CHAPTER 2

Methodology.

2.1: Study Area.

The study area is located in Seberang Prai Selatan. It is about 60 sq.km. in area, covering Nibong Tebal Town and its peripheral areas. These peripheral areas include Caledonia, Victoria, Tanjung Berembang, Telok Ipil, Sang Lang Village, Bukit Panchor Road and Air Lintas. These areas are partly urbanized. They were included in this study because eventually they will be a part of the greater Nibong Tebal Town when fully urbanized.

2.2: Data sources- primary and secondary.

Primary data were obtained from direct field measurements of a flood phenomenon. Secondary data were obtained mainly from reports or publications by government departments. This research is based on both.

2.3: Demarcation of sub-basin boundaries.

For this study, the demarcation of sub basin boundaries of the study area were based on DID's drainage map. These boundaries were synthesized from a set of interpolated spot heights. The spot heights were measured by the Survey Department in the mid 60's, and subsequently the drainage boundaries were reconstructed because of the urbanization process which shifted the boundaries at several areas. Realignment was made, after the current drainage network, flow direction and service area were identified. They were identified by referring to a drainage map and through field observations. After that, the sub-basins were divided into smaller drainage units.

Demarcation of sub-basins and drainage units boundaries are important for reasons listed below:

1- Sub-basin forms the basic spatial unit for analyses.

2- Surface hydrological processes could be confined within the sub basin for study.

3- Flash floods are localized events and best studied within the context of the effected sub basin.

2.4: Adopting a broader urban and related landuse classification.

The Malaysian landuse classification for urban and related areas are listed separately under the 'U' series. This current landuse classification is based on economic function rather than on physical characteristics. However, in this study, the physical characteristics of landuse is more relevant when examining rainfall and runoff relationship. Therefore, urban and related built up areas were included in a same category. Thus, a broader classification for urban landuse was adopted for this study area.

2.5: Computing urban changes.

The spatial and temporal dimensions of urbanization were considered when changes in land usage was studied.

Temporal dimension refers to changes through time, while spatial dimension refers to directions of areal changes from a focal point. DOA's landuse maps of 1966, 1974, 1989 and 1995 were used to determine these changes (from agricultural to urban landuse). As for newly urbanized areas (as at February 1998), field observations were conducted to identify them. Then, these newly urbanized or built up areas were demarcated. Their boundaries were confirmed on a cadastral sheet printed by the Land Survey Department. Cadastral sheets were used as they were bigger in scale and more accurate.

The methods employed to determine urban changes are as follows:

1- The urban areas were traced out from non urban areas that fell within the sub-basins (agriculture or forested areas). The non urban areas are mainly oil palm plantations.

2- The urban boundaries were digitized first. Their sizes were calculated automatically by using a computer aided design or CAD software. This method is more accurate than the conventional planimetric method.

3- Spatial dimension was constructed based on the digitized urban boundaries. The former police station located at the old town was selected to be the focal point for constructing this dimension. The old town was the administrative and commercial centre for Nibong Tebal. From the focal point, the town was divided into few sections according to geographical directions. These sections are: North (covering between North West and North East reaching Jawi), East (covering between North East and South East reaching the junction of Sungai Kechil town), South (covering between South East and South West reaching Tanjung Berembang) and West (covering between South West and North West reaching the coastal strip of Penang South Channel). Areas located south from Krian River were not considered because they were beyond the study area

4- The rates of urbanization were computed and their temporal trends from 1966-1995 were synthesized for each section by using a spreadsheet software. Polynomial function was used to construct the trendlines because it gave the closest representation of the data. After that, the rates of urbanization and directions of change were related to produce the spatial and temporal dimensions.

2.6: Underlying factors of urbanization.

Trends (spatial and temporal dimensions) will be used when analyzing the underlying factors of urbanization. The factors that were taken into consideration were demography, economics and physiography.

2.6.1: Demography.

Population data for 1947, 1970,1980 and 1990 census were used to generate growth rates for mukims that lies within the sub-basins. The latest data is not available because the following census would be on year 2000. Although mukim and sub-basin boundaries differed, observations had shown that residential areas were concentrated within the study area. Therefore, population data for a specific mukim could be used to indicate growth rate in a particular sub-basin or two sub-basins together if the mukim extend over a sub-basin under study. The population growth rates would imply urbanization rate. Housing data could indicate changes of landuse from agricultural to residential as a result of growth in population.

Several conditions were considered before computing population change and making projections. They were constant area, constant classification and coverage. All the mukim boundaries had remained the same throughout all the census period. Therefore, all the census areas were constant. This was determined by comparing the census maps for each census. Classification and coverage were ignored because growth rate and projection were meant for total population only.

