

## Evaluation of Drainage System of Light Rapid Transport (LRT) Depo – Kelapa Gading – Jakarta City

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### Abstract

LRT Depo is a vital infrastructure in the operation of Jakarta's LRT system. The LRT Depo is located in the Kelapa Gading area. Kelapa Gading is an inundation-prone area in Jakarta. Hence a drainage system should be prepared to manage surface runoff in the area to avoid additional runoff to the surrounding drainage system. In order to reduce runoff in Jakarta Special Province, the Governor of Jakarta has imposed a regulation on surface runoff management for every developed area. The runoff control measures, promoted in the regulation to be applied, are in the form of infiltration wells and storage ponds. The principle of reducing peak discharge by a possible storage system for LRT Depo was designed and applied to comply with regional regulations on rainwater control. The drainage system, initially based on the regulations, was also modeled in the Storm Water Management Modelling software (SWMM). This study evaluated the drainage system by elaborating the reduction of the peak discharge based on the simulation. A reduction of peak discharge was observed in the modeling results. The proposed runoff control at LRT Depo Kelapa Gading is a proper design of infrastructure development for a flood prone area.

**Keywords:** *drainage system; infiltration well; runoff control; storage ponds; stormwater.*

### Introduction

Jakarta, the capital city of Indonesia was established in 1527. It is the largest metropolitan in Southeast Asia. The area of Jakarta is 622 km<sup>2</sup> (0.03 % of the total land area of Indonesia) and is currently populated by 10,374,235 people (3.9% of the total population of Indonesia). The population density is 16,678 people per km<sup>2</sup>. As a capital city, it has high attraction in terms of economic opportunities, which pull people from all over Indonesia and also expatriates. The annual population growth rate in 2017 was 0.94% [1]. Due to the high population density and the limited land resources, Jakarta faces severe problems related to public transportation, urban settlement and housing, and flooding. New public transportation infrastructure is under development, providing special bus lanes, introducing mass rapid transport (MRT), and light rapid transport (LRT) lanes, imposing odd-even number car licensing and also extra inner-city toll-roads. The development of Jakarta as metropolitan area and the situation regarding its growth are well described in [2]. Related to flooding, Jakarta has thirteen rivers crossing its territory. Other than that Jakarta also sees progressing land subsidence [3]. In flood management, Jakarta has two problems, i.e., managing the drainage system and managing flooding that come from rivers. The development of new infrastructure is now emphasized by the government to minimize their runoff in order to reduce the probability of flooding.

The government of Jakarta has imposed a regulation on minimizing surface runoff caused by the development of new commercial and industrial areas. It is mentioned in the regulation that development of undeveloped areas must provide water retention and detention facilities. The government has determined regulations for controlling surface runoff. However, the existing conditions require modifications. This paper aims to demonstrate the application of applicable regulations and adjustments to a location. The area of study is located in the Kelapa Gading area, North Jakarta, Jakarta, Indonesia. This area, shown in Figure 1, is a developed area that consists of commercial area and public housing. Initially, the area was a low-land area with a low elevation. In its past condition, this area had insignificant value in terms of its utilization [4].

