and the surface runoff. The results are summarized in ***Table 5.2.*** It is seen from this table that the relative errors range from 0.026 to 1.973, which are not beyond tolerance, indicating successful model run.

Table. 5.2. **Water Balance Computation**

Subcatchment	Total Precipitation (mm)	Total Infiltration (mm)	Total Runoff (mm)	Runoff Coeff.	%error*
1	2	3	4	5	6
S1	161.800	88.990	72.405	0.447	0.250
S2	161.800	88.990	72.653	0.449	0.097
S3	161.800	88.990	72.684	0.449	0.078
S4	161.800	80.900	80.588	0.498	0.193
S5	161.800	80.900	80.612	0.498	0.178
S6	161.800	88.990	72.768	0.450	0.026
S7	161.800	80.900	80.528	0.498	0.230
S8	161.800	56.630	102.707	0.635	1.522
S9	161.800	88.990	72.670	0.449	0.087
S10	161.800	56.630	104.447	0.646	0.447
S11	161.800	56.630	104.261	0.644	0.562
S12	161.800	48.540	110.068	0.680	1.973
S13	161.800	56.630	104.249	0.644	0.569
S14	161.800	56.630	104.406	0.645	0.472
Totals	**161.800**	**69.190**	**91.664**	**0.567**	**0.585**

* Error in mass conservation = (1-(Infiltration+Runoff)/Precipitation)*100

Sensitivity Analysis

To quantify the level or magnitude of flood and the corresponding return period, a sensitivity analysis was carried out. To this end, the rainfall time series (TS) was multiplied by a factor greater than 1, as shown in ***Table 5.3.***, and different runs were taken for the water surface profile derivations. By increasing the time series by 1.4 (or +40% as shown in table), the flooding was seen in different parts of the flow path and it is shown in ***FIG. 5.12.*** Thus, it follows that the drainage system will be under flood only when the actual design storm exceeds by 40% and it is safe otherwise.

Table. 5.3. Time Series Data

Trial Simulation for Check Flooding in Sekanak Drainage System by Increase Time Series/ Rainfall Data, 10% - 40%.

Hour	TS	TS+10%	TS+20%	TS+30%	TS+40%
1	27.79	30.57	33.35	36.13	**38.91**
2	16.35	17.99	19.62	21.26	**22.89**
3	14.71	16.18	17.65	19.12	**20.59**
4	11.44	12.58	13.73	14.87	**16.02**
5	8.17	8.99	9.80	10.62	**11.44**
6	8.17	8.99	9.80	10.62	**11.44**
7	8.17	8.99	9.80	10.62	**11.44**
8	6.54	7.19	7.85	8.50	**9.16**
9	4.90	5.39	5.88	6.37	**6.86**
10	4.90	5.39	5.88	6.37	**6.86**
11	4.90	5.39	5.88	6.37	**6.86**
12	4.90	5.39	5.88	6.37	**6.86**
13	4.90	5.39	5.88	6.37	**6.86**
14	4.90	5.39	5.88	6.37	**6.86**
15	3.27	3.60	3.92	4.25	**4.58**
16	3.27	3.60	3.92	4.25	**4.58**
17	3.27	3.60	3.92	4.25	**4.58**
18	3.27	3.60	3.92	4.25	**4.58**
19	3.27	3.60	3.92	4.25	**4.58**
20	3.27	3.60	3.92	4.25	**4.58**
21	3.27	3.60	3.92	4.25	**4.58**
22	3.27	3.60	3.92	4.25	**4.58**
23	1.63	1.79	1.96	2.12	**2.28**
24	1.63	1.79	1.96	2.12	**2.28**

Note: TS = Original Time Series Data
TS+10% = Original Time Series Data + 10% increase
TS+20% = Original Time Series Data + 20% increase
TS+30% = Original Time Series Data + 30% increase
TS+40% = Original Time Series Data + 40% increase

Trial Simulation for Check Flooding in Sekanak Drainage System.

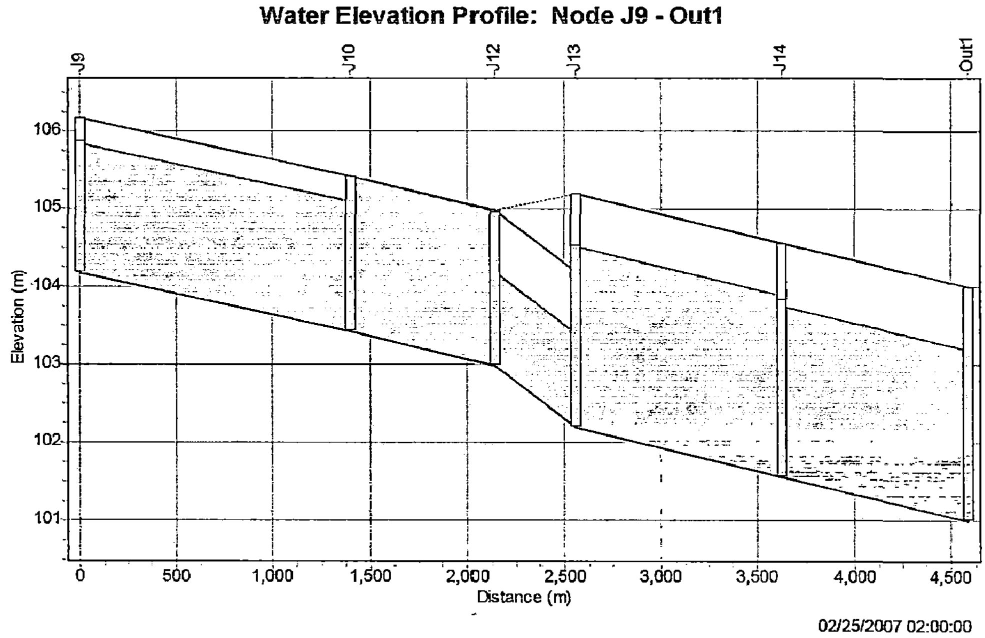


FIG. 5.12. Water Elevation Profile for Sensitivity Analysis.

- Water elevation profile (*Figure 5.12*) for link J9 – J10 – J12 – J13 – J14 – Out1.
- Flooding occur for the first time in the study area when rainfall data increase up to 35%, in J13 and J14, total minutes flooded only 1 minutes and 2 minutes respectively, total volume of flooding nil in both Junction, J13 and J14, no surcharge in all conduits.
- When rainfall data increase up to 40%, in J10, J12 and J13, total minutes flooded are 6 minutes, 2 minutes and 2 minutes respectively. Total volume of flooding are 2.21 ha-mm for J10 and nil for J12 and J13. Surcharge occur at C10 (2 minutes).
- The status report of this trial are shown in ***Annexure 6.***

CHAPTER VI

SUMMARY & CONCLUSIONS

The Environmental Protection Agency's Storm Water Management Model (SWMM) has seen myriad applications world over, specifically in sewer and stormwater studies. Its typical applications include the design and sizing of drainage system components for flood control, sizing of detention facilities and their appurtenances for flood control and water quality protection, flood plain mapping of natural channel system, designing control strategies for minimizing combined sewer overflows, evaluating the impact of outflow and infiltration on sanitary sewer overflows, generating non-point source pollutant loadings for waste load allocation studies, evaluating the effectiveness of Best Management practices (BMP) for reducing wet weather pollutant loadings. In this study, the Sekanak Drainage System located in Palembang City (Indonesia) taken up for evaluating the efficacy of the system using the SWMM. The following can be derived from the present study:

1. The EPA SWMM can be applied for the study of flooding in a drainage basin. In other words, the model has potential to evaluate the efficacy of the drainage system.

