

PLANNING OF ECO-DRAINAGE SYSTEM THROUGH THE IMPLEMENTATION OF INFILTRATION WELLS IN THE TUNJUNGSEKAR AREA

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ABSTRACT

The Tunjungsekar area is a densely populated region with residential settlements, facing hydrological problems due to the increase in impervious areas and a conventional drainage system that cannot optimally accommodate the rainfall runoff. To mitigate these issues and maintain groundwater quality to prevent contamination from wastewater, environmentally friendly urban drainage is implemented by constructing infiltration wells. The purpose of this study is to redesign the drainage system, calculate flood discharge, plan the dimensions of new channels, design the dimensions of the infiltration wells, and calculate the required costs. This planning uses rainfall data collected from three stations: Singosari, Ciliwung, and Karangploso from 2015 to 2024. Other data used include topographic maps, site layouts, soil data, and the HSPK Malang city data for 2024. Rainfall data is analyzed using the Log Pearson Type III method with a recurrence interval of 10 years. The design rainfall was obtained at 140.50 mm, and cumulative flow rates ranged from approximately 0.01 m³/sec to 0.07 m³/sec using the rational method. Based on the analysis results, the dimensions of the new rectangular channel were found to be 30 cm and height is 50 cm. The type of channel used is a U-Ditch equipped with inlets, manholes, and culverts. The construction of infiltration wells uses brick masonry with a diameter ranging of 20 cm to 45 cm and a height of 100 cm, totaling 15 units, built scattered in the midst of the channel. The construction cost amounted to Rp509,820,000.00.

Keywords: Eco-Friendly Drainage; Infiltration Wells; Redesigning Channel Planning

1. INTRODUCTION

Tunjungsekar is a dense residential area and a center of economic activity with a variety of supporting infrastructure. The dense population of this area has a problem, namely, the decline in soil absorption of rainwater is one of the environmental problems that arise along with the progress of development.

To solve the problem of rainwater management in the Tunjungsekar area is the eco-drainage system. The concept of eco-drainage allows excess rainwater not to be immediately discharged into the nearest river. Instead, rainwater can be stored in various places in the area concerned in various ways, such as filling or conserving groundwater, improving the quality of the environment and ecosystems, and as a means of reducing flooding.

This research aims to develop an environmentally friendly drainage system in the Tunjungsekar area by implementing infiltration ponds. This research was conducted as an important contribution to sustainable urban environmental management to address local flooding and groundwater conservation issues.

2. METHOD

Consistency Test

The consistency test can be done with the double mass curve formula. This method is carried out by comparing the cumulative rainfall from the station under study with the cumulative average rainfall from several other stations.

$$M = \frac{n \sum xi.yi - (\sum xi)(\sum yi)}{n \sum xi^2 - (\sum xi)^2} \dots\dots\dots (1)$$

$$F = M1/M2 \dots\dots\dots (2)$$

Description

- n = number of years
- xi = cumulative average and combined stations
- yi = cumulative station

Rainfall Area

Using the algebraic average method for large topographic areas, DAS < 500 km². Here is the algebraic method formula:

$$P = \frac{P_1 + P_2 + \dots + P_n}{n} = \frac{\sum_{i=1}^n P_i}{n} \dots\dots\dots (3)$$

Description:

P_1, P_2, \dots, P_n = Rainfall recorded at the measuring post (mm)

P = Rainfall height (mm)

n = Number of measuring posts

Distribution of Design Rainfall

The selection of the distribution method to be used is based on the value of the calculation of the coefficient of skewness (Cs) and the coefficient of kurtosis (Ck)

Table 1. Value Cs And Ck

Normal Distribution	Normal Log Distribution	Distributio n Gumbel	Log Person Type III
0.05<Cs<0.05	Cs = 3.Cv Cs positive	CS<1.1395 Ck<5.4	
0.05<Cs<0.05	Cs = 3.Cv Cs Positive	CS<1.1395 Ck<5.4	No limit