Projections were made under the assumptions of: no war or natural disaster; no economic fluctuations and no change to current demographic status. There was a limit to projections as growth could not continue on a same pattern indefinitely. Usually, projections of more than 25 years were meant for development of water and forestry resources, major transport and recreational facilities. For this study, the above length of projection was adopted.

Rates of change were calculated for single or multiple periods to produce a trendline. Population growth trend were computed by using the exponential formula (see equation 1). From this exponential trendline, future growth or projections were done.

Exponential formula: $$Pt = Po\ e^{rt} \quad \text{.............................} \quad (1)$$

Pt -- population at t period, Po- base year population, r - rate of change, t-number of years.
e- constant 2.71828

This formula was chosen because it was simple to use and gave a continuos compounding or constant rate of change.

2.6.2: Economic changes.

Change of economic activities from agricultural to industrial and commercial indicates changes of landuse were considered. Economic activities within or near to a sub-basin usually served as a catalyst to landuse change. For example, a newly established industrial area can be a catalyst of landuse change, from surrounding agricultural area to residential and commercial area. By using the economic data for Penang, rate of change was computed and future trend extrapolated.

2.6.3: Physiography.

By relating spatial trend of urbanization with physiology, the influence of relief upon urbanization processes can be understood. This will assist in predicting future trend. The physiography of the study area was examined from a topographic map, drainage map and through fieldwork. Terrain analysis was conducted using the maps and informations obtained from fieldwork. Using the results of the terrain analysis, the physiography of the study area were divided into: coastal strip; flood plain; pineplain (0.5 to 15 m) undulating region (15 to 30 m) and highland (above 30 m).

A digital terrain model or DTM was constructed to visualize the physiology of the study area. By using a DTM the terrain and it's influence on surface hydrology can be better described. Furthermore, with a DTM, slope analysis can be done digitally. Results from the slope analysis were an important input for providing estimates of average surface steepness. Estimates of average surface steepness were needed for calculating overland flow time.

The DTM was produced by using IDRISI 4.1 a raster based Geographic Information System software. Raster format is easier to use to produce surface models. The steps taken are as stated below:

1-A Base (topographic map 1:50,000) map was digitized. All reference points were registered in MRSO coordinates and not the Cartesian coordinates of the digitizing board. All important features that outlined the physiography of the study area were digitized.

2-The data file was exported to a CAD program in 'dxf' format. Elevation values were added to every points or spot heights. These points were digitized along contour lines with elevation values assigned to them.

3- Then, the data was exported to Idrisi4.1 using the 'dxfidris' command in Idrisi4.1.

4-The point data was used to generate DTM. Initial or blank raster file was not needed when using point data.

5-Suitable number of column and rows were computed based on the coordinates' minimum and maximum. Number of rows and columns must tally with the coordinate's difference: example: if X max. -X min. is 5000 (which is 5 km in MRSO reference) Y max. - Y min. is 2500, suitable number of rows and columns will be: 250 columns by 125 rows. Column-row ratio is in equal proportion to the ratio of X and Y maximum. Base on this, a blank raster file was generated. The outline of the study area, contained in a vector file (converted from the dxf file earlier) was converted into a raster file and stored in the blank file. This new raster file overlaid on the DTM will relate hydrological features to terrain characteristics for further analyses.

2.7: Climatological background.

2.7.1: Rainfall.

DID's monthly rainfall records from five stations were used. Four stations had 45-65 years of record and the other 110 years of record. All these stations are about 5-7 km away from study area. There are no stations located within study area. All these stations record daily rainfall only (8.00 a.m. today to 8.00 a.m. next day). The stations are listed below.

Table 2.1
List of Rainfall Stations

ID.	Location	Operated by:	Records
5104052	Kawasan Sg. Acheh	DID	Daily - manual
5104011	Hospital Parit Buntar	MMS	Daily- manual
5105051	Bukit Panchor Filtration Plant.	DID	Daily - manual
5205050	Sek. Keb. Sg. Bakap	DID	Daily - manual
5204049	Ladang Batu Kawan	DID	Daily - manual
5204048	Simpang Ampat*	DID	Continuos Recording- automatic

Note:* This station located about 14 km away. Its daily rainfall data is not relevant to this study. However, because it is the nearest continuos recording station, it was used for developing an IDF curve.

Rainfall data was processed first, before being utilized. The processes are as stated below:

1- Missing records were computed by using Normal Ratio Method (using three nearby stations as suggested by U. S. Weather Bureau) and Ratio Method (based on one nearby station, if there is no three stations nearby).

Normal Ratio Method: $Px = 1/3(NxPA/NA + NxPB/NB + NxPC/NC)$ ....(2)

Ratio Method: $Px = NxPA/NA$ .... (3)

Px-rainfall at station X , Nx- annual mean rainfall of station X, NA...NC - annual mean rainfall of station A,B and C, PA...PC- rainfall at station A,B and C which coincide with the missing value at station X.