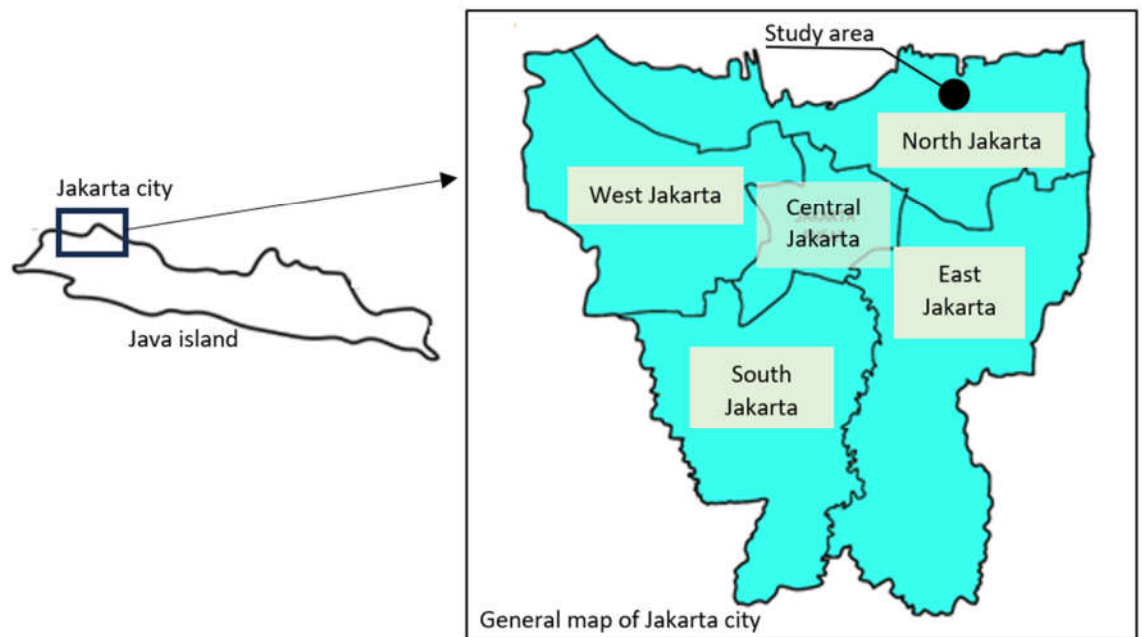


Figure 1 Location of study area.

## Theoretical Background

### Storm Water Management

The management of stormwater runoff can be in the form of retention and detention. Retention process facilities consist of infiltration wells and storage ponds. The conditions required for the retention method are:

1. infiltration wells must be made in the land area of the building concerned;
2. the drainage channel leading to the infiltration well is separated from the sewage channel;
3. infiltration wells must be built in locations where the soil structure is stable and/or not steep;
4. infiltration wells must be made at a minimum distance of 5 m from the location of landfills, former landfills, septic tanks, or soil containing pollutants;
5. infiltration wells must consider the safety of buildings at least 1 m from the foundation;
6. infiltration wells should not be placed under a basement;
7. the depth of the groundwater is at least 1.5 m when it rains.

The water balance at the earth's surface depends on precipitation, evapotranspiration, infiltration, and surface depression storage. The hydrological budget of the earth's surface can be expressed in Eq. (1) as follows [5]:

$$\Delta S = P - (E + T + I + RO) \quad (1)$$

where,  $\Delta S$  = change of surface storage,  $P$  = precipitation,  $E$  = evapotranspiration,  $I$  = infiltration,  $RO$  = surface runoff,  $T$  = transpiration and  $E$  = evaporation. The unit of the variables can be depth of water. For a short time of evaluation and considering a saturated condition due to antecedent rainfall, the expression can be further simplified for a certain time interval  $\Delta t$  in Eq. (2):

$$\frac{\Delta S}{\Delta t} = \frac{P - RO}{\Delta t} \quad (2)$$

For a certain value of precipitation  $P$ , control of storage and surface runoff will be a tradeoff. In order to avoid high storage, the surface runoff will be high and vice versa. To minimize surface runoff, sufficient storage must be provided.

### Peak Discharge

Peak discharge is calculated based on a rational formula in Eq. (3):

$$Q = 0.00278 CIA \quad (3)$$

Q: discharge, m<sup>3</sup>/s

I: rainfall intensity, mm/hours, related to time of concentration

A: catchment area, hectare

The rainfall intensity is determined based on time of concentration. The overland flow time is calculated using the Hathaway equation in Eq. (4) [6]:

$$t_c = \left( \frac{2Ln}{3\sqrt{S}} \right)^{0.47} \quad (4)$$

where:

t<sub>c</sub>: time of concentration for overland flow, minutes

L: length of flow path, feet

S: mean slope of the basin

n: average surface retardance value as shown in Table 1.

**Table 1** Average surface retardance value for the Hathaway equation [7].

Surface	n
Smooth pavement	0.02
Bare packed soil, free of stones	0.10
Poor grass cover or moderately rough surface	0.20
Average grass cover	0.40
Dense grass cover	0.80

### Storm Water Management Model (SWMM)

Rainfall-runoff modeling of a complex drainage system, which consist of several components such as sub-catchments, inflow nodes, channel networks of different geometries, and outfall points, is too complex to be done manually. A drainage system can be modeled as a network of drainage components, such as catchment areas, inlet and outlet points, junction nodes, and reaches, where the simulated based on the physical processes involved. The main physical processes are rainfall, infiltration, ponding, flow along the channel network, and surface runoff generation. A well-recognized rainfall-runoff processes model is the Stormwater Management Model (SWMM). The basic conceptual process includes precipitation, infiltration, evaporation, and runoff over a catchment. A catchment has characteristics of depression storage and infiltration capacity. Balance among the component upon rainfall input, reach capacity, and ponding mechanism determine the runoff resulting from the process [8].