2. Storm sewers are usually designed to handle peak flow corresponding to 10 year return period. However in consideration of the importance of particular urban areas, higher return period may be adopted for design storm. Thus storm runoff drainage in Sekanak Drainage System has been checked for 50 year flood.

3. The existing Sekanak Drainage System located is capable of handling a flood that corresponds to 50-yr return period design storm. The system will be under flood only when the rainstorm exceeds the design storm by about 40%, it is other protected from rainfall-generated floods.

4. The dimension of open channel drainage (rectangular and trapezoidal) in Sekanak Drainage System are satisfied and economical.

5. The study provides for the first time a detailed simulation study of urban drainage of Sekanak Drainage System in Palembang City. It is possible to make further improvements by increasing the number of subcatchments (through subdivision) and incorporating more accurate properties of subcatchments and channels based on field measurements. Further, the drainage system in Sekanak System should be simulated as combined sewer system because it carries domestic sewage also.

REFERENCES

1. Akan, A. Osman, and Houghtalen, Robert J, 2003, ***"Urban Hydrology, Hydraulics, and Stormwater Quality"***, Engineering Applications and Computer Modeling, John Wiley & Sons, Inc., Hoboken, New Jersey.

2. Akan, A. Osman, Ph.D., P.E, 1993, ***"Urban Stormwater Hydrology"***, A Guide to Engineering Calculations, Technomic Publishing Company Inc., Lancaster, Pennsylvania 17604 U.S.A.

3. Hall, M.J., 1984 ***"Urban Hydrology"***, Elsevier Applied Science Publishers Ltd., London and New York.

4. Kartika, Endra, 2005 ***"Runoff Estimation in Urban Catchment"***, Seminar Report, WRD&M, Indian Institute of Technology Roorkee.

5. Kartika, Endra, 2006 ***"Urban Flood Drainage Planning"***, A Dissertation, WRD&M, Indian Institute of Technology Roorkee.

6. ***"Kinematic Wave Method for Peak Runoff Estimates"***, Journal of Transportation Engineering, 111(4), ASCE.

7. Lazaro, Timothy R., 1993 ***"Urban Hydrology"***, A Multidisciplinary Perspective, Revised Edition, Technomic Publishing Company Inc., Lancaster, Basel, Pennsylvania 17604 U.S.A.

8. Maidment, D.R., 1992, ***"Handbook of Hydrology"***, McGraw Hill, New York.

9. N. Narayana Pillai and C.R. Ramakhrisna, 2006, ***"Principles of Fluid Mechanics and Fluid Machines"***, Second Edition, Universities Press, Engineering.

10. Rossman, L.A., October 2005, ***"Storm Water Management Model User's Manual Version 5"***, United State Environmental Protection Agency, EPA/160/R-05/040.

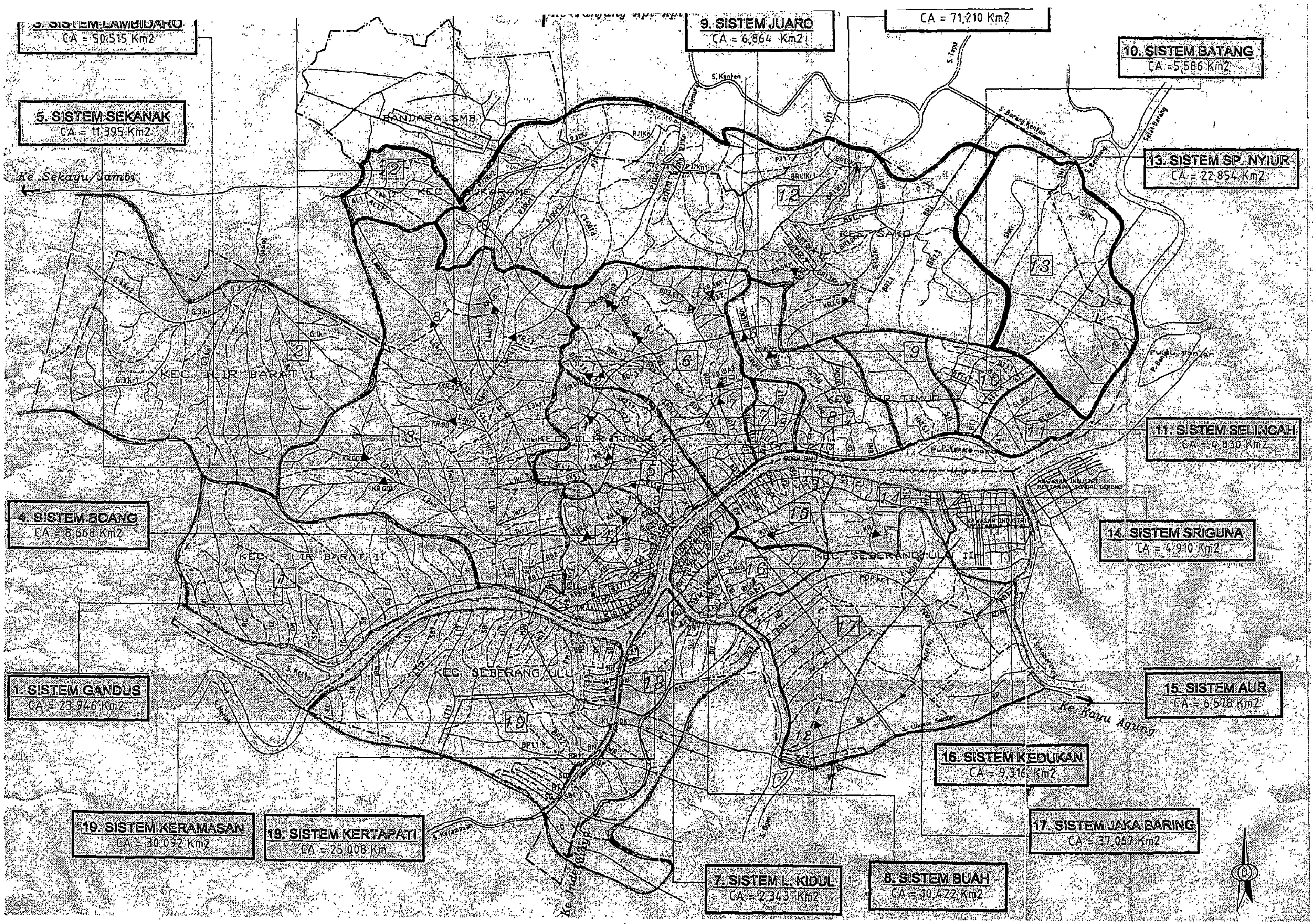
11. ***"Similarity Solution of Overland Flow on Pervious Surface"***, Journal of Hydraulic Engineering, 111(7), ASCE.