2- The consistency of records were analyzed by using the Double Mass Curve. It is a method of adjusting rainfall records that take into account non representative factors, such as change in location or exposure of the rain gauge.

Cumulative rainfall for each station was plotted against the mean cumulative rainfall of five stations. The curves were examined for their linearity. If at any point a diversion occurs for more than five years, adjustments are needed. To synthesize a continuos record, the observed data was adjusted by multiplying them by the ratio of the slopes of the two line segments (Equation 4).

$Pa = (ba/bo)Po$ .... (4)

Pa- adjusted rainfall, Po- observed rainfall, ba- slope of line which records adjusted. bo- slope of line at observed Po.

The processed data was used for several relevant analyses such as:

1- Computing mean areal rainfall for monthly and annual rainfall by using Thiessen Polygon Method. This method was chosen because it was suitable for this study area which is relatively flat with minor orographic effect.

2- Calculating long term annual and monthly rainfall pattern based on 110 years of record in order to observe climatic fluctuations. This was because increase in occurrence of flash flood could be attributed to climatic condition other than drainage efficiency or urbanization processes. A trendline was drawn to examine the possible influence of climatic fluctuations on occurance of flash floods.

3- Computing long term mean annual rainfall distribution based on 49 years of records from five the stations.

2.7.2: Temperature.

Daily temperature was monitored by MMS at Hospital Parit Buntar. Annual monthly mean temperature records were computed from these daily records by MMS. Short term monthly mean temperatures (average of records taken at 8.00 a.m. and 2.00 p.m.) were computed from these annual records for a period of 20 year.

2.8: Intensity, Duration and Frequency (IDF) curves.

Currently Bagan Seria IDF curves are being used for most planning and design purpose in this region. A new IDF curve was developed based on data recorded at Simpang Ampat station. This was because Simpang Ampat is the nearest recording station to the study area (14 km away compared with Bagan Seria and Butterworth that is 30-32 km away). 1988 to 1995 continuos records were used.

Annual maximum intensities ranging from 15 minutes to 24 hours were extracted for computing the IDF curves. The Extreme Value Type I or EV I distribution method was employed for this purpose. It had been widely used in hydrology. Storm rainfalls were most commonly modeled by EV 1. This method was recommended by DID.

Extreme values (maximum intensities for various durations) are located in the extreme tail of the parent probability distribution. The EV1 method is as described below.

1- EV1 function:

$$F(x) = \exp\left[-\exp\left(-\frac{x-u}{\alpha}\right)\right]$$

$$-\infty \leq x \leq \infty \qquad \text{.....................................} (5)$$

$$\alpha = \frac{\sqrt{6}}{\pi} s$$

$$u = \bar{x} - 0.5772\alpha$$

s- standard deviation, $\bar{x}$- mean value: for data set containing annual maximum rainfall depths (1988-1995) for duration considered.

2- Variate y substituted into the function (Equation 5), the function redefined as:

$$F(x) = \exp[-\exp(-y)]$$

$$y = -\ln\left[\ln\left(\frac{1}{F(x)}\right)\right] \qquad \text{.................................... (6)}$$

3- Return period T taken as an alternate axis to y. The relationship is as below and substituted into YT :

$$F(x_T) = \frac{T-1}{T}$$

$$y_T = -\ln\left[\ln\left(\frac{T}{T-1}\right)\right] \qquad \text{.................................... (7)}$$

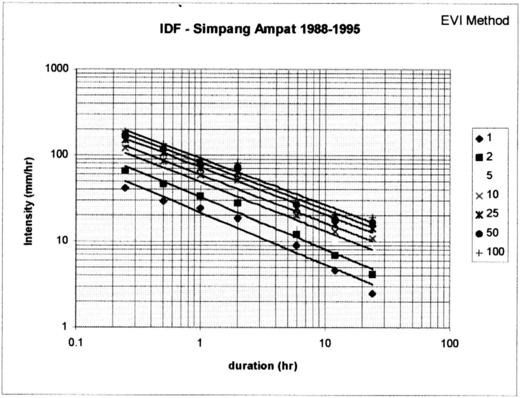
$$x_T = u + ay_T$$

For EV I distribution, XT is related to YT in the Equation 7 above. T is the return period and XT is the corresponding rainfall depth. This was done for rainfall duration of 0.25, 0.5, 1, 2, 6, 12 and 24 hours with return periods of 2, 5, 10, 25, 50 and 100. The computed values were standardized into mm/hr. Then, the values were plotted according to their duration, intensity and return period using a logarithmic scale to produce a series of almost linear downward slopping IDF curves (see Chart 2.1). Return period of one year can not be computed using above formulas. To compute it, the formula below was employed:

Tr = N/m .... (8)

N- years of record, Tr-return period, m - is the corresponding rank (ascending order), whereby the annual maximum rainfalls for a particular duration in the data set are arranged to it (but in descending order).