### Methodology

This study aimed to evaluate the capacity of the drainage system at LRT Depo located at Kelapa Gading, North Jakarta. The design rainfall data was obtained from a project report [9]. Sub catchment in the study area was delineated based on the site drainage system layout to determine its contribution towards runoff. The runoff coefficient of the sub catchment was then applied in the calculation of runoff. A rational method was used as the area is considered small, i.e., less than 100 ha [5]. The required volume of detention based on government regulations and the generated runoff were then computed. Data of soil type and groundwater level were also used in the selecting method to control the runoff. The SWMM model was then used to test the performance of detention ponds to store runoff and postpone a peak discharge event. The impact of the detention ponds on the discharged run off at each outlet could be obtained from the model.

### Soil and Hydrological Data

The area layout during the construction stage of the LRT Depot is presented in Figure 2. Most of the surface will be changed into low permeability surface, i.e., around 78%, with a runoff coefficient of 0.9, and open land of around 22%, with a runoff coefficient of 0.5. In this situation, the combined runoff coefficient will be 0.81. Soil investigation results (bore hole) from a previous survey [9] showed that most of the soil can be classified as clay and most of the bore holes showed that the ground water level depth was less than 1.5 m.

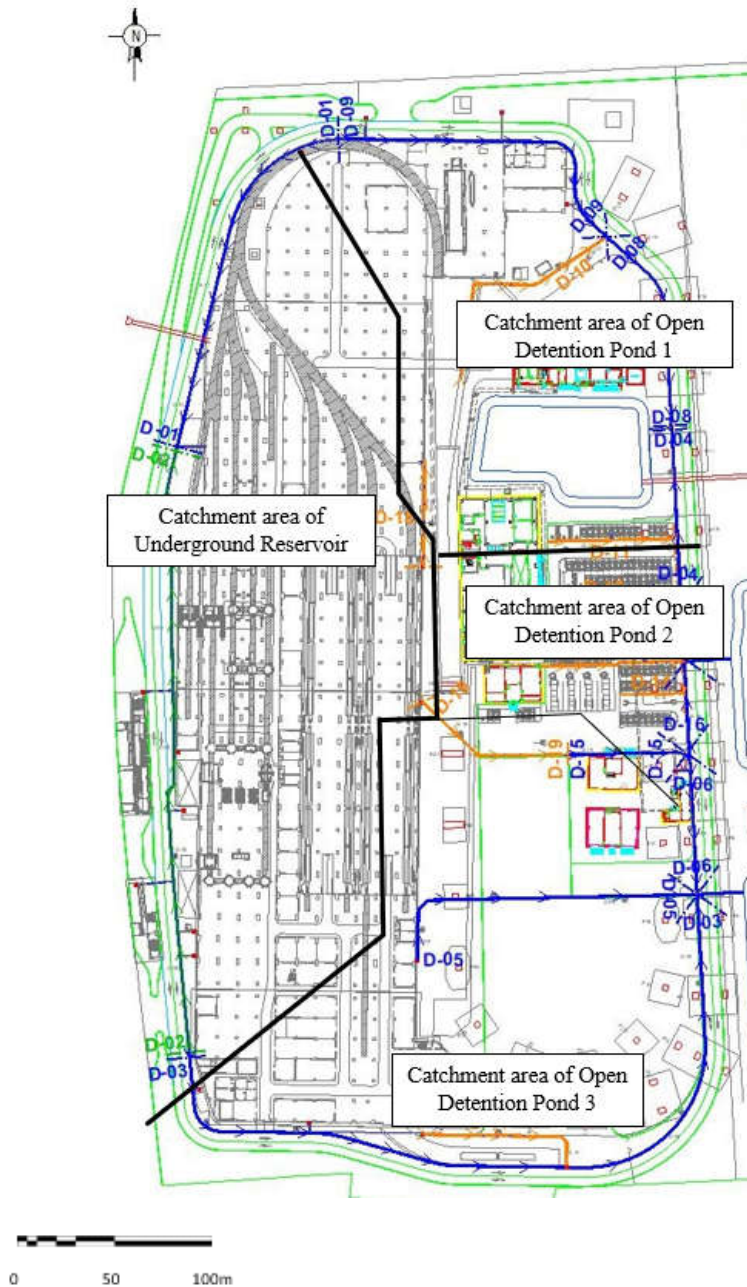


Figure 2 Storm water drainage layout [9].

