JOURNAL OF GREEN SCIENCE AND TECHNOLOGY

SURFACE FLOW ANALYSIS AS AN EFFORTS FLOOD MITIGATION

Awliya Tribhuwana [1] , Fathur Rohman [2] , Ohan Farhan [3]

[1]) Civil Engineering Study Program , Swadaya University Gunung Jati, Cirebon.
Corresponding Email: tribhuwana69@gmail.com

ABSTRACT

Problems that often arise in the city of Cirebon are flooding caused by rain, land changes not supported by adequate infrastructure, narrowing of drainage channels and sedimentation of channels. This is the location where flooding often occurs in the Pemuda Street area and its surroundings. Therefore, it is necessary to carry out research on surface flow analysis as an effort to handle floods with the aim of inundation management strategies to reduce excessive rainwater runoff in drainage areas and channels. The research methods used were problem identification, literature study, data collection, analysis and design planning. The results can show that inundation is handled in two ways, first by changing the dimensions of the channel at the initial height H to 0.45m, it still experiences inundation, this is due to the difference in elevation which causes inundation in the channel. There was a change in the flood inundation area to 23,127 ha from the original 58,958 ha, meaning that the flood free area increased from 9,388 ha to 35,831 ha. The second way is by making 378 infiltration wells spread across 34 channels that experience flooding. With these infiltration wells, the flooding can be reduced to a minimum from an area of 23,127 ha to 0.040 ha. The success rate for flood management efforts reached 99.908%, with failure being 0.092%. Of the area of 68,346 ha, the flood-free area reached 68,306 ha and the remaining inundated area was 0.040 ha..

Keywords: *Rain , Runoff , Drainage , Wells Absorption*

1. INTRODUCTION

Population development is accompanied by an increasing number of infrastructure developments, causing more and more land surface to be covered by land changes. Population development that ignores land use functions causes excessive surface flow [1]. Changes in land use function cause rainwater which should be able to infiltrate [2] into the soil and become additional groundwater reserves [3] which cannot occur, because rainwater becomes surface flow and is directly discharged into drainage channels [4]. The accumulation of rainwater will exceed the drainage capacity, resulting in flooding [4].

One of the impacts of changes in land use is a direct increase in surface runoff [5], reducing water seeping into the ground. As a result, water distribution becomes increasingly uneven between the rainy season and the dry season, flood discharge increases and the threat of drought occurs [6]. Efforts are made to control and reuse rain runoff that occurs during the rainy season in an area, including efforts to infiltrate it back into the soil. One of the steps used to manage runoff is by using infiltration wells. Infiltration wells are wells or holes made to collect rainwater or surface water flow so that it flows to the ground so that it can maintain or even increase the ground water level and reduce the rate of surface runoff because the water is directly absorbed [7].

Infiltration wells are an alternative for groundwater conservation and minimize surface flow, this is because infiltration wells are easy to apply and are expected to be able to maintain a balance in

 P- ISSN: 2598-1277
E- ISSN : 2621-3966

groundwater use [8]. Infiltration wells are wells that are created to store excess rainwater so that it has time and space to seep into the ground through an infiltration process.

One of the areas that experiences problems with inundation and flooding due to excessive rainwater runoff when the intensity of rain is high is Jalan Pernuda, Cirebon City. The problem that often occurs is flooding and inundation every rainy season caused by the capacity of drainage channels which are no longer able to accommodate the runoff and discharge. due to changes in land use. With these conditions, this research aims to carry out analysis and identification of areas that will be reviewed by implementing infiltration wells. It is hoped that this research can contribute to the community and the Government in implementing the infiltration well technique as an effort and strategy to handle inundation and flooding in reducing excessive rainwater runoff in drainage channels.