Source : Triadmojo, 2010

$$Cs = \frac{n \sum_{i=1}^n (X_i - \bar{X})^3}{(n-1)(n-2)s^3} \dots\dots\dots (4)$$

$$Ck = \frac{n^2 \sum_{i=1}^n (X_i - \bar{X})^4}{(n-1)(n-2)(n-3)s^4} \dots\dots\dots (5)$$

Description:

S = standard deviation

n = Number of data

X_i = The value of the variance (mm)

\bar{X} = Mean value (mm)

Goodnes Of Fit Test

In the distribution suitability test, there are several methods, including:

a. Metode Smirnov-Kolmogrov

Formula Smirnov Kolmogorov Test, according to (Suripin, 2004):

$$\Delta P = [P(\text{empiris}) - P(\text{teoritis})] \dots\dots\dots (6)$$

b. Metode Chi-Square

$$\chi^2 = \sum \frac{(d \text{ empiris} - d \text{ teoritis})^2}{d \text{ teoritis}} \dots\dots\dots (7)$$

Description:

χ^2 = Parameter Chi-Square

dempiris = d based on the distribution paper

dteoritis = d based on theoretical

Rainfall Intensity

According to Suripin (2004). Rain intensity is proportional to the depth of water per unit time; if the shorter the rain lasts, the more intense the rain. If there is no short-term rain data, it can be made with the formula written as:

$$I = \frac{R_{24}}{24} \left(\frac{24}{t} \right)^{\frac{2}{3}} \dots\dots\dots (8)$$

Description:

I = rainfall intensity (mm/h)

t = rain duration (hours)

R_{24} = maximum rainfall for 24 hours (mm/day)

Concentration Time

The following formula can be used to calculate the concentration time. The concentration time can be calculated by distinguishing into two components, namely (1) the time required to flow T_o . (2) the travel time from first entering the channel T_d , thus

$$t_c = t_o + t_d \dots\dots\dots (9)$$

$$t_o = \left[\frac{2}{3} \times 3,28 \times L_o \times \frac{n}{\sqrt{s}} \right] \dots\dots\dots (10)$$

$$t_d = \frac{L_s}{60V} \dots\dots\dots (11)$$

Description:

t_o = time required for water to flow on the land surface (minutes)

t_d = time required for water to reach downstream (minute)

t_c = rain concentration time (hour)

n = coefficient of resistance (manning)

s = land slope (%)

L_o = length of flow path above the land surface (m)

L_s = length of the path in the channel (m)

V = flow velocity in the channel (m/sec)

Design Flood Discharge

This calculation uses the following formula (Suripin,2004) :

$$Q = C \times I \times A \dots\dots\dots (12)$$

Description:

Q = Design flood discharge (m^3 /sec)

C = flow coefficient

I = Rain Intensity (m/sec)

A = drainage area (m^2)

Infiltration Discharge

Basically, the purpose of infiltration ponds is to give rainwater that falls on roofs or impermeable soil a chance and a way to seep into the ground by collecting it. With storage, rainwater soaks into the ground faster. (Suripin, 2004).

$$Q_o = 5.5 R .K .H \dots\dots\dots (13)$$

k = Soil Permeability Coefficient (m/sec)
 H = Water Level in the well (m)
 R = Radius wells (m)
 Q = Inflow Discharge (m^3/sec)

Channel Dimension Planning

According to, Suripin, (2004). An economical channel cross-sectional is a channel that can pass the maximum discharge for a given wet cross-sectional area, roughness, and slope. Dapat dirumuskan sebagai berikut :

$A = b \cdot h$ (14)

$P = b + 2h$ (15)

$R = A/P$ (16)

Description :

A = cross-sectional area of the channel (m^2)

P = wet perimeter of the channel (m)

R = Hydraulic radius (m)

Hydraulic Analysis

To calculate the channel flow velocity can be calculated using the Manning formula, which is as follows :

$V = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$ (17)

Description:

V = flow velocity (m/second)

R = hydraulic radius

n = roughness coefficient

S = channel slope

The gravitational force and inertial force are represented in the form of a Froude number with the following formulat :

$Fr = \frac{v}{\sqrt{g \cdot h}}$ (18)

Description:

V = Flow velocity (m/second)

g = acceleration of gravity ($m/second^2$)

h = flow depth (m)

The plan discharge is known through the area to be drained of water runoff. It can be calculated with a formula such as (Tri et al. 2021):

$Q = V \times A$ (19)

Cost Budget Plan

Cost Budget Plan is a calculation of the estimated costs required to build a project. Cost Budget Plan usually contains a breakdown of costs for several components, such as materials, labor, equipment, incidental expenses, and other costs associated with the construction. The Cost Budget Plan is usually organized as follows :

Cost Budget Plan= $\sum (VOLUME+UNIT PRICE)$ (20)

The formula for the unit price of work is as follows

HSP = $\sum (Coefficient \times Unit Price)$ (21)

Inlet

Curb inlet dapat dihitung dengan rumus :

$Q = 0.36 g d^{\frac{3}{2}} L$

Description :

Q = inlet curb capacity (m^3/sec)

L = width of inlet kerb opening (m)

g = Gravity (m/s^3)

d = Curb Depth (m)

3. RESULT AND DISCUSSION

Evaluation and re-planning of drainage was carried out in the Tunjungsekar area with an area of 12 hectares and a road width of 4 meters.

Exsisting Condition

The channels on Jl. Ikan Tombro to Jl. Ikan Mas are mostly open channels, but at some points there are closed channels, so the condition of the channels can still be seen. Some channels are in good condition. However, not all channels are in good condition. Many leaf litter, plastic waste, and soil sediment were found inside and outside the channel. To see how the condition of the channel can be seen in the table below

Rainfall Data

Rainfall data was taken from the closest station to the tested area. These stations include ciliwung station, karangploso station, and singosari station. The data used is the maximum rainfall data from 2015 to 2024.

Cinsistency Test

The consistency test at Ciliwung station against Singosari station and Karangploso station shows a break in the graph and the F value is not one. Therefore, it is required to conduct a consistency test. Corrections were made to M2 in the years 2019 to 2014.

Table 2. Cumulative Value of Ciliwung Station (Corrected) and Average Cumulative Value of Singosari Station & Karangploso Station

NO	YEAR	STA CILIWUNG	CUM STA. CILIWUNG	AVG STA. SINGOSARI & STA KALIURANG	CUM AVG STA SINGOSARI & STA KARANGPLOSO
1	2024	2399	2399	2612	2612
2	2023	2440	4839	2064	4676
3	2022	2977	7816	3055	7730
4	2021	2111	9927	1304	9034
5	2020	2427	12354	1408	10441
6	2019	2009	14363	986	11427

7	2018	1938	16301	1272	12699
8	2017	2339	18640	1889	14587
9	2016	2338	20978	2104	16691
10	2015	1348	22325	1118	17809

Source : Calculation 2025



Figure 1 Consistency Test Graph of Ciliwung Station (before corrected) Against Karangploso Station and Singosari Station

Source : Calculation 2025

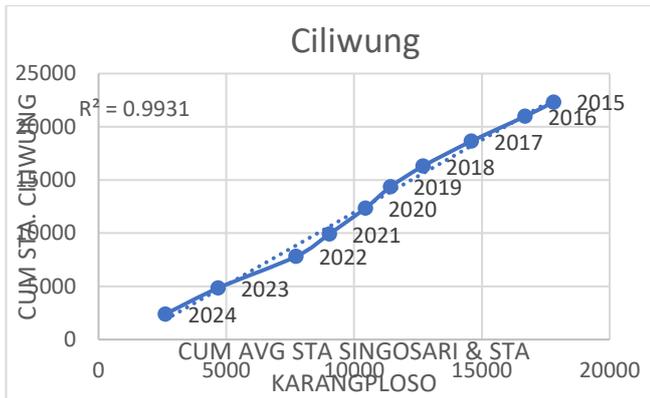


Figure 2. Consistency Test Graph of Ciliwung Station (corrected) Against Karangploso Station and Singosari Station