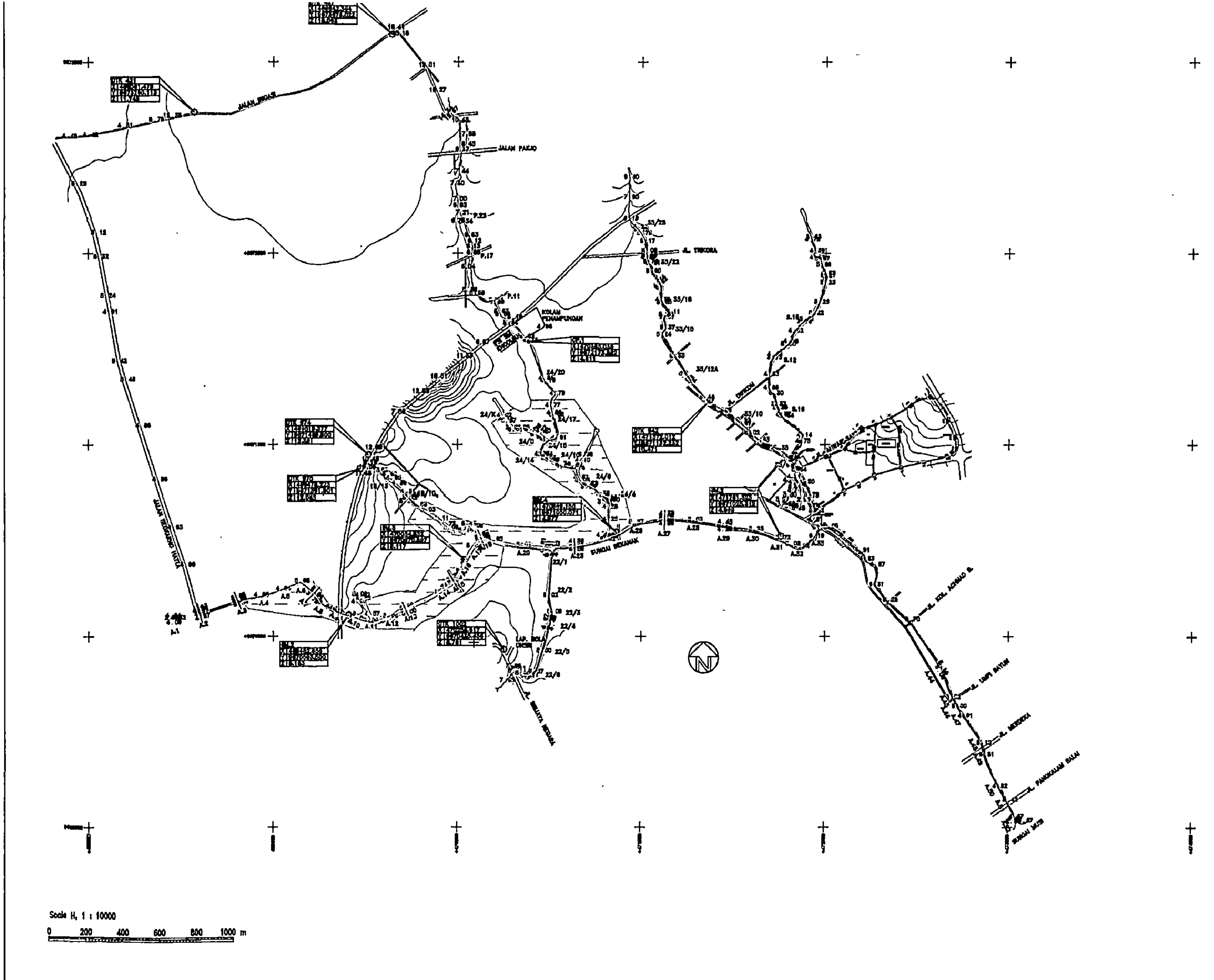
12. Subramaya, K, 1994, ***"Engineering Hydrology"***, Tata McGraw Hill, New Delhi.

13. Wurbs, Ralph A., and James, Wesley P., 2002, ***"Water Resources Engineering"***, Prentice-Hall of India, Private Limited, New Delhi.

ANNEXURES

1. Drainage System in Palembang City
2. Contour Map of Sekanak Drainage System
3. Sekanak Drainage System-SWMM Input Data
4. Sekanak Drainage System-SWMM Output/ Status Report
5. Sensitivity Analysis for Check Flooding in Sekanak Drainage System-SWMM Input Data
6. Sensitivity Analysis for Check Flooding in Sekanak Drainage System-SWMM Output/ Status Report


3. SISTEM LAMBIDARO
CA = 50,515 Km2
9. SISTEM JUARO
CA = 6,864 Km2
CA = 71,210 Km2
10. SISTEM BATANG
CA = 5,586 Km2
5. SISTEM SEKANAK
CA = 11,395 Km2
13. SISTEM SP. NYIUR
CA = 22,854 Km2
Ke. Sekayu/Jambi
BANDARA SMB
KEC. SUKARAMI
KEC. SAKO
KEC. ILIR BARAT I
KEC. ILIR TIMUR
11. SISTEM SELINCAH
CA = 4,830 Km2
4. SISTEM BOANG
CA = 8,668 Km2
14. SISTEM SRIGUNA
CA = 4,910 Km2
KEC. ILIR BARAT II
KEC. SEBERANG ULU II
KEC. SEBERANG ULU I
1. SISTEM GANDUS
CA = 23,946 Km2
15. SISTEM AUR
CA = 6,578 Km2
Ke Kayu Agung
16. SISTEM KEDUKAN
CA = 9,316 Km2
19. SISTEM KERAMASAN
CA = 30,092 Km2
18. SISTEM KERTAPATI
CA = 25,008 Km
17. SISTEM JAKA BARING
CA = 37,067 Km2
7. SISTEM L. KIDUL
CA = 2,343 Km2
8. SISTEM BUAH
CA = 30,422 Km2



KECAMATAN : ILIR BARAT I DAN II
KOTA : PALEMBANG
PROPINSI : SUMATERA SELATAN

LEGENDA

NO	TANGGAL	PERUBAHAN	PARAF

PEMBERI TUGAS

PEMERINTAH KOTA PALEMBANG
BADAN PERENCANAAN PEMBANGUNAN DAERAH
(BAPPEDA)

PEKERJAAN :
Perencanaan Master Plan Drainase dan Detail desain Drainase DAS Sungai Sekanak, Kota Palembang