Chart 2.1



2.9: Hydrogeological background

2.9.1: Quaternary geology.

Descriptions were based on geological map provided by Geological Survey of Malaysia. As this is a coastal area, quaternary geological survey was used to describe the formation of various sediment layers.

2.9.2: Aquifers.

Based on 24 bore holes' data from IKRAM (JKR), GSM and Binnie & Partners the sub-surface condition was described. These bore holes were located within the study area. Out of the 24 bore holes, 4 were deep bore holes (>30m deep), 16 shallow bore holes (10 to 20 m deep) and other 4 were shallow bore holes located on the Krian River (2 to 9 m deep). Further analyses were conducted using the bore holes. The analyses are stated below.

1- Identifying aquifers: From a network of bore holes, layers of clay (mainly marine) and sand were identified. Sand and sandy layers formed the aquifers.

2- Types of aquifer (multi layers or single layer; confined or unconfined): A cross sectional extrapolation was done for this purpose. The cross section followed a series of grid network ranging from coastal strip to inland hills and parallel to the coast.

Recharge into the aquifer from within study area is by deep percolation through the surface layer that is generally clayish. Recharge from without is by percolation through a generally sandy surface layer found on the granite hills (which possibly joined to the sandy layers found within the study area). GSM estimated less than 15 % of the rainfall percolates into the aquifer from the granite hills and even lesser percentage through the clayish layer (depending on soil physics).

Retardation of flow or discharge resulting from sea water intrusion was not estimated because of insufficient data. Sea water intrusion can offset the losses from the aquifer, causing the water table to have only minor fluctuations. Generally, this denser saline water flows underneath the fresh water flowing out from the aquifer (salt wedge). The saline water flows in and replaces the losses especially during drier periods, thus offsetting the effect of groundwater discharge on water table level.

2.9.3: Water table.

Most of the bore hole logs recorded the water table at time of drilling. Records of water table from bore holes all over Seberang Prai had shown the level to be between 0.5 to 1.5 m below surface on the coastal plain despite the difference in season. These records were analyzed for minimum mean and maximum mean water table. The minimum mean was used as an estimate of water table for wetter months and the latter for drier months. These values could be obtained by:

1- Computing the mean level first.

2- Computing their standard deviation.

3- All values that fell within one standard deviation below the mean were averaged to produce the minimum mean. Values that fell within one standard deviation above mean were averaged to produced the maximum mean. One standard deviation was chosen to exclude extreme values. These values were used for choosing a more representative effective saturation value when calculating infiltration rates.

2.10: Pedalogical background.

2.10.1: Malaysian soil series.

Various soil types for each sub basin within the study area were identified from the Malayan Soil Survey map for Province Wellesly (Seberang Prai) by using map overlay method.

2.10.2: Textural classification.

The relevant soil series were reclassified into their respective textural classification (USDA) by using the TAL software. TAL was developed by the Soil Science Department of University Putra Malaysia. Reclassification was needed when using Green-Ampt method for calculating infiltration rates.

2.10.3: Green-Ampt Method for estimating infiltration rates and the computation of composite runoff coefficients.

The Green-Ampt parametres for each soil type are listed in Table 2.2.

Table 2.2
Green-Ampt Parametres

total porosity	effective porosity	wetting head (cm)	K (cm/hr)	Malaysian Series	USDA
0.475	0.385	31.63	0.03	Kranji- Linau	clay
0.464	0.398	21.32	0.56	Manik-Sagomano	sandy loam and clay
0.475	0.385	31.63	0.03	Selangor	clay
0.425	0.371	16.43	0.62	Manik- Lunas	sandy clay loam and sandy loam
0.43	0.382	15.37	0.24	Lunas -Holyrood	sandy clay loam and loamy
0.453	0.412	11.01	1.09	Rengam	sandy loam
0.475	0.385	31.63	0.03	Telok -Selangor	clay
0.475	0.385	31.63	0.03	Linau -Permatang	clay

Base on their textural classification, relevant values (Green-Ampt parameters) for each soil type was identified for further analysis. However, the values for these parameters were estimation only based on research done by Brakensiek, Engleman and Rawls (in Chow, 1981).

Most method for estimating infiltration is based on the assumption that water is ponded to a small depth on the surface. Ponding time for a set of rainfall intensity for return periods of 1, 10 and 50 years was calculated. This data was used for estimating infiltration rates after the soil surface layer reached saturation point or when ponding occurred. After that, ponding time was related to rainfall duration (1 hour -- a normal duration for most rain storm in this area), in order to account for the change in infiltration rate. The rainfall intensity, duration for these return periods were taken from the IDF graph.