Rainfall data was obtained from Kemayoran Meteorological Station, located at 8 km from the study area. Log Pearson III distribution was used to evaluate maximum daily rainfall for certain periods, as this distribution fits with Jakarta's rainfall distribution [10]. Table 2 shows the calculated maximum daily rainfall for different return periods for Kemayoran rainfall station [9]. The return periods for drainage designs according to government regulations, which consider city type and catchment area, are shown in Table 3.

**Table 2** Rainfall of certain return periods in mm per day, based on data of years 1980-2014 [9].

Return period (year)	Log Pearson Type III	Return period (year)	Log Pearson Type III
1.01	53	20	211
1.02	57	25	220
1.05	65	50	249
1.11	73	100	280
1.25	84	200	312
2	112	500	357
5	153	1000	393
10	182	10000	527

**Table 3** Return period of urban drainage design (Ministry of Public Works, Regulation No. 12 /PRT/M/2014) [6] in years.

City Type	Catchment Area (Ha)			
	< 10	10 - 100	101 - 500	> 500
Metropolitan	2	2 - 5	5 - 10	10 - 25
Big city	2	2 - 5	2 - 5	5 - 20
Medium city	2	2 - 5	2 - 5	5 - 10
Small city	2	2	2	2 - 5

## Analysis

### Estimated Existing Channel Capacity around the LRT Depot Area

Estimation of the existing channel capacity is required to determine the volume of rainwater that should be held and managed in the LRT Depo area before discharging to the existing external drainage system in the area. The LRT Depot area's proximity/boundaries are: Gading Grande Road (west perimeter), Pegangsaan Dua Road (east perimeter), Kelapa Nias Road (north perimeter) and Gading Grande Residence (south perimeter). However, there are only two channels, i.e. the Pegangsaan Dua Channel and a city drainage channel, that receive discharge directly from the area.

Figures 3 and 4 show the physical conditions of the city drainage channel and the Pegangsaan Dua Channel, respectively. Estimation of discharge into each channel was calculated using a rational method as the area still falls within limits of the method.



**Figure 3** City drainage channel along the east perimeter of the project area. (View towards north direction).



**Figure 4** City drainage channel at the west perimeter of the project area.

The capacity of channel was calculated by the uniform flow formula based on Manning's roughness approach. A summary of the peak discharge and channel estimations are presented in Table 4.

**Table 4** Runoff of adjacent area and capacity of existing channels at east and west boundary.

Boundary	Runoff coefficient	Catchment area, ha	Peak Discharge, m <sup>3</sup> /s	Channel			
				Manning's n	Slope	dimension (width x depth)	Capacity, m <sup>3</sup> /s
Pegangsaan Dua, east boundary	0.90	0.213	0.069	0.015	0.0009	0.80 m x 1.11 m	0.705
City drainage channel, west boundary	0.90	0.289	0.094	0.015	0.0009	0.65 m x 0.87 m	0.418

From the calculation above, the channel capacity is far greater than the burden due to rain. Hence, the available capacity is 0.705 m<sup>3</sup>/s and still has 10% freeboard. However, it should be noted that this area is included in flood-prone areas, so the drainage conditions above will be affected by the condition of the regional drainage system. In order not to become an additional burden when the drainage conditions are flooded, the drainage system within the LRT Depo area is equipped with several detention ponds.

### Conclusion on Existing Condition

The LRT Depot is located in a flood prone area, hence the actual capacity of the channels in the surroundings depends on the drainage system condition. The drainage in the surrounding project area consists of side ditches along Pegangsaan Dua Road, a channel on the south side of Gading Grande Regency, a channel (Kali Betik) at the west side of the project location and a connecting channel (PHB) along Kelapa Nias Road. The connecting channel receives all discharges from the side ditch at Pegangsaan Dua Road and Kali Betik. The existing capacity of the drainage channels are limited, hence the surface runoff from LRT Depot should be managed by detention facilities to control the outflow to the existing drainage system, especially during the rainy season/flood events.