Based on the description of the background of the problem above, the problem in this research can be formulated, namely: How to deal with floods which often occur in every rainy season, especially on Jalan Pemuda Cirebon City, apart from reconstruction of the channel dimensions. This research has the following objectives, First, to carry out analysis and identification of the areas that will be reviewed by implementing infiltration wells, Second, Absorption wells as one of the efforts and strategies to handle inundation and flooding in reducing excessive rainwater runoff in drainage channels, Third, To find out the number of infiltration wells needed to be used as an effort to deal with flooding problems that often occur on Jalan Pemuda.

2. LITERATURE REVIEW

2.1 Analysis Hydrology

Hydrological analysis not only requires the volume and height of rain, but also the distribution of rain over time and place. In hydrological analysis and planning, the characteristics of rain are carefully reviewed, including:

a. Intensity I, is the rate of rain or water height per unit of time, for example mm/minute, mm/hour, or mm/day.
b. Length of time (duration) t, namely the length of time during which rain falls in minutes or hours.
c. Rain height d, namely the amount or depth of rain that occurs during the duration of the rain, is expressed in terms of the thickness of the water above the surface in mm.
d. The frequency of events is usually expressed in terms of a return period T, for example once every 2 years.
e. Area is the geographical area of the rain distribution area.

2.2 Analysis Frequency and Probability

Frequency analysis is an analysis of the recurrence of an event, both the number of frequencies per unit of time and the return period. To analyze planned rainfall, existing hydrological data from an event consists of several theories that suggest similarities regarding the analysis. The planned rainfall calculation uses rainfall data with a certain return period which is calculated using 4 frequency distribution methods, namely:

1. Normal Distribution
2. Log Normal Distribution
3. Log Pearson Distribution III
4. Gumbel Distribution

Frequency distribution is used to determine the relationship between the magnitude of extreme hydrological events such as floods and the number of events that have occurred so that the probability of extreme events over time can be predicted [9]. Data analysis carried out using the four methods includes the average, b deviation coefficient of variation, skewness coefficient and kurtosis coefficient.

P- ISSN: 2598-1277
E- ISSN : 2621-3966

2.3 Testing Compatibility Distribution

The results obtained for. of these four methods, then a suitability test was carried out using the Smimov-Kolmogorov method or non-parametric suitability test. This suitability test is used to determine the design rainfall values from the four frequency distribution methods that are most suitable for use at the research location.

2.3.1 Square Test

$$X^2 = \sum_{i=1}^{G} \frac{(Ef - Of)^2}{Ef} \quad \ldots \ldots 1$$

Condition: Mark X^2 must < from X^2 CR

2.3.2 Smirmov Kolmogorov test

The Smimov-Kolmogorov goodness-of-fit test is often called a non-parametric goodness-of-fit test because the test does not use a specific distribution function..

Condition: Δ max < Δ critical

2.4 Rainfall Intensity

Formula Mononobe :

$$I = \frac{R_{24}}{24} * \left[\frac{24}{t}\right]^{\frac{2}{3}} \quad \ldots \ldots 2$$

Where :

$I \quad = Rainfall\ intensity \left(\frac{mm}{hour}\right)$
$R_{24} = Cmaximum\ rainfall\ in\ 24\ hour\ (mm)$
$t \quad = duration\ of\ rainfall\ (hour)$

2.5 Planned Flood Discharge

Rational Method

$$Q = 0.278 \text{ X } C \text{ X } I \text{ X } A \quad \ldots \ldots 3$$

$$I = \frac{R_{24}}{24} * \left[\frac{24}{t_c}\right]^{\frac{2}{3}} \quad \ldots \ldots 4$$

$$t_c = \left(\frac{0{,}87 x L^2}{1000 x S}\right)^{0{,}385} \quad \ldots \ldots 5$$

Where :

$Q \quad = maximum\ water\ discharge \left(\frac{m^3}{second}\right)$
$A \quad = Watershen\ Area\ (Km)^2$
$C \quad = runoff\ coefficient$
$I \quad = rainfall\ intensity\ during\ the\ concentration\ time \left(\frac{mm}{hour}\right)$