MENGETAHUI
KEPALA BAPPEDA KOTA PALEMBANG

Toni K. Panggarbeni, SH, MSi

DIPERIKSA	DISETUJUI
	PEMIMPIN PROYEK
Ir. R. M. Cahyono	Yulianto, ST

GAMBAR
PETA SITUASI TRACE
SISTEN DRAINASE SUNGAI SEKANAK

SKALA 1 : 1000

DIGAMBAR	NOMOR
PERENCANA	SK-009
TANGGAL	
DIPERIKSA	
DISETUJUI	REVISI

KONSULTAN TEKNIK
LAPI ITB
LEMBAGA AFILIASI PENELITIAN DAN INDUSTRI - ITB

Simulation Study Of Sekanak Drainage System in Palembang City.

```
[TITLE]
Simulation Study Of Sekanak Drainage System in Palembang City.

[OPTIONS]
 FLOW_UNITS             CMS
 INFILTRATION           GREEN_AMPT
 FLOW_ROUTING           KINWAVE
 START_DATE             02/25/2007
 START_TIME             00:00:00
 REPORT_START_DATE      02/25/2007
 REPORT_START_TIME      00:00:00
 END_DATE               02/26/2007
 END_TIME               00:00:00
 SWEEP_START            01/01
 SWEEP_END              12/31
 DRY_DAYS               0
 REPORT_STEP            00:15:00
 WET_STEP               00:15:00
 DRY_STEP               00:15:00
 ROUTING_STEP           0:00:30
 ALLOW_PONDING          NO
 INERTIAL_DAMPING       PARTIAL
 VARIABLE_STEP          0.75
 LENGTHENING_STEP       0
 MIN_SURFAREA           0
 NORMAL_FLOW_LIMITED    NO
 SKIP_STEADY_STATE      NO

[RAINGAGES]
;;                 Rain      Recd.  Snow   Data       Source           Station    Rain
;;Name             Type      Freq.  Catch  Source     Name             ID         Units
;;----------------------------------------------------------------------------------------
 Gage1             INTENSITY 1:00   1.0    TIMESERIES TS1

[SUBCATCHMENTS]
;;                                                  Total    Pcnt.             Pcnt.    Curb     Snow
;;Name             Raingage         Outlet          Area     Imperv   Width    Slope    Length   Pack
;;------------------------------------------------------------------------------------------------------
 S1                Gage1            J1              53.59    45       1317.68  0.49     0
 S2                Gage1            J2              53.07    45       1592.96  1.04     0
 S3                Gage1            J3              58.78    45       1365.06  2.12     0
 S4                Gage1            J4              64.05    50       1803.25  0.77     0
 S5                Gage1            J5              47.50    50       1210.26  1.04     0
 S6                Gage1            J6              50.15    45       1438.62  2.89     0
 S7                Gage1            J7              77.53    50       2129.89  0.64     0
 S8                Gage1            J8              139.07   65       1453.85  0.28     0
 S9                Gage1            J9              107.04   45       2420.70  2.04     0
 S10               Gage1            J10             90.89    65       1822.51  1.09     0
 S11               Gage1            J11             82.20    65       2305.23  0.37     0
 S12               Gage1            J12             88.65    70       1564.77  0.07     0
 S13               Gage1            J13             138:30   65       2173.63  1.15     0
 S14               Gage1            J14             88.68    65       1992.05  0.79     0
```

[SUBAREAS]

;;Subcatchment	N-Imperv	N-Perv	S-Imperv	S-Perv	PctZero	RouteTo	PctRouted
S1	0.014	0.13	0	2.54	100	OUTLET	
S2	0.014	0.13	0	2.54	100	OUTLET	
S3	0.014	0.13	0	2.54	100	OUTLET	
S4	0.014	0.13	0	2.54	100	OUTLET	
S5	0.014	0.13	0	2.54	100	OUTLET	
S6	0.014	0.13	0	2.54	100	OUTLET	
S7	0.014	0.13	0	2.54	100	OUTLET	
S8	0.014	0.13	0	2.54	100	OUTLET	
S9	0.014	0.13	0	2.54	100	OUTLET	
S10	0.014	0.13	0	2.54	100	OUTLET	
S11	0.014	0.13	0	2.54	100	OUTLET	
S12	0.014	0.13	0	2.54	100	OUTLET	
S13	0.014	0.13	0	2.54	100	OUTLET	
S14	0.014	0.13	0	2.54	100	OUTLET	

[INFILTRATION]

;;Subcatchment	Suction	HydCon	IMDmax
S1	109.98	10.92	0.263
S2	109.98	10.92	0.263
S3	109.98	10.92	0.263
S4	109.98	10.92	0.263
S5	109.98	10.92	0.263
S6	109.98	10.92	0.263
S7	109.98	10.92	0.263
S8	109.98	10.92	0.263
S9	109.98	10.92	0.263
S10	109.98	10.92	0.263
S11	109.98	10.92	0.263
S12	109.98	10.92	0.263
S13	109.98	10.92	0.263
S14	109.98	10.92	0.263

[JUNCTIONS]