### Stormwater Runoff Management

Based on bore-log data, the ground water level is less than 1.5 meters. Hence, retention wells are not suitable for stormwater runoff management. Another method that is suitable for controlling runoff is by using detention facilities. Detention facilities utilize storage volume to hold stormwater runoff during a rainfall event. The held rainwater is released after the water at the discharge points/channel has receded. Hence the runoff from LRT Depo will not add significant flow when the public drainage channel is at its maximum capacity. This approach will reduce the flood risk in the surrounding area.

Current available ponds that will be utilized as detention ponds are:

1. Underground detention pond : 13,907m<sup>3</sup>
2. Detention Pond 01 : 4,898 m<sup>3</sup>
3. Detention Pond 02 : 2,338 m<sup>3</sup>
4. Detention Pond 03 : 3,110 m<sup>3</sup>

Total capacity of detention = 24,253 m<sup>3</sup>.

To reduce storm water runoff, other than the above facilities, also provided are:

1. Rainwater tanks (86 m<sup>3</sup>)
2. Retention wells (23 units of 2.5 m<sup>3</sup> capacity, total capacity = 57.5 m<sup>3</sup>)

### Catchment Area Delineation

The catchment area inside Depo LRT was divided based on the detention facilities. The detention facilities and the catchment area are presented in Table 5.

**Table 5** Detention facilities and related catchment area in Depo LRT.

Detention Outlet	Volume (m <sup>3</sup> )	Sub Catchment (Ha)	Runoff Coefficient	Total Area (Ha)	Composite Runoff Coefficient
Underground Detention	13,907	1.049	0.600	3.503	0.60
		2.454	0.600		
Open Surface Detention Pond 1	4,898	0.344	0.600	2.060	0.73
		0.370	0.800		
		0.926	0.800		
		0.202	0.600		
Open Surface Detention Pond 2	2,338	0.218	0.600	0.714	0.70
		0.246	0.700		
		0.234	0.700		
Open Surface Detention Pond 3	3,110	0.234	0.700	2.733	0.67
		0.844	0.600		
		1.226	0.700		
		0.663	0.700		

### Peak Discharge at Depo LRT

Peak discharge at Depo LRT was calculated using Equation (3). The ditch flow time component was first calculated based on an estimated flow velocity. A velocity of 0.6 m/s was used as the initial value. This initial value was a first guess to get the time of concentration, which was checked during iteration. The iteration process stopped when the flow discharge at a channel was equal to the runoff discharge obtained from a rational method. The peak discharges before and after development are presented in the following table. The peak discharges were higher than the capacity of the existing city drainage channel, hence, a detention system is required to manage the outflow from the project area.

**Table 6** Peak discharge from project area before and after development.

Detention Outlet	Prior to Development				After Development			
	Q 2 (m <sup>3</sup> /s)	Q 5 (m <sup>3</sup> /s)	Q 10 (m <sup>3</sup> /s)	Q 25 (m <sup>3</sup> /s)	Q 2 (m <sup>3</sup> /s)	Q 5 (m <sup>3</sup> /s)	Q 10 (m <sup>3</sup> /s)	Q 25 (m <sup>3</sup> /s)
Underground Detention	0.53	0.64	0.70	0.76	1.02	1.16	1.20	1.34
Open Surface Detention Pond 1	0.40	0.48	0.52	0.56	0.65	0.76	0.79	0.84
Open Surface Detention Pond 2	0.13	0.16	0.17	0.19	0.23	0.26	0.27	0.29
Open Surface Detention Pond 3	0.46	0.56	0.61	0.66	0.79	0.90	0.94	1.05
Total	1.52	1.84	2.00	2.17	2.69	3.08	3.20	3.52

### Stormwater Drainage

The dimensions of the storm water drainage were calculated based on the rational formula and Manning's uniform flow formula. The IDF curve for a 10-year return period was used in the determination of the intensity. The time concentration was obtained by a similar procedure used for the calculation of the peak discharge. Values of 5 to 30 percent of the channel height are commonly used as reference to determine sufficient freeboard. In the proposed design, 10 percent of channel height is used as minimum freeboard. The discharge from the internal drainage channel will flow to underground detention ponds before being discharged to the external drainage system. Assumption of uniform flow was applied in the flow depth calculation.