Source : Calculation 2025

Rainfall Area

In regional rainfall calculations, there are three methods that can be utilized: the algebraic method, the Thiessen polygon method, and the isohyet method. In this study, the algebraic average method is employed due to the watershed area being less than 500 km². The data used in the calculation of regional rainfall is the recap of the maximum annual rainfall from the three stations. The data utilized is that which has been corrected in the consistency test calculations

Table 3 Calculation Rainfall Area

Year	d max
------	-------

2015	105.460
2016	147.368
2017	130.118
2018	128.316
2019	97.604
2020	67.369
2021	94.849
2022	105.817
2023	66.471
2024	113.747

Source : Calculation 2025

Rainfall Design

In this calculation, the value of *s* is 26.0 The value of *C_s* - 0.15 and *C_k* 3.53. Then the calculation of rainfall using log person type III with a return period of 10 years.

Distribution Suitability Test

Two types of methods were used: smirnov-kolmogrov and chi-square. Data obtained from reading the graph of design rainfall test results.

Table 4. Horizontal Variance Test

X sorted	P empiris	P teoritis	Δp
147.37	9%	9%	0.09%
130.12	18%	18%	0.18%
128.32	27%	28%	-0.73%
113.75	36%	37%	-0.64%
105.82	45%	46%	-0.55%
105.46	55%	55%	-0.45%
97.60	64%	64%	-0.36%
94.85	73%	71%	1.73%
67.37	82%	80%	1.82%
66.47	91%	90%	0.91%

Pmax 1.82%

Ptable 41%

Source: Calculation 2025

With α of 5% and a value of *N* = 10, *Do* is obtained at 41%. The calculation results can be concluded as 1.82% < 41% because $\Delta p < Do$, so that the Log Person Type III calculation is acceptable.

Table 5 Vertical Variance Test

X sorted	P empiris	P teoritis	<i>X²hit</i>
147.37	9%	9%	0.950
130.12	18%	18%	0.412
128.32	27%	28%	0.576

113.75	36%	37%	0.128
105.82	45%	46%	0.230
105.46	55%	55%	0.298
97.60	64%	64%	0.020
94.85	73%	71%	0.101
67.37	82%	80%	0.039
66.47	91%	90%	0.033
			2.787

Source : Calculating 2025

With an α of 5%, the X^2 table value is 14.07 Since the X^2 hit $< X^2$ table value, the Log Person Type III distribution method is acceptable.

Rainfall Intensity

In channel number 1-2, it is known that the length of the surface flow path (L_o) for settlements is 11.29 m and the road is 2 m. The coefficient of resistance due to the surface roughness of the drainage area (n_d) is 0.013 for asphalt, concrete, and cement layers. With a channel length of 140.58 m. then the results of the calculation of concentration time and rain intensity are as follows :

Table 6. Calculaion

CALCULATE	RESEDENTIAL	ROAD
to	1.708	0.86
Ld	140.58	140.58
Vd	1.5	1.5
td	1.56	1.562
tc	0.055	0.040
R	140.51	140.51
I	0.00	0.00

Source: Calculating 2025

Design Flood

In channel 1-2, the drainage area of the settlement is 1587.1 m^2 , and the road is 281.2 m^2 . Then the rainwater discharge in channel 1-2 is :

$Q_{cumulative} = Q_{rain} + Q_{waste\ water} + Q_{previously} - Q_{wells}$

Example,

$Q_{cumulative} = 0.0253 + 0.000555556 + 0 - 0.0000091 = 0.0258\ m^3/s$

Waste Water Discharge

For the calculation of dirty water discharge, it is assumed that the occupants of each house are 5 people and the number of settlements is 32. Then it can be calculated in this way:

$Q_{dirty\ water/person} = 300\ liters/person / day$
 $Q_{dirty\ water} = 160 \times 300 = 48000\ liters / day$
 $= 0.000555556\ m^3/s$

Infiltration Wells

Infiltration wells are planned in the center of the channel with a diameter of 75 cm and a depth of 300 cm. The coefficient value is assumed based on lab tests of 0.00000739 m/sec. then the absorption discharge at all points is 0.000640 m^3/sec .