 P- ISSN: 2598-1277
E- ISSN : 2621-3966

R_{24} $= 24 - hour\ daily\ maximum\ rainfall\ (mm)$
t_c $= concentration\ time\ (hour)$
L $= length\ of\ the\ main\ channel\ from\ upstream\ to\ downstream\ (km)$
S $= average\ slope\ of\ the\ inner\ channel$

2.6 Dimensions Channel

The cross-sectional shape of the channel used is a trapezoidal shape, the capacity of the channel is determined by the area being irrigated. In general, the bottom width of the channel (b) is taken to be greater than or equal to the depth of the channel (h), with the aim of preventing silting in the channel when water flows through the channel. In this study, an open channel type with a trapezoidal shape was used.
Flow velocity (V) can be calculated using the Manning Formula, namely:

$$V = \frac{1}{n} * R^{\frac{2}{3}} * S^{\frac{1}{2}} \dots \dots \dots \dots \dots \dots \dots \dots \quad 6$$

Where :

V = Flow speed (m/sec)
R = Hydraulic radius (m)
A = wet cross-sectional area (m^2)
P = Wet perimeter (m)
n = Manning roughness coefficient
S = Slope of the channel bottom
b = Channel width (m)
m = Slope of the embankment (1 vertical : m horizontal)
h = Water height (m)

2.7 Channel Capacity

The channel capacity calculation can be estimated by the amount of runoff on a piece of land or based on the planned discharge. Channel capacity is influenced by two factors, namely cross-sectional area and flow speed. The flow velocity is determined by the slope of the channel, the hydraulic radius (the quotient between the cross-sectional area and the channel parameters) and the roughness coefficient of the channel.

2.8 Slope Channel (S)

The slope of the channel bed is generally determined by the topography of the land or the desired height and end of the channel. The slope of the channel bed depends on the material forming the channel. The formula used to calculate the slope of the channel is Robert Manning's formula, namely:

$$S = \left(\frac{Q}{(A*R)^{2/3}}\right)^2 \dots \dots \dots \dots \dots \dots \dots \dots 7$$

Continuity formula:

$$Q = A \,.\, V \dots \dots \dots \dots \dots \dots \; 8$$

Where :

A = Wet cross-sectional area of the channel (m2)
V = Flow speed in the channel (m/sec)
R = Hydraulic radius of the channel (m)
M = Slope of the Talud
n = Manning roughness coefficient
h = Water height in the channel (m)
S = Slope of the channel bottom
Q = Flow rate (m3/sec)

 P- ISSN: 2598-1277
E- ISSN : 2621-3966

2.9 Freeboard.

Freeboard is the vertical distance from the top of the embankment to the water surface under design conditions. The freeboard in the drainage channel is planned to avoid runoff, at the highest possible surface elevation of the water flow, plus the height of the waves and the possibility of floating objects in the flow. Freeboard varies according to the size and location of the channel, soil type, amount of water entering due to rain and rise in water level due to control structures. The formula used to obtain freeboard is:

$$W = (c * h)^{0,5} \ldots\ldots 9$$

For Q < 0.8 m3/sec ; c = 0.14
0.8 m3/s ≤ Q ≤ 8 m3/s ; c = 0.14 – 0.23
Q ≥ 8 m3/sec ; c = 0.23

Or you can also use the formula:

$$W = 0{,}25h + 0{,}3 \ldots\ldots 10$$

Where :

w = Freeboard (m)
h = Water height in the channel (m)
c = Coefficient that depends on discharge

2.10 Hydraulic Radius.

The hydraulic radius is the ratio between the area of the channel's wet reservoir and the channel's perimeter. The formula used is:

$$R = \frac{A}{P} \ldots\ldots 11$$

Where :

R = hydraulic radius (m)
A = Wet cross-sectional area of the channel (m2)
P = Wet perimeter of the channel (m)