### Drainage Modeling

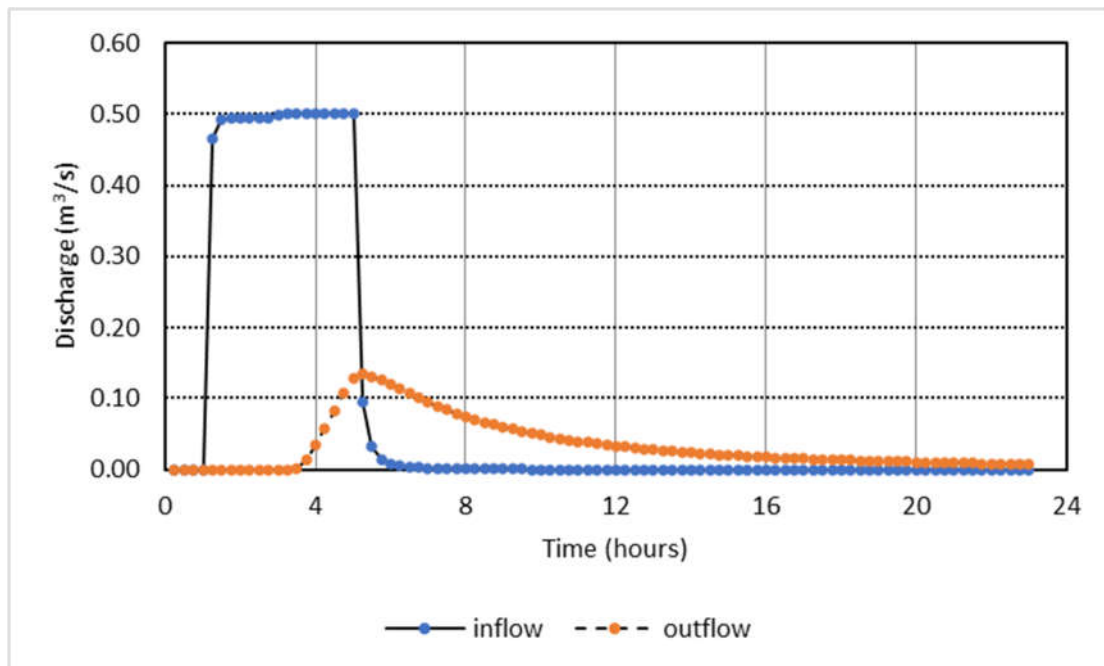
The drainage system was modeled in an SWMM model. The inputs were design rainfall, networks of drainage system and channel characteristics. Sub catchments, channel networks, outfalls were identified, and the network was created accordingly and simulated in SWMM. In the simulation, rainfall was assumed to be



distributed in four hours. Figures 4 and 5 show inflow and outflow of Reservoir 1 and Pond 3, respectively, obtained from SWMM modeling. Table 7 shows the peak ratio of a simulated event in the year 2007 with a maximum daily rainfall depth of 235 mm.

**Table 7** SWMM simulation result for maximum daily rainfall event in the year 2007.

Reservoir or pond	Volume m <sup>3</sup>	Peak Inflow (m <sup>3</sup> /s)	Peak Outflow (m <sup>3</sup> /s)	Peak ratio
Reservoir 1	13,907	0.502	0.135	27%
Pond 1	4,898	0.311	0.288	93%
Pond 2	2,338	0.106	0.095	90%
Pond 3	3,110	0.510	0.506	99%