$Q_{infiltrate} = F K H$
 $F = 5.5 R$
 $= 5.5 * 0.75$
 $= 4.25$
 $Q_{infiltrate} = 4.25 \times 0.00000739 \times 3$
 $= 0.0000914\ m^3/sec$

Existing Channel Calculation

For the calculation of the existing channel, it is obtained from the following calculation

For example, channel number 2-3 :

1. Calculating the wet cross-sectional area of a square channel.

$b = 0.43\ m$
 $h = 0.48\ m$
 $A = b \times h$
 $= 0.430 \times 0.480$
 $= 0.206\ m^2$

2. Calculating the perimeter of the wet cross-section of a square channel.

$P = b + 2h$
 $= 0.430 + 2 \times 0.480$
 $= 1,390\ m$

3. Calculating the radius of the channel.

$R = A/P$
 $= 0.206/1390$
 $= 0.148\ m$

4. Calculating the flow velocity of the channel 2-3 Manning hardness coefficient (n) = 0.014 (concrete)

$V = \frac{1}{n} \times R^{\frac{2}{3}} \times S^{0.5}$
 $= \frac{1}{0.014} \times 0.148^{\frac{2}{3}} \times 0.003^{0.5}$
 $= 1,097\ m/s$

5. Calculating the capacity of the discharge flow channel 2-3

$Q = A \cdot V$
 $= 0.206 \times 1.097$
 $= 0.226\ m^3/s$

6. Calculating Froude Value

$Fr = \frac{v}{\sqrt{g \times h}}$
 $= \frac{1,097}{\sqrt{9.81 \times 0.480}}$
 $= 0.506$

7. Channel Control

$$\begin{aligned} Q &= Q_{calc} < Q_{plan} = \text{Ok} \\ V &= 0.2 < V_{calc} < 2 = \text{Ok} \\ Fr &= Fr < 1 = \text{Ok} \end{aligned}$$

So it is possible to redesign the channel 2-3.

New Channel Dimension Planning

By using in situ materials, the channel dimensions are planned with control of the planned discharge.

For example, channel numbers 2-3.

1. Q Plan From Cumulative Discharge
 $Q_{plan} = 0,06 \text{ m}^3/s$
2. Calculating the wet cross-sectional area of a square channel.
 Description :
 $b = 0.30 \text{ m}$
 $h = 0.50 \text{ m}$
 $A = b \times h$
 $= 0.30 \times 0.50$
 $= 0.15 \text{ m}^2$

Calculating the perimeter of the wet cross-sectional area of a square channel.

$$\begin{aligned} P &= b + (2 \times h) \\ &= 0.30 + (2 \times 0.5) \\ &= 1.30 \text{ m} \end{aligned}$$

Calculating the radius of a square hydraulic.

$$\begin{aligned} R &= A/P \\ &= 0.15/1.3 \\ &= 0,12 \text{ m} \end{aligned}$$

3. Calculating Q Capacity.

$$\begin{aligned} Q &= V_{plan} \times A \\ &= 2 \times 0,15 \\ &= 0.30 \text{ m}^3/s \end{aligned}$$

4. Calculating Froude Value.

$$\begin{aligned} Fr &= \frac{Vr}{\sqrt{g \cdot H}} \\ &= \frac{2}{\sqrt{9.81 \cdot 0.5}} \\ &= 0,90 \end{aligned}$$

5. Calculating Slope Plan

Description :

$$N \text{ (Manning's hardness value)} = 0.017$$

$$\begin{aligned} S &= \left(\frac{n \times Vr}{R^2} \right)^2 \\ &= \left(\frac{0.017 \times 2}{0,14^2} \right)^2 \\ &= 0,02 \end{aligned}$$

6. Calculating Δh Of Channel

$$\begin{aligned} \Delta h &= S_{plan} \times L_d \\ &= 0,02 \times 250 \\ &= 5.145 \end{aligned}$$

7. Channel Control Q, V, Dan Fr

$$\begin{aligned} Q &= Q_{plan} < Q_{calc} = \text{OK} \\ V &= 0.2 < V_{calc} < 2 = \text{OK} \\ Fr &= Fr < 1 = \text{OK} \end{aligned}$$