Weighted infiltration rates (according to percentage of area size) were used to produce new composite runoff coefficients that take into account local conditions. Using these runoff coefficients, overland flow for each drainage unit for a set of rainfall intensity and duration was calculated.

The sub-basins within the study area contain more than one type of soil and different percentage of landuses. The infiltration rates differed between the different landuses (i.e., bareland, vegetated and built up area) and soil types. However, the effect of bareland condition to the average infiltration rate and overland flow was not significant as it was less than 5% of the total study area.

The composite runoff coefficients were produced for each drainage unit on each sub-basin to account for their differences in soil types and landuses. For built up area the runoff coefficient was estimated using Pattison Method.

The composite runoff coefficients were used in the Modified Rational Method. By using these composite runoff coefficients, better discharge estimates could be produced because they were closer to the condition of the study area. The procedure is as stated below:

1- The potential infiltration rate for each soil type was calculated. Equation 9 was solved by using successive substitution method. The obtained value was substituted into Equation 10 to calculate the potential infiltration rate.

$$F(t) = Kt + \psi\Delta\theta\,(1 + F(t)/\,\psi\Delta\theta) \quad \text{..............................} (9)$$

$$f(t) = K(\psi\Delta\theta/F(t) + 1) \quad \text{..............................} (10)$$

Note that Equations 9-16 are from the Green-Ampt Method. F(t)- cumulative infiltration rate, could be solved faster using the more complex Newton's iteration method. (f(t))- potential infiltration rate. t- duration of rainfall. Kt- hydraulic conductivity, ψ-wetting front soil suction head, $\Delta\theta$ - Is the change of moisture content as the wetting front passes as the infiltrated water moves downward, calculated using Equation 12:

$$Se = \theta - \theta r / \eta - \theta r \quad \text{..............................} (11)$$

$$\Delta\theta = (1 - Se)\,\theta e \quad \text{..............................} (12)$$

Se- effective saturation, θr- residual moisture content after the soil being thoroughly drained. θe- effective porosity the product of subtracting residual moisture from total porosity denoted by η. θ - available moisture in soil.

Se (Equation 10) values ranged from 0 to 1.0 and could be calculated for various degrees on initial saturation. For this study, the initial saturation level was taken at 0.5 degree of saturation for all types of soil. This was done by changing the initial available moisture to be at 50 % of the total porosity of each soil type. 50% initial saturation was chosen after taking into consideration the water table levels mentioned earlier. Infiltration rate for initial saturation level of 0.2 and 0.9 degrees was calculated for comparison only and not for further analysis.

2- Ponding time was calculated for each soil type.

$$tp = (K\psi\Delta\theta)/i(i-K) \quad \text{.................... (13)}$$

$$Fp = itp \quad \text{.................... (14)}$$

tp- time for ponding to occur, i-rainfall intensity , Fp-depth of water infiltrated.

3- Infiltration rate after ponding occurred was calculated for each soil type.

$$F - Fp - \psi\Delta\theta \ln [(\psi\Delta\theta + F)/(\psi\Delta\theta + Fp)] = K(t-tp) \quad \text{........ (15)}$$

$$f = K(\psi\Delta\theta/F + 1) \quad \text{.................... (16)}$$

4- A representative infiltration rate for each soil type was selected. The infiltration rate before and after ponding was compared for all soil type. The difference was insignificant for all cases (less than 0.1 mm/hr). Therefore, the infiltration rate after ponding was used. Moreover, for all cases ponding occurred early during the rain storm event.

5- A Composite runoff coefficient was calculated for each drainage unit under different rainfall intensity. For every drainage unit, the total area was divided firstly into urban and agriculture and forested area. The runoff coefficients for urban areas depended on their built-up densities. They were computed using the Pattison Method (see Equation 17).

$$c = \exp(-\{k/[I-a]\}^n) \quad \text{.................... (17)}$$

c- runoff coefficient.

The values for parametres a, k, n are shown in Table 2.3.