**Figure 5** Inflow and outflow discharges of Reservoir 1.

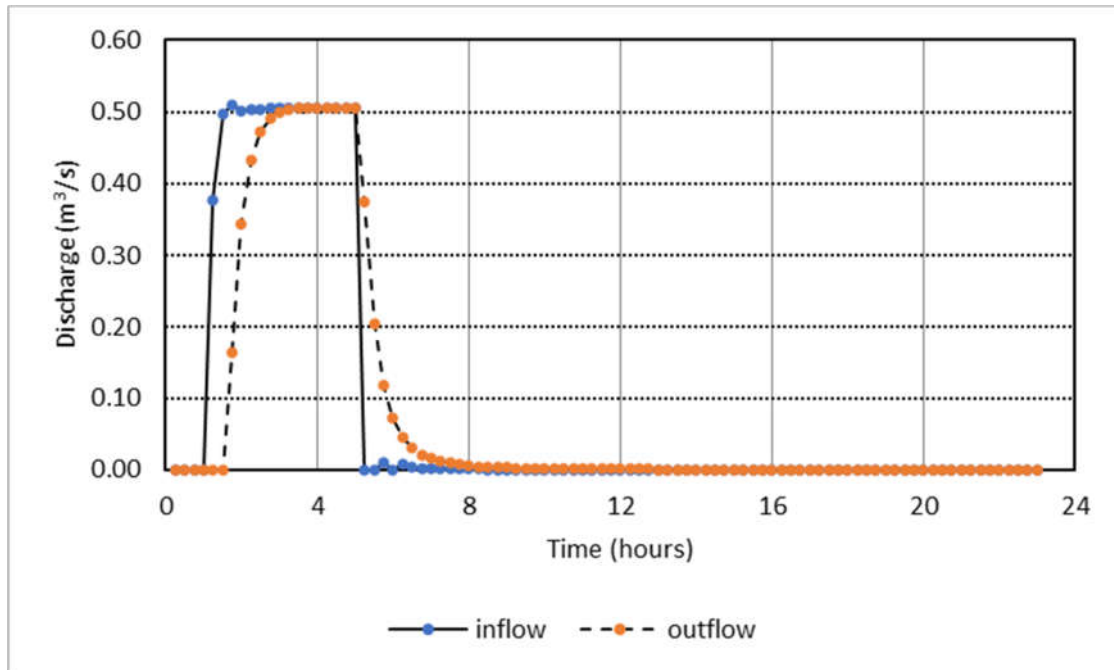


Figure 6 Inflow and outflow discharges of Pond 3.

### Required Volume of Detention Facilities

According to the regulations [11], every building must provide retention/infiltration facilities and detention facilities. In case the local soil condition is not suitable for retention, the required volume for retention is added to the volume required for detention.

1. The volume required for retention is 2 m<sup>3</sup> for 50 m<sup>2</sup> of built area. Hence the required volume for retention is:

$$\text{Required Retention Volume} = \frac{\text{Total built area}}{50} \times 2 = \frac{90,100}{50} \times 2 = 3,604 \text{ m}^3$$

2. The required detention volume is 0.05 m<sup>3</sup> for every m<sup>2</sup> of ground floor area. In this case, the watertight surface is considered part of the ground floor in the calculation. The required detention volume is:

$$\begin{aligned} \text{Required Detention Volume} &= (\text{Ground floor} + \text{water tight area}) \times 0.05 \\ &= 90,100 \times 0.05 = 4,505 \text{ m}^3 \end{aligned}$$

3. As the project area is 90,100 m<sup>2</sup> > 5,000 m<sup>2</sup>, an additional area of 1% of the total project area is required. The possible maximum depth of detention is equal to the ground water level depth. Based on bore hole data, the average ground water depth is 1.5 meters. Hence the volume detention to comply with 1% area is: 1% \* Detention Volume = 1% x 90,100 x 1.5 = 1,351 m<sup>3</sup>.

The total volume required to accommodate all requirements (points a, b, and c) is:  $\text{Total volume} = 3,604 + 4,505 + 1,351 = 9,460 \text{ m}^3$ . The total current detention ponds volume is 24,253 m<sup>3</sup>, which is larger than the volume required by the regulations [12]. Hence, the provided detention ponds fulfill the requirement on the basis of volume provision based on the area criterion.

### Compliance Regarding to Maximum Outflow

According to the triangular shaped inflow hydrograph, the formula to obtain the amount of detention volume is:

$$V_{reference} = t'_b \frac{(Q_2 - Q_1)}{2} \times 3,600 \quad (5)$$

where:

$V_{reference}$ : detention volume as a reference (m<sup>3</sup>)

$t'_b$ : base time of the design hydrograph

$Q_2$ : peak discharge without detention, m<sup>3</sup>/s

$Q_1$ : allowable peak discharge by considering existing city drainage channel capacity, m<sup>3</sup>/s

According to [5], by considering type of region and the total project area, the suggested minimum return period used was 2 to 5 years. In this calculation a return period of 10 years was used. The required detention pond capacity was calculated as follows:

$$V_{req.} = 1.3 \times (V_{reference} - V_{additional}) + \text{Volume of 1\% additional detention area} \quad (6)$$

where:

$V_{req.}$ : required detention volume to comply with peak reduction and the governor's decree [12].