;; ;;Name	Invert Elev.	Max. Depth	Init. Depth	Surcharge Depth	Ponded Area
J1	104.51	1.5	0	0	0
J2	103.36	1.75	0	0	0
J3	105.16	1.5	0	0	0
J4	103.12	1.75	0	0	0
J5	103.81	1.5	0	0	0
J6	103.59	1.5	0	0	0
J7	103.29	2	0	0	0
J8	102.95	2	0	0	0
J9	104.18	2	0	0	0
J10	103.43	2.25	0	0	0
J11	104.71	1.75	0	0	0

```
  J12              102.98     1.75      0          0          0
  J13              102.19     2.75      0          0          0
  J14              101.56     2.75      0          0          0

[OUTFALLS]
;;                 Invert     Outfall   Stage/Table      Tide
;;Name             Elev.      Type      Time Series      Gate
;;-------------------------------------------------------------
;Outfall
  Out1             100.99     FREE                       NO

[CONDUITS]
;;                 Inlet            Outlet                        Manning    Inlet      Outlet     Init.      Maximum
;;Name             Node             Node             Length       N          Height     Height     Flow       Flow
;;----------------------------------------------------------------------------------------------------------------------
;Sekanak Drainage A1-A18
  C1               J1               J2               1942.53      0.014      0          0          0          0
;Bukit Lama Drainage 18/14-A.18
  C2               J3               J2               791.91       0.014      0          0          0          0
;Drainage Sekanak A18-A22
  C3               J2               J4               401.28       0.014      0          0          0          0
;Kampus Drainage 22/10-A22
  C4               J5               J4               821.72       0.014      0          0          0          0
  C5               J4               J8               291.44       0.014      0          0          0          0
;Khodijah Drainage P31-P11
  C6               J6               J7               1855.41      0.014      0          0          0          0
;Khodijah Drainage P11-A24
  C7               J7               J8               1133.06      0.014      0          0          0          0
;Sekanak Drainage A24-A35
  C8               J8               J13              1266.19      0.014      0          0          0          0
;Baung Drainage 35/31-35/12A
  C9               J9               J10              1400.76      0.014      0          0          0          0
;Baung Drainage 35/12A-35/6
  C10              J10              J12              735.17       0.014      0          0          0          0
;Tridinanti Drainage
  C11              J11              J12              1471.38      0.014      0          0          0          0
;Baung Drainage 35/6-A35
  C12              J12              J13              418.37       0.014      0          0          0          0
;Sekanak Drainage A35-A44
  C13              J13              J14              1068.38      0.014      0          0          0          0
;Sekanak Drainage A44-Out1
  C14              J14              Out1             968.02       0.014      0          0          0          0

[XSECTIONS]
;;Link             Type         Geom1      Geom2      Geom3      Geom4      Barrels
;;--------------------------------------------------------------------------------
  C1               TRAPEZOIDAL  1.5        1.5        1.25       1.25       1
  C2               RECT_OPEN    1.5        1.5        0          0          1
  C3               TRAPEZOIDAL  1.75       2.5        1.25       1.25       1
  C4               RECT_OPEN    1.5        2          0          0          1
  C5               TRAPEZOIDAL  1.75       4          1.25       1.25       1
  C6               RECT_OPEN    1.5        2.5        0          0          1
```

```
C7              RECT_OPEN    2      3      0      0      1
C8              TRAPEZOIDAL  2      6      1.25   1.25   1
C9              RECT_OPEN    2      2.5    0      0      1
C10             RECT_OPEN    2.25   3.5    0      0      1
C11             RECT_OPEN    1.75   2.5    0      0      1
C12             TRAPEZOIDAL  1.75   6      1.25   1.25   1
C13             TRAPEZOIDAL  2.75   8      1.25   1.25   1
C14             TRAPEZOIDAL  2.75   10     1.25   1.25   1

[TIMESERIES]
;;Name          Date        Time        Value
;;-------------------------------------------------
;Kenten Station
 TS1                        1           27.79
 TS1                        2           16.35
 TS1                        3           14.71
 TS1                        4           11.44
 TS1                        5           8.17
 TS1                        6           8.17
 TS1                        7           8.17
 TS1                        8           8.17
 TS1                        9           6.54
 TS1                        10          4.90
 TS1                        11          4.90
 TS1                        12          4.90
 TS1                        13          4.90
 TS1                        14          4.90
 TS1                        15          3.27
 TS1                        16          3.27
 TS1                        17          3.27
 TS1                        18          3.27
 TS1                        19          3.27
 TS1                        20          3.27
 TS1                        21          3.27
 TS1                        22          3.27
 TS1                        23          1.63
 TS1                        24          1.63

[REPORT]
 INPUT      NO
 CONTROLS   NO

[OPTIONS]
 TEMPDIR    "C:\DOCUME~1\User\LOCALS~1\Temp\"
```

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0 (Build 5.0.006a)
--------------------------------------------------------------

Simulation Study Of Sekanak Drainage System in Palembang City.

****************
Analysis Options
****************
Flow Units ............... CMS
Infiltration Method ...... GREEN_AMPT
Flow Routing Method ...... KINWAVE
Starting Date ............ FEB-25-2007 00:00:00
Ending Date .............. FEB-26-2007 00:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:15:00
Wet Time Step ............ 00:15:00
Dry Time Step ............ 00:15:00
Routing Time Step ........ 30.00 sec


**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Total Precipitation ......       184.371       161.800
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........        78.842        69.190
Surface Runoff ...........       104.451        91.664
Final Surface Storage ....         1.379         1.210
Continuity Error (%) .....        -0.163


**************************        Volume        Volume
Flow Routing Continuity        hectare-m       Mliters
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......       104.451      1044.521
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........       103.213      1032.141
Surface Flooding .........         0.000         0.000
Evaporation Loss .........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         1.748        17.480
Continuity Error (%) .....        -0.488


***************************
Subcatchment Runoff Summary
***************************

---------------------------------------------------------------------------
                         Total      Total      Total      Total      Total     Runoff
                        Precip      Runon       Evap      Infil     Runoff      Coeff
Subcatchment                mm         mm         mm         mm         mm
---------------------------------------------------------------------------
S1                     161.800      0.000      0.000     88.990     72.405      0.447
S2                     161.800      0.000      0.000     88.990     72.653      0.449
S3                     161.800      0.000      0.000     88.990     72.684      0.449
S4                     161.800      0.000      0.000     80.900     80.588      0.498
S5                     161.800      0.000      0.000     80.900     80.612      0.498
S6                     161.800      0.000      0.000     88.990     72.768      0.450
S7                     161.800      0.000      0.000     80.900     80.528      0.498
S8                     161.800      0.000      0.000     56.630    102.707      0.635
S9                     161.800      0.000      0.000     88.990     72.670      0.449
S10                    161.800      0.000      0.000     56.630    104.447      0.646
S11                    161.800      0.000      0.000     56.630    104.261      0.644
S12                    161.800      0.000      0.000     48.540    110.068      0.680
S13                    161.800      0.000      0.000     56.630    104.249      0.644
S14                    161.800      0.000      0.000     56.630    104.406      0.645
---------------------------------------------------------------------------
Totals                 161.800      0.000      0.000     69.190     91.664      0.567
```

****
Node Depth Summary
****

Node	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days	hr:min	Total Flooding ha-mm	Total Minutes Flooded
J1	0.31	0.74	105.25	0	02:00	0	0
J2	0.46	1.06	104.42	0	02:01	0	0
J3	0.26	0.76	105.92	0	02:00	0	0
J4	0.49	1.17	104.29	0	02:00	0	0
J5	0.27	0.77	104.58	0	02:00	0	0
J6	0.39	1.10	104.69	0	02:00	0	0
J7	0.53	1.43	104.72	0	02:00	0	0
J8	0.61	1.40	104.35	0	02:11	0	0
J9	0.43	1.24	105.42	0	02:00	0	0
J10	0.54	1.52	104.95	0	02:00	0	0
J11	0.35	0.98	105.69	0	02:00	0	0
J12	0.54	1.51	104.49	0	02:04	0	0
J13	0.80	1.84	104.03	0	02:05	0	0
J14	0.80	1.84	103.40	0	02:11	0	0
Out1	2.62	2.75	103.74	0	01:08	0	0

****
Conduit Flow Summary
****

Conduit	Maximum Flow CMS	Time of Max Occurrence days	hr:min	Maximum Velocity m/sec	Length Factor	Maximum /Design Flow	Total Minutes Surcharged
C1	1.79	0	02:11	1.15	1.00	0.24	0
C2	2.06	0	02:01	1.85	1.00	0.43	0
C3	5.47	0	02:04	1.37	1.00	0.38	0
C4	1.86	0	02:02	1.25	1.00	0.42	0
C5	9.66	0	02:02	1.52	1.00	0.48	0
C6	1.68	0	02:15	0.73	1.00	0.64	0
C7	4.21	0	02:11	1.04	1.00	0.63	0
C8	19.03	0	02:11	1.82	1.00	0.52	0
C9	3.80	0	02:05	1.34	1.00	0.55	0
C10	8.15	0	02:04	1.57	1.00	0.59	0
C11	4.03	0	02:04	1.73	1.00	0.46	0
C12	15.63	0	02:05	2.48	1.00	0.31	0
C13	40.27	0	02:11	2.15	1.00	0.49	0
C14	43.65	0	02:14	2.12	1.00	0.43	0

****
Routing Time Step Summary
****