Table 2.3
Values of a, k, and n used in computing runoff-coefficients

Catchment Character.	a (mm/h)	k (mm/h)	n
Urban Impervious roofs, concrete Solidly built up.	2	1	0.49
Urban Surface clay, poor paving Closely built up.	4	2	0.49
Urban Semi-detached houses on bare earth.	6	4	0.46
Urban Bare earth, or with sandstone outcrops urban residential.	10	8	0.47
Urban Suburban residential with gardens bare loam.	10	28	0.65
Urban Widely detached houses on loam or sand strata.	12	43	0.61
Urban Parks, lawns, meadows.	12	68	0.66
Urban Cultivated fields.	15	105	0.55

For non-urban areas, the soil type and vegetation cover were taken into consideration. They were divided into smaller units, each containing the same soil and vegetation type. All the divisions were weighted according to their area sizes divided by total area covered by the drainage unit. Then, the runoff percentages were calculated for each soil type for non-urban areas. Forested areas were given 0 values because all the existing forests are either coastal or riverine wetlands. A simple equation for calculating was formulated based on interception, surface retention and rainfall intensity.

$$C = [(I*(1-i) - f) *(1- 0.1 \text{ or } 0.15)] / I \qquad \text{.............................. (18)}$$

C-runoff coefficient, I- intensity of rainfall , i- interception ratio 0.3, 0, f- infiltration rate, 0.1, 0.15 assumed surface detention ratio for clay and non-clay.

The runoff coefficients for urban and non-urban area were multiplied with the weightage given to their respective divisions within each drainage unit and their product were added to produce a composite runoff coefficient.

2.11: Current drainage condition.

Field study was conducted to examine:

1- Drainage networks and flow directions.

2- Drainage boundaries of internal and external drains.

3- Type and condition of drains; the width and depth were measured, and drain surface condition noted. This data was used for the Manning's equation.

4- Design and condition of relevant tidal gates.

2.12: Tides

Tidal pattern for Sg. Krian: Hourly tidal data for 1996 was obtained from DID. Relevant informations were extracted from this data set. The relevant informations are listed below.

1- Mean tide level: Obtained by computing the arithmetic mean of the data set.

2- Range: Obtained by caculating the difference between annual maxima and minima. Neap and spring range were calculated by using the same method.

3- Mean high tide: Obtained by computing the average of values that fell within three standard deviations above mean.

4- Mean low tide: Obtained by computing the average of values that fell within three standard deviations below mean.

The tidal distribution or cases were assumed to be normally distributed. Therefore, three standard deviation limit was adopted to represent 99.9% of the cases. The values at both tail ends of the normal distribution curve had a probability value less than or equal to 0.1 %. They were considered to be insignificant or abnormal cases and were discarded. As a result, extreme values were omitted from analysis. This made the calculated range, upper and lower means to be closer to normal condition.

5-Seasonal pattern: This was described directly by plotting the hourly tidal data for the whole year. Mean tide level, mean high and low tide were extracted for different season (equinox and solsiltis). The seasonal means were related to the annual rainfall pattern to examine their relationship. As a result, sensitive months were identified, when both the seasonal tide and rainfall were higher.

Tidal pattern for Sg. Daun/ Tengah: No existing data was available. Observation was carried out periodically for a period of one month to estimate the typical high tide level. Highest daily tide level was observed directly from water mark left on a measuring stick. To make the water mark more distinguishable, the measuring stick was covered with a thick layer of chalk.

2.13: Flash flood phenomena.

Frequency of flash floods was based on DID annual reports and other records. Other sources were needed because DID's records were insufficient. A long term frequency was unable to be established because previous flood events were not well documented.

Flash flood records for the last five years were kept in a school. However, only 1996 and 1997 records were available because the rest was destroyed during a major flood in 1995. As a result, correlation between flash flood frequency and urbanization cannot be determined for the past.

Flash flood locations and boundaries were based on DID's annual report. Although, DID did not record every flash flood event, areas that were often flooded had been roughly demarcated.

The maximum flood depths for 1997 were recorded by measuring ferric oxide stains left by flood water on concrete structures all over the affected areas. Depths for previous years were based on DID's records. Locations of the sampling points were marked on a cadastral sheet for the purpose of georeferencing and mapping out the flood depths.

Present and future potential flood boundaries for various rainfalls and tidal scenarios were constructed using the existing sampling points and spot heights. The steps taken to construct these potential flash flood boundaries as follows:

1- Points of flood depths (marked on the cadastral sheet) and spot heights were interpolated to produce contour lines of 0.3048m (1 foot) intervals.

2- A detail terrain map of the flooded area was produced from the contour lines.

3- Using the detail terrain map, depth-volume curves were produced. These curves were used to estimated flood depths and extend for a given flood volume.

The steps taken to produce the depth-volume curves are as stated below:

1-Using the detail terrain map, the areas covered by each contour interval or elevation (starting from the lowest areas) were cumulatively calculated.

2- Knowing the elevation and its corresponding area size, its volume was calculated.

3- A series of depths and corresponding volumes were plotted to produce each curve. The curves were constructed for Nibong Tebal town area, East and South East of Nibong Tebal.

4-Adjustments were made to the curves by including storage capacities of the major drains. Storage capacities of drains were calculated by:

- Plotting the bed levels of all major drains, using 0 LSD as datum.
- For each hypothetical flood depth, the total lengths of major drains covered fully and partially were multiplied with their respective cross sectional area to compute the volume. Further adjustments were made for partially covered sections. The obtained values were multiplied by 0.8 to obtained a closer estimated of

normal capacity. This was because field observations throughout the study area had shown that the drains wère usually about 20% filled.