2.11 Infiltration Wells

The depth of the infiltration well can be calculated using the Sunjoto formula, 1988. The formula used is as follows

$$H = \frac{Q}{F.K}\left[1 - e\ \frac{F.K.T}{\pi.R^2}\right] \ldots\ldots 12$$

Where :

$H = Water\ level\ height\ in\ well\ (m)$
$F = geometric\ factors\ (m)$
$Q = incoming\ water\ discharge\ \left(\frac{m^3}{second}\right)$
$T = flow\ time\ (second)$
$K = soil\ permeability\ coefficient\ (m/sec)$
$R = jari - jari\ (m)$

Calculating the volume of the well absorption cross section circular

 P- ISSN: 2598-1277
E- ISSN : 2621-3966

$$V_{sumur} = \pi * R^2 * H \quad \ldots \quad (13)$$

Where :

$V_{sumur} = well\ volume\ (m)^3$
$R = jari\ jari\ sumur\ (m)$
$H = well\ height\ (m)$

Count amount well absorption

$$Jumlah\ sumur\ resapan = \frac{V_{total\ sumur}}{V_{rencana}} \quad \ldots \quad (14)$$

Calculate the incoming water discharge well absorption

$$Q_{sumur\ rencana} = \frac{F * K * H_{rencana}}{1 - e^{\frac{F*K*T}{\pi*R^2}}} \quad \ldots \quad (15)$$

Calculate the infiltration discharge well

$$Q_{resap} = F * H * K \quad \ldots \quad (16)$$

Count time absorb well absorption

$$t_{resap} = \frac{V_{sumur}}{Q_{resap}} \quad \ldots \quad (17)$$

Count time charging well absorption

$$t_{isi} = \frac{V_{sumur}}{Q_{sumur\ renc}} \quad \ldots \quad (18)$$

3. METHODOLOGY

The research location is on Jalan Pemuda, Cirebon City with an area of 1 = 301,405.23 m2, 2 = 165,581.63 m2, 3 = 115,134.69 m2, and 4 = 128,191.84 m2, with land used for offices, campuses, supermarkets and space. open. Types of research are quantitative and qualitative. The data used in this research are primary data and secondary data, primary data was obtained from direct observations and measurements in the field [10], while secondary data was obtained from related agencies.

Figure 4.1. Research sites

P- ISSN: 2598-1277
E- ISSN : 2621-3966

After all the necessary data has been collected, analysis can be carried out. The rainfall obtained was analyzed using frequency analysis to obtain a suitable distribution, the frequency analysis used was the Normal Distribution method, Log Nomial Distribution, Log Person III, Gumbel method. After obtaining a suitable distribution, the next step is to test the distribution suitability, which in this research uses the Smimov-Kolmogorov test. The next step is to determine the intensity of the rain, the equation that can be used is the Talbot, Sherman, Ishiguro equation. The results of this equation then depict the IDF curve. Then the next stage is to determine the planned flood discharge, the method used is the National method. This method requires the value of the runoff coefficient, rain intensity, and area of the rain catchment area. After the discharge data is obtained, the next step is to look for the capacity of the channel, handle runoff and the need for infiltration wells and discuss it.

4. RESULTS AND DISCUSSION

4.1 Planned Discharge Analysis

In analyzing the planned discharge using the modified rational method by taking into account land use and topography, several things that influence the magnitude of the planned discharge are area (A), channel length (Ls), furthest flow length over the land (Lt), land slope (St), time required to flow from the land surface to the nearest channel (to), flow time in the channel (td), water collection time or concentration time (tc), rainfall intensity (I), drainage coefficient (C), concentration coefficient factor (Cs), flow velocity in the channel (V), then the planned discharge is produced as in the table below. If there is a merger in a channel then the planned discharge is added with additional discharge to produce a combined planned debut as in numbers 3,4,6,7,8 and 10 (see table 1)