```
Minimum Time Step           :   30.00 sec
Average Time Step           :   30.00 sec
Maximum Time Step           :   30.00 sec
Percent in Steady State     :    0.00
Average Iterations per Step :    1.07

Analysis begun on:  Sat Jun 02 17:55:10 2007
Total elapsed time: < 1 sec
```

Simulation Study Of Sekanak Drainage System in Palembang City.

```
[TITLE]
Simulation Study Of Sekanak Drainage System in Palembang City.

[OPTIONS]
FLOW_UNITS           CMS
INFILTRATION         GREEN_AMPT
FLOW_ROUTING         KINWAVE
START_DATE           02/25/2007
START_TIME           00:00:00
REPORT_START_DATE    02/25/2007
REPORT_START_TIME    00:00:00
END_DATE             02/26/2007
END_TIME             00:00:00
SWEEP_START          01/01
SWEEP_END            12/31
DRY_DAYS             0
REPORT_STEP          00:15:00
WET_STEP             00:15:00
DRY_STEP             00:15:00
ROUTING_STEP         0:00:30
ALLOW_PONDING        NO
INERTIAL_DAMPING     PARTIAL
VARIABLE_STEP        0.75
LENGTHENING_STEP     0
MIN_SURFAREA         0
NORMAL_FLOW_LIMITED  NO
SKIP_STEADY_STATE    NO

[RAINGAGES]
;;                  Rain      Recd.  Snow   Data       Source           Station    Rain
;;Name              Type      Freq.  Catch  Source     Name             ID         Units
;;--------------------------------------------------------------------------------------
  Gage1             INTENSITY 1:00   1.0    TIMESERIES TS1

[SUBCATCHMENTS]
;;                                                Total    Pcnt.             Pcnt.    Curb     Snow
;;Name              Raingage         Outlet          Area     Imperv   Width    Slope    Length   Pack
;;----------------------------------------------------------------------------------------------------
  S1                Gage1            J1              53.59    45       1317.68  0.49     0
  S2                Gage1            J2              53.07    45       1592.96  1.04     0
  S3                Gage1            J3              58.78    45       1365.06  2.12     0
  S4                Gage1            J4              64.05    50       1803.25  0.77     0
  S5                Gage1            J5              47.50    50       1210.26  1.04     0
  S6                Gage1            J6              50.15    45       1438.62  2.89     0
  S7                Gage1            J7              77.53    50       2129.89  0.64     0
  S8                Gage1            J8              139.07   65       1453.85  0.28     0
  S9                Gage1            J9              107.04   45       2420.70  2.04     0
  S10               Gage1            J10             90.89    65       1822.51  1.09     0
  S11               Gage1            J11             82.20    65       2305.23  0.37     0
  S12               Gage1            J12             88.65    70       1564.77  0.07     0
  S13               Gage1            J13             138.30   65       2173.63  1.15     0
  S14               Gage1            J14             88.68    65       1992.05  0.79     0
```

```
[SUBAREAS]
;;Subcatchment   N-Imperv   N-Perv   S-Imperv   S-Perv   PctZero   RouteTo   PctRouted
;;---------------------------------------------------------------------------------------
  S1              0.014      0.13     0          2.54     100       OUTLET
  S2              0.014      0.13     0          2.54     100       OUTLET
  S3              0.014      0.13     0          2.54     100       OUTLET
  S4              0.014      0.13     0          2.54     100       OUTLET
  S5              0.014      0.13     0          2.54     100       OUTLET
  S6              0.014      0.13     0          2.54     100       OUTLET
  S7              0.014      0.13     0          2.54     100       OUTLET
  S8              0.014      0.13     0          2.54     100       OUTLET
  S9              0.014      0.13     0          2.54     100       OUTLET
  S10             0.014      0.13     0          2.54     100       OUTLET
  S11             0.014      0.13     0          2.54     100       OUTLET
  S12             0.014      0.13     0          2.54     100       OUTLET
  S13             0.014      0.13     0          2.54     100       OUTLET
  S14             0.014      0.13     0          2.54     100       OUTLET

[INFILTRATION]
;;Subcatchment   Suction    HydCon   IMDmax
;;---------------------------------------------
  S1              109.98     10.92    0.263
  S2              109.98     10.92    0.263
  S3              109.98     10.92    0.263
  S4              109.98     10.92    0.263
  S5              109.98     10.92    0.263
  S6              109.98     10.92    0.263
  S7              109.98     10.92    0.263
  S8              109.98     10.92    0.263
  S9              109.98     10.92    0.263
  S10             109.98     10.92    0.263
  S11             109.98     10.92    0.263
  S12             109.98     10.92    0.263
  S13             109.98     10.92    0.263
  S14             109.98     10.92    0.263

[JUNCTIONS]
;;                Invert     Max.     Init.      Surcharge  Ponded
;;Name            Elev.      Depth    Depth      Depth      Area
;;------------------------------------------------------------------
  J1              104.51     1.5      0          0          0
  J2              103.36     1.75     0          0          0
  J3              108.16     1.5      0          0          0
  J4              103.12     1.75     0          0          0
  J5              103.81     1.5      0          0          0
  J6              103.59     1.5      0          0          0
  J7              103.29     2        0          0          0
  J8              102.95     2        0          0          0
  J9              104.18     2        0          0          0
  J10             103.43     2.25     0          0          0
  J11             104.71     1.75     0          0          0
```

```
  J12              102.98     1.75       0          0          0
  J13              102.19     2.75       0          0          0
  J14              101.56     2.75       0          0          0

[OUTFALLS]
;;                 Invert     Outfall    Stage/Table      Tide
;;Name             Elev.      Type       Time Series      Gate
;;-------------------------------------------------------------
;Outfall
  Out1             100.99     FREE                        NO