2.14: Flash flood estimation and analyses.

Over bank flow occurrs when discharge exceeds channel capacity. Discharge and channel capacity were estimated by using the Modified Rational Method and Manning's Equation.

The Rational Method was used to model the effects of urbanization on surface runoff for present and future condition. Future urbanized area assumed to equal the areas designated for development under the current town council plan.

For estimating discharge, the steps taken are listed below :

1- Each sub-basin within the study area was divided into smaller drainage units. Major convergence points (design points) were identified for each drainage system.

2- Discharge for each drainage unit was estimated using the Modified Rational Method.

3- When discharges from separate units converged in a trunk drain, the discharge values were simply added and vice versa.

4-When the accumulated peak discharges exceed drainage capacity at a particular design point, their overflow or exceedence values were subtracted. Therefore the discharge to the next design point downstream was equal to the drain's full capacity. By doing this, the cumulative discharge and overflow a long the drainage networks for each sub-basin were studied. Thus, enabling detection of any section where discharge exceeds drain discharge capacity (causing flood).

Rainfall intensities with return periods of 1, 10 and 50 years for duration of 1 hour were used for all cases. As a result, peak discharges for various return periods for current condition and future were computed and comparison were made to examine effect of urbanization.

Rational Method:

$$Q = CIA \qquad \text{..............................} (19)$$

Modified rational method:

$$\mathbf{Q = Cs\ CIA} \qquad \text{..................................} (20)$$

Cs = 2 tc/(2 tc + td) .....for tc equals to rainfall duration.
Cs = 2 te /(2 tc + td).......for tc shorter than rainfall duration.
Cs = tc+ te / (tc + te + td)...for tc longer than rainfall duration.

$$tc = to + td \qquad \text{..................................} (21)$$

Q- peak discharge in cumecs, Cs-storage function; tc,td,te,to- time of concentration, detention,rainfall duration and overland flow; C-composite runoff coefficient, I- rainfall intensity, A- area.

Steps taken to calculate 'tc' or time of concentration is as stated below.

1- 'to' or time of overland flow was calculated using Equation 22. 'to' is the travel time for overland flow from the edge of the sub-basin or drainage unit to the nearest drain or identifiable stream. Travel time was calculated by using the kinematic wave method developed by Morgali, Linsley (1965) and Aron and Erborge (1975) (in Chow, 1988), shown in the Equation 22:

$$to = 0.94\, L^{0.6}\, n^{0.6} / (I^{0.4}\, s^{0.3}) \quad \text{.................................} (22)$$

L- length of travel in ft, n- Manning's roughness coefficient, I- rainfall intensity in in/hr, s- average overland slope in ft/ft.

2- 'td' or time of detention in the drain was calculated using Equation 23.

$$td = L/V \quad \text{.................................} (23)$$

L- length of main stream or drain , V-velocity calculated by using Manning's equation shown in Equation 23:

$$V = (1/n)(R^{\wedge}2/3)(S^{\wedge}1/2) \quad \text{.................................} (24)$$

V - velocity in m/s,n - roughness coef.,R - hydraulic radius,A - flow cross sec. area, S - slope m/m, P - wetted perimeter,R=A/P.

3- 'tc' was calculated using Equation 21.

Estimating drain capacity:

1- The drainage network for each sub-basin was identified first from drainage map and field study.

2- The current width and depth (from free board) of each drain was measured during field study (to calculated the cross sectional area). Bed level could not be measured directly because of time, man power and equipment constrain. Bed levels were needed in order to compute drainage gradient for Manning's equation. Bed levels for external trunk drains were provided by DID.

Gradient of 1: 5000 was assumed for most external branches (based on DID's design standard) except in sub basin A (units A1-A2), 1:783, and 1:800 to

1:2500 for internal or urban drains (based on plans submitted to MPSP). The type and condition of the drains were recorded (to estimate the roughness coefficient).

3- Employing the Manning's equation (see Equation 24), flow velocity was computed for each major section of the drainage network.

4- The flow velocity was multiplied with the cross sectional area of a particular drain to produce drain discharge capacity (Q = V A).

The peak discharges and flow velocity were computed by using the Modified Rational Method and Manning's equation assumed free flow condition. Adjustments had to be made because free flow conditions do not always occur especially on flood plain. Therefore few scenarios were fowarded to give a more detailed study of flash flood.