8. Calculating Freeboard

$$\begin{aligned} \text{Freeboard} &= \frac{H}{3} \\ &= \frac{0.5}{3} \\ &= 0.167 \text{ m} \end{aligned}$$

9. Calculating start elevation and end elevation

Start Elevation

Description :

$$\text{Original Starting Elevation} = 487$$

Can Be calculated by

$$\begin{aligned} \text{Water Level} &= \text{Start Elevation} - \text{Freeboard} \\ &= 487.58 - 0.167 = 487.42 \end{aligned}$$

$$\begin{aligned} \text{Base Channel} &= \text{Water Level} - H \\ &= 487.42 - 0.5 = 486.92 \end{aligned}$$

$$\begin{aligned} \text{Top Channel} &= \text{Water Level} + \text{Freeboard} \\ &= 487.42 + 0.167 = 487.58 \end{aligned}$$

End Elevation

Description :

$$\text{Original End Elevation} = 484.79$$

Can be calculated by

$$\begin{aligned} \text{Water Level} &= \text{Water Level Start Elevation} - \Delta h \\ &= 487.42 - 5.145 = 482.27 \end{aligned}$$

$$\begin{aligned} \text{Base Channel} &= \text{Water Level} - \text{End Elevation} - h \\ &= 482.27 - 0.5 = 481.77 \end{aligned}$$

$$\begin{aligned} \text{Top Channel} &= \text{Water Level} + \text{End Elevation} + \text{Freeboard} \\ &= 482.27 + 0.167 = 482.44 \end{aligned}$$

10. Water Level Control

$$\begin{aligned} \text{First Elevation} &= \text{Water level} - \text{original elevation} \\ &= 487.42 - 487.58 = -0.17 \end{aligned}$$

$$\begin{aligned} \text{Last Elevation} &= \text{Water level} - \text{original last elevation} \\ &= 482.27 - 484.79 = -2.52 \end{aligned}$$

To control the water level, the water level value must be less than one.

$$\text{First Elevation} < 1 \sim -0.17 < 1$$

$$\text{Last Elevation} < 1 \sim -2.52 < 1$$

It can be concluded that the water level in channels 2-3 adequate.

Inlet

An inlet is a supplementary structure used as an entry waterway for flooding. In this planning, the type of inlet used is a kerb inlet. To determine the dimensions of the inlet and how many inlets are required for a particular channel,

calculations can be made as follows. For example, channel number 2-3.

$$Ld = 174.8 \text{ m}$$

$$\text{Opening width} = 0.3 \text{ m}$$

$$\text{Lo Road} = 2 \text{ m}$$

$$\text{Road Slope} = 0.2$$

$$\text{Water Level (d)} = \text{Lo} \times \text{S}$$

$$= 2 \times 0.2$$

$$= 0.040 \text{ m}$$

$$Q \text{ road} = 0.026 \text{ m}^3/\text{s}$$

Discharge that can be collected.

$$Q_{\text{inlet}} = 0.36 \times g \times d^{\frac{3}{2}} \times L$$

$$= 0.36 \times 9.81 \times 0.04^{\frac{3}{2}} \times 0.2$$

$$= 0.057 \text{ m}^3/\text{s}$$

$$\text{Number of Inlets Needed} = Q \text{ road} / Q \text{ inlet}$$

$$= 0.006 / 0.02$$

$$= 5 \text{ pcs}$$

$$\text{Distance Between Inlet} = \frac{Ld}{n}$$

$$= \frac{174.8}{5} = 35 \text{ m}$$

4. CONCLUSION

Based on the re-planning of drainage in the Tunjungsekar area of Malang city, it can be concluded that :

1. Based on the survey results, existing channels are in poor condition, and there is still a lot of garbage and sedimentation that settles. So, if there is heavy rain, there are still several points that experience excessive inundation, which results in potholes.
2. The result of return period flood of 10 yr is 140.5 m³/dt.
3. From the construction of infiltration well analysis, the % reduction of flood is 0.026%
4. The total cost required in this planning is Rp.509,820,000.00.

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