[CONDUITS]
;;                 Inlet            Outlet                    Manning    Inlet      Outlet     Init.      Maximum
;;Name             Node             Node             Length   N          Height     Height     Flow       Flow
;;---------------------------------------------------------------------------------------------------------------
;Sekanak Drainage A1-A18
  C1               J1               J2               1942.53  0.014      0          0          0          0
;Bukit Lama Drainage 18/14-A.18
  C2               J3               J2               791.91   0.014      0          0          0          0
;Drainage Sekanak A18-A22
  C3               J2               J4               401.28   0.014      0          0          0          0
;Kampus Drainage 22/10-A22
  C4               J5               J4               821.72   0.014      0          0          0          0
  C5               J4               J8               291.44   0.014      0          0          0          0
;Khodijah Drainage P31-P11
  C6               J6               J7               1855.41  0.014      0          0          0          0
;Khodijah Drainage P11-A24
  C7               J7               J8               1133.06  0.014      0          0          0          0
;Sekanak Drainage A24-A35
  C8               J8               J13              1266.19  0.014      0          0          0          0
;Baung Drainage 35/31-35/12A
  C9               J9               J10              1400.76  0.014      0          0          0          0
;Baung Drainage 35/12A-35/6
  C10              J10              J12              735.17   0.014      0          0          0          0
;Tridinanti Drainage
  C11              J11              J12              1471.38  0.014      0          0          0          0
;Baung Drainage 35/6-A35
  C12              J12              J13              418.37   0.014      0          0          0          0
;Sekanak Drainage A35-A44
  C13              J13              J14              1068.38  0.014      0          0          0          0
;Sekanak Drainage A44-Out1
  C14              J14              Out1             968.02   0.014      0          0          0          0

[XSECTIONS]
;;Link             Type         Geom1      Geom2      Geom3      Geom4      Barrels
;;--------------------------------------------------------------------------------
  C1               TRAPEZOIDAL  1.5        1.5        1.25       1.25       1
  C2               RECT_OPEN    1.5        1.5        0          0          1
  C3               TRAPEZOIDAL  1.75       2.5        1.25       1.25       1
  C4               RECT_OPEN    1.5        2          0          0          1
  C5               TRAPEZOIDAL  1.75       4          1.25       1.25       1
  C6               RECT_OPEN    1.5        2.5        0          0          1
```

```
C7              RECT_OPEN    2       3       0       0       1
C8              TRAPEZOIDAL  2       6       1.25    1.25    1
C9              RECT_OPEN    2       2.5     0       0       1
C10             RECT_OPEN    2.25    3.5     0       0       1
C11             RECT_OPEN    1.75    2.5     0       0       1
C12             TRAPEZOIDAL  1.75    6       1.25    1.25    1
C13             TRAPEZOIDAL  2.75    8       1.25    1.25    1
C14             TRAPEZOIDAL  2.75    10      1.25    1.25    1

[TIMESERIES]
;;Name          Date         Time    Value
;;------------------------------------------------
;Kenten Station
 TS1                         1       38.91
 TS1                         2       22.89
 TS1                         3       20.59
 TS1                         4       16.02
 TS1                         5       11.44
 TS1                         6       11.44
 TS1                         7       11.44
 TS1                         8       9.16
 TS1                         9       6.86
 TS1                         10      6.86
 TS1                         11      6.86
 TS1                         12      6.86
 TS1                         13      6.86
 TS1                         14      6.86
 TS1                         15      4.58
 TS1                         16      4.58
 TS1                         17      4.58
 TS1                         18      4.58
 TS1                         19      4.58
 TS1                         20      4.58
 TS1                         21      4.58
 TS1                         22      4.58
 TS1                         23      2.28
 TS1                         24      2.28

[REPORT]
 INPUT      NO
 CONTROLS   NO

[OPTIONS]
 TEMPDIR    "C:\DOCUME~1\User\LOCALS~1\Temp\"
```

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0 (Build 5.0.006a)
--------------------------------------------------------------

Simulation Study Of Sekanak Drainage System in Palembang City.

****************
Analysis Options
****************
Flow Units ............... CMS
Infiltration Method ...... GREEN_AMPT
Flow Routing Method ...... KINWAVE
Starting Date ............ FEB-25-2007 00:00:00
Ending Date .............. FEB-26-2007 00:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:15:00
Wet Time Step ............ 00:15:00
Dry Time Step ............ 00:15:00
Routing Time Step ........ 30.00 sec

**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Total Precipitation ......       252.935       221.970
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........       103.236        90.598
Surface Runoff ...........       148.520       130.338
Final Surface Storage ....         1.658         1.455
Continuity Error (%) .....        -0.190

**************************        Volume        Volume
Flow Routing Continuity        hectare-m       Mliters
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......       148.530      1485.317
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........       147.064      1470.656
Surface Flooding .........         0.003         0.032
Evaporation Loss .........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         2.099        20.993
Continuity Error (%) .....        -0.428

***************************
Subcatchment Runoff Summary
***************************
```

Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Runoff Coeff
S1	221.970	0.000	0.000	117.038	104.541	0.471
S2	221.970	0.000	0.000	116.119	105.755	0.476
S3	221.970	0.000	0.000	115.981	105.931	0.477
S4	221.970	0.000	0.000	105.724	115.979	0.522
S5	221.970	0.000	0.000	105.654	116.076	0.523
S6	221.970	0.000	0.000	115.517	106.482	0.480
S7	221.970	0.000	0.000	105.887	115.742	0.521
S8	221.970	0.000	0.000	75.258	143.832	0.648
S9	221.970	0.000	0.000	116.042	105.851	0.477
S10	221.970	0.000	0.000	73.831	147.400	0.664
S11	221.970	0.000	0.000	74.020	146.981	0.662
S12	221.970	0.000	0.000	64.522	153.687	0.692
S13	221.970	0.000	0.000	74.032	146.955	0.662
S14	221.970	0.000	0.000	73.871	147.307	0.664
Totals	221.970	0.000	0.000	90.598	130.338	0.587

```
******************
Node Depth Summary
******************
```

Node	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Total Flooding ha-mm	Total Minutes Flooded
J1	0.38	0.90	105.41	0 02:00	0	0
J2	0.56	1.32	104.68	0 02:01	0	0
J3	0.24	0.73	108.89	0 02:00	0	0
J4	0.60	1.46	104.58	0 02:02	0	0
J5	0.35	1.04	104.85	0 02:00	0	0
J6	0.50	1.50	105.09	0 01:53	3.15	9
J7	0.69	1.94	105.23	0 02:00	0	0
J8	0.76	1.89	104.84	0 02:11	0	0
J9	0.55	1.71	105.89	0 02:00	0	0
J10	0.68	2.02	105.45	0 02:00	0	0
J11	0.44	1.30	106.01	0 02:00	0	0
J12	0.68	2.01	104.99	0 02:04	0	0
J13	0.98	2.75	104.94	0 01:37	0.00	3
J14	0.98	2.75	104.31	0 01:46	0.00	4
Out1	2.62	2.75	103.74	0 01:07	0	0

```
********************
Conduit Flow Summary
********************
```