$V_{additional}$ : additional volume of storage other than from detention ponds

$V_{reference}$ : detention volume based on peak discharge reduction

### Comparison Regarding to Rainfall Depth and Detention Capacity

The total volume of the detention ponds is 24,253 m<sup>3</sup>. If this volume is converted to flow depth over the entire project area, it is equivalent to 269 mm of water depth. This value is still higher than for a 50-year return period of rainfall (249 mm). Hence this condition means that the zero-runoff concept is fulfilled with proper detention pond operation.

### Peak Outflow Discharge and Existing City Drainage Channel Capacity

The comparison between peak outflow discharge from the detention ponds and the existing drainage capacity is presented in the following table. The peak outflow from the detention and pond is less than the drainage capacity.

**Table 8** Comparison between existing drainage capacity and peak outflow.

Outlet	Peak outflow (m <sup>3</sup> /s)	Total (m <sup>3</sup> /s)	Connected To	Capacity (m <sup>3</sup> /s)	Check Capacity
Underground Detention	0.32	0.32	City drainage along Gading Grande Road	0.678	Ok
Open Pond 1	0.15				
Open Pond 2	0.04	0.42	City drainage along Pegangsaan Dua Road	0.481	Ok
Open Pond 3	0.23				

### Reservoir Outlet

The required outlet dimensions are calculated based on the flow conditions. For underground reservoirs, the flow calculation is based on the orifice equation in Eq. (7):

$$Q = \frac{\pi D^2}{4} C_d \sqrt{2g(H + D/2)} \quad (7)$$

where,

Q: discharge, m<sup>3</sup>/s

D: pipe diameter, m

Cd: discharge coefficient, 0.6 ~ 0.8  
 H: flow depth over the pipe, m  
 g: gravitational acceleration, m/s<sup>2</sup>

For open detention ponds, the flow calculation is based on flow over broad crested spillways:

$$Q = CLH^{3/2} \tag{8}$$

where,

Q: discharge, m<sup>3</sup>/s  
 C: discharge coefficient, 1.8 ~ 2.2  
 H: flow depth over the weir, m  
 L: width of the weir, m

**Table 9** Outlet dimension calculation.

Type of outlet		Underground Reservoir	Pond 1	Pond 2	Pond 3
		Orifice	Overflow	Overflow	Overflow
Q	m <sup>3</sup> /s	0.32	0.15	0.04	0.23
Flow depth over weir or pipe	m	1.00	0.20	0.20	0.20
C, discharge coefficient	-	0.6	1.80	1.80	1.80
Pipe diameter (minimum)	m	0.48	N/A	N/A	N/A
Width of weir (minimum)	m	N/A	0.96	0.24	1.40

## Conclusion

The project area is located in a flood prone area, hence the storm water in the area should be managed to mitigate the impact on the existing drainage system in the surroundings. Most of the area has a high ground water level, i.e., 1.5 m on average, and the soil is dominated by clay. In this condition, control of excess rainfall by a retention approach is not effective. The runoff discharge without treatment/control is also higher than the existing external drainage system. Hence, a detention system for storm water was chosen as the approach to control the discharge.

The proposed storm water drainage system consists of:

1. Storm water drainage channels
2. Rainwater tanks (86 m<sup>3</sup>)
3. Retention wells (23 units of 2.5 m<sup>3</sup> capacity, total capacity = 57.5 m<sup>3</sup>)
4. Underground detention reservoir (capacity = 13,907 m<sup>3</sup>), with controlled outlet (orifice)
5. Three detention ponds (total capacity = 24,253 m<sup>3</sup>), with controlled outlets (overflow)

The total capacity of the detention storage is equal to 269 mm rainfall falling in the project area. These detention facilities, with proper detention pond operation, may fulfil the zero-runoff concept. The individual volume of the underground detention ponds and retention wells is capable of reducing the peak discharge, and the capacity of the existing city drainage channels is still sufficient to convey the controlled discharge from the ponds. The discharge from the ponds will flow to the city drainage channel in the east and west of the project area.

Analysis of the storm water drainage channels and detention system showed that the designed facilities comply with the current regulations. This study can be used as an example of the application of surface runoff control on a facility located in a flood prone area.

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