Scenario:	Adjustments:
-low tide, -tidal gates opened fully, -ideal scenario.	-not needed, -free flow.
-mean high tide (R.L 0.9 m), -tidal gates were fully closed for three hours (based on study of the tidal pattern; high tide levels normally last for three hours), -pumps not operational, -worst case scenario.	-Storage capacity of trunk drains calculated. Method of calculation mentioned earlier. -Flood occurs when storage capacity exceeded. -Storage capacity drawn against total volume of runoff accumulated (two hours after rain stopped) of a particular rainfall

	event. Calculated directly from design hydrographs. -Difference between the total volume of water that flowed into the drains and the storage capacity calculated. -The excess water formed the flood volume. The flood depths and extend obtained by referring to the depth-volume graph and the detail terrain map.
-mean high tide, -tidal gates fully shut, -pumps operating at full capacity.	-The discharge at the outlet equaled to the pumping rate. -For cases with lower pumping rate, backwater curves* were drawn. -Sections affected by backwater, their drain discharge capacity was lowered. -Elsewhere, free flow condition assumed.
-Mean tide level (outlets are submerged). -pumps not operating. -tidal gates fully opened.	-Discharge at outlet will be estimated using Bernoulli's Equation **. -Backwater curve will be drawn along the horizontal profile of trunk drains.

	-Sections that were affected by backwater, their drain discharge capacity was lowered. -Elsewhere, free flow condition was assumed.

*Constructing a backwater curve:

1. Ruhlmann's equation was used (suitable for small projects). Several assumptions were made when using this equation:

a- Flow was assumed to be steady and discharge was known.

b- Slope of channel was assumed to be constant. Channel cross sections assumed to have regular geometric shape.

$$L = t/s\ [\psi\,(h/t) - \psi\,(z/t)] \qquad \text{..............................} (25)$$

L- distance sought between two preassigned depths, namely z. h- is the deepest portion of the pool right behind the tidal gate. t- is the normal depth. ψ- is a function that correspond to z/t and h/t values; precalculated set of value usually given with the equation, however it can be roughly estimated as $\psi = 1.00012$ (z/t or h/t) + 1.344.

t - was calculated by using the equation below:

$$t = Q/(wv) \qquad \text{..............................} (26)$$

Q- discharge in cumecs, for scenario 2, Q equals pumping rate; w- channel width; v-velocity calculated using Manning's equation.

h- was calculated using the equation below:

$$h = (\forall s / w0.5)^{0.5} + t \qquad (27)$$

$\forall$- volume of excess water, s - channel slope.

$\forall$- was calculated by plotting the pumping rate against the hydrograph at the outlet. Overbank flow that occurs at any upstream design points were subtracted from the hydrograph first. The volume below the pumping rate line was subtracted from the total volume.

**Calculating discharges of submerged outlets by using Bernoulli's equation:

Values were automatically computed using a software developed by Archon Engineering.

1- The program obtained a starting flow rate using the orifice equation:

Q = Ca(2gh)^.5 .... (28)

Q - flow rate cfs, C - orifice coef., a - area of orifice, g - 32.2 ft/s^2,h - head from center of orifice.

2- The velocity was calculated from Manning's equation

V=(1.486/n)(R^2/3)(S^1/2) .... (29)

V - Velocity,n - roughness coef.,R - hydraulic radius, S - slope of culvert barrel.

3- The flow area was calculated using Equation 30.

Q = VA .... (30)

A- is the culvert's cross sectional area.

4- From this the normal depth or 'Dn' of flow was obtained. For cases where 'Dn' was greater than the top of the culvert (under pressure), the following form of Bernoulli and Manning's equation was used:

H = (1+Ke)(V^2/2g)+[(V^2)(n^2)(L)/(2.21(r^4/3))] .... (31)

Ke- entrance lost coefficient; depends on shape of entrance. r - hydraulic radius of culvert barrel; V- velocity in the culvert barrel; n- roughness coefficient; depends on the type surface inside barrel, L- length of culvert barrel, g- acceleration of gravity 32.16 ft/sec^2, H- required head was set as the difference between outlet elevation and mean tide level at R.L 0.479m .

Field observation was conducted to identify type of entrance; length of culvert barrel; type, shape and dimension of culvert barrel. Slope of culvert barrel was obtained from the drainage map.

From the above a value for 'V' was calculated and final flow was produced from Q=VA.

For cases where 'Dn' was less than the top of the culvert, the program checked for the critical slope using:

$$A^3/T = Q^2/g \quad \text{.................................} (32)$$

A- flow area, T- width of top of water area, Q - flow rate.

For cases where the slope was not critical, the entrance did not control the flow and open channel flow was used to calculate the capacity. If the slope was equal to or greater than critical, the entrance would control the flow and flow was based on the orifice equation. Note, if entrance controls, the slope of the channel can be continually increased, but the flow rate will not increase. For cases with submerged exit, the program assumed that the culvert was under pressure and critical slope was not an issue.