Conduit	Maximum Flow CMS	Time of Max Occurrence days hr:min	Maximum Velocity m/sec	Length Factor	Maximum /Design Flow	Total Minutes Surcharged
C1	2.65	0 02:10	1.28	1.00	0.35	0
C2	3.13	0 02:01	2.92	1.00	0.40	0
C3	8.26	0 02:04	1.53	1.00	0.57	0
C4	2.74	0 02:03	1.38	1.00	0.62	0
C5	14.44	0 02:03	1.71	1.00	0.71	0
C6	2.54	0 02:15	0.79	1.00	0.96	0
C7	6.23	0 02:11	1.14	1.00	0.93	0
C8	28.50	0 02:10	2.05	1.00	0.78	0
C9	5.65	0 02:06	1.47	1.00	0.81	0
C10	11.89	0 02:04	1.72	1.00	0.86	0
C11	5.89	0 02:04	1.91	1.00	0.68	0
C12	23.05	0 02:05	2.81	1.00	0.45	0
C13	59.86	0 02:10	2.42	1.00	0.72	0
C14	64.94	0 02:13	2.40	1.00	0.65	0

```
*************************
Routing Time Step Summary
*************************
Minimum Time Step           :    30.00 sec
Average Time Step           :    30.00 sec
Maximum Time Step           :    30.00 sec
Percent in Steady State     :     0.00
Average Iterations per Step :     1.10

Analysis begun on:  Sat Jun 02 17:50:08 2007
Total elapsed time: 00:00:01
```

****
Node Depth Summary
****

Node	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Total Flooding ha-mm	Total Minutes Flooded
J1	0.37	0.88	105.39	0 02:00	0	0
J2	0.54	1.28	104.64	0 02:01	0	0
J3	0.24	0.70	108.86	0 02:00	0	0
J4	0.59	1.42	104.54	0 02:01	0	0
J5	0.34	1.00	104.81	0 02:00	0	0
J6	0.48	1.48	105.07	0 02:00	0	0
J7	0.66	1.86	105.15	0 02:00	0	0
J8	0.73	1.82	104.77	0 02:11	0	0
J9	0.54	1.63	105.81	0 02:00	0	0
J10	0.66	1.94	105.37	0 02:00	0	0
J11	0.43	1.26	105.97	0 02:00	0	0
J12	0.66	1.94	104.92	0 02:04	0	0
J13	0.96	2.75	104.94	0 01:42	0.00	1
J14	0.96	2.75	104.31	0 01:54	0.00	2
Out1	2.62	2.75	103.74	0 01:07	0	0

****
Conduit Flow Summary
****

Conduit	Maximum Flow CMS	Time of Max Occurrence days hr:min	Maximum Velocity m/sec	Length Factor	Maximum /Design Flow	Total Minutes Surcharged
C1	2.53	0 02:10	1.26	1.00	0.33	0
C2	2.95	0 02:01	2.88	1.00	0.37	0
C3	7.83	0 02:04	1.51	1.00	0.54	0
C4	2.61	0 02:03	1.37	1.00	0.59	0
C5	13.72	0 02:03	1.68	1.00	0.68	0
C6	2.42	0 02:15	0.79	1.00	0.92	0
C7	5.94	0 02:11	1.13	1.00	0.89	0
C8	27.13	0 02:10	2.02	1.00	0.74	0
C9	5.36	0 02:05	1.45	1.00	0.77	0
C10	11.34	0 02:04	1.71	1.00	0.82	0
C11	5.64	0 02:04	1.89	1.00	0.65	0
C12	22.02	0 02:05	2.77	1.00	0.43	0
C13	57.10	0 02:10	2.39	1.00	0.69	0
C14	61.96	0 02:13	2.37	1.00	0.62	0

****
Routing Time Step Summary
****

```
Minimum Time Step           :    30.00 sec
Average Time Step           :    30.00 sec
Maximum Time Step           :    30.00 sec
Percent in Steady State     :     0.00
Average Iterations per Step :     1.10
```

```
Analysis begun on:  Tue Jun 05 01:42:23 2007
Total elapsed time: < 1 sec
```

```
EPA STORM WATER MANAGEMENT MODEL - VERSION 5.0 (Build 5.0.006a)
--------------------------------------------------------------

Simulation Study Of Sekanak Drainage System in Palembang City.

****************
Analysis Options
****************
Flow Units ............... CMS
Infiltration Method ...... GREEN_AMPT
Flow Routing Method ...... KINWAVE
Starting Date ............ FEB-25-2007 00:00:00
Ending Date .............. FEB-26-2007 00:00:00
Antecedent Dry Days ...... 0.0
Report Time Step ......... 00:15:00
Wet Time Step ............ 00:15:00
Dry Time Step ............ 00:15:00
Routing Time Step ........ 30.00 sec


**************************        Volume         Depth
Runoff Quantity Continuity     hectare-m            mm
**************************     ---------       -------
Total Precipitation ......       243.864       214.010
Evaporation Loss .........         0.000         0.000
Infiltration Loss ........       100.696        88.369
Surface Runoff ...........       142.000       124.616
Final Surface Storage ....         1.625         1.426
Continuity Error (%) .....        -0.188


**************************        Volume        Volume
Flow Routing Continuity        hectare-m       Mliters
**************************     ---------     ---------
Dry Weather Inflow .......         0.000         0.000
Wet Weather Inflow .......       142.007      1420.083
Groundwater Inflow .......         0.000         0.000
RDII Inflow ..............         0.000         0.000
External Inflow ..........         0.000         0.000
External Outflow .........       140.571      1405.728
Surface Flooding .........         0.000         0.000
Evaporation Loss .........         0.000         0.000
Initial Stored Volume ....         0.000         0.000
Final Stored Volume ......         2.057        20.569
Continuity Error (%) .....        -0.438


***************************
Subcatchment Runoff Summary
***************************

-----------------------------------------------------------------------------------------
                         Total      Total      Total      Total      Total     Runoff
                        Precip      Runon       Evap      Infil     Runoff      Coeff
Subcatchment                mm         mm         mm         mm         mm
-----------------------------------------------------------------------------------------
S1                     214.010      0.000      0.000    114.119     99.497      0.465
S2                     214.010      0.000      0.000    113.316    100.594      0.470
S3                     214.010      0.000      0.000    113.187    100.756      0.471
S4                     214.010      0.000      0.000    103.153    110.581      0.517
S5                     214.010      0.000      0.000    103.092    110.671      0.517
S6                     214.010      0.000      0.000    112.766    101.267      0.473
S7                     214.010      0.000      0.000    103.300    110.365      0.516
S8                     214.010      0.000      0.000     73.264    137.913      0.644
S9                     214.010      0.000      0.000    113.245    100.683      0.470
S10                    214.010      0.000      0.000     72.050    141.220      0.660
S11                    214.010      0.000      0.000     72.219    140.825      0.658
S12                    214.010      0.000      0.000     62.810    147.504      0.689
S13                    214.010      0.000      0.000     72.229    140.801      0.658
S14                    214.010      0.000      0.000     72.089    141.132      0.659
-----------------------------------------------------------------------------------------
Totals                 214.010      0.000      0.000     88.369    124.616      0.582
```