Table 4.1. Plan Debit Genre Surface

No	Channel	Area served (Ha)	Q Runoff (m3/s)	Additional Q (m3/s)	Q Total (m 3.s)
1	A11-A12	0.3977	0.2607	-	0.2607
2	A12-A13	0.3977	0.1519	-	0.1519
3	A13-A14	0.3977	0.1966	0.044	0.2406
4	A14-A11	0.3977	0.1454	0.070	0.2154
5	A21-A22	1.5209	0.3306	-	0.3306
6	A22-A23	1.5209	0.4483	0.031	0.4793
7	A23-A24	1.5209	0.5831	0.244	0.8271
8	A24-A21	1.5209	0.7335	0.298	1.0815
9	A31-A32	0.3030	0.1137	-	0.1137
10	A32-A33	0.3030	0.2062	0.362	0.5672

Source : Calculation results

4.2 Planned Discharge Analysis Channel Existing

To find out if a channel or area is experiencing flooding, an analysis must be carried out between the planned discharge in table 4.1. with the existing channel discharge in table 4.2, from the two tables for number 1 it states that the channel discharge capacity is 0.8873 m3/second which is greater than the planned discharge of 0.2607 m3/second, meaning that the channel does not experience runoff or overflow so that the area being around the channel does not experience flooding.

P- ISSN: 2598-1277
E- ISSN : 2621-3966

Table 4.2. Channel Capacity Existing

No	Channel	Care (F)	Dimensions B	Dimensions H	Cross-sectional area A(m2)	Capacity Discharge (m3/ sec
1	A11-A12	0.2	0.3	0.36	0.048	0.8873
2	A12-A13	0.2	0.3	0.36	0.048	0.9927
3	A13-A14	0.2	0.3	0.40	0.060	0.6344
4	A14-A11	0.2	0.3	0.36	0.048	0.7745
5	A21-A22	0.2	0.3	0.36	0.048	0.2449
6	A22-A23	0.2	0.3	0.36	0.048	0.6707
7	A23-A24	0.2	0.3	0.32	0.036	0.6955
8	A24-A21	0.2	0.3	0.23	0.009	0.0876
9	A31-A32	0.2	0.9	0.30	0.090	0.9863
10	A32-A33	0.2	0.2	0.30	0.020	0.2953

Source : Calculation results

After get channel discharge existing done comparison with planned debit , results can seen in table 4.3

Table 4.3. Comparison of Plan Discharge and Channel Discharge Existing

No	Channel	Q Plan (m3/ sec) Q Runoff	Q Plan (m3/ sec) Additional Q	Q Plan (m3/ sec) Q Total	Q Channel Existing (m3/ sec)	Q Inundation (m3/ sec)	Condition Channel
1	A11-A12	0.2607	-	0.2607	0.8873	0	OK
2	A12-A13	0.1519	-	0.1519	0.9927	0	OK
3	A13-A14	0.1966	0.044	0.2406	0.6344	0	OK
4	A14-A11	0.1454	0.070	0.2154	0.7745	0	OK
5	A21-A22	0.3306	-	0.3306	0.2449	0.0857	Overflow
6	A22-A23	0.4483	0.031	0.4793	0.6707	0	OK
7	A23-A24	0.5831	0.244	0.8271	0.6955	0.1316	Overflow
8	A24-A21	0.7335	0.298	1.0815	0.0876	0.9439	Overflow
9	A31-A32	0.1137	-	0.1137	0.9863	0	OK
10	A32-A33	0.2062	0.362	0.5672	0.2953	0.2719	Overflow

Source : Calculation results

Table 4.3 at the location of channels No. 5, 7, 8 and 10 experienced flooding or the channel had runoff. The first step taken in handling floods was to make changes to the dimensions of the channel, namely by increasing the height at the initial H to 0.45m, for the width this was not done. changes due to the condition of the area no longer having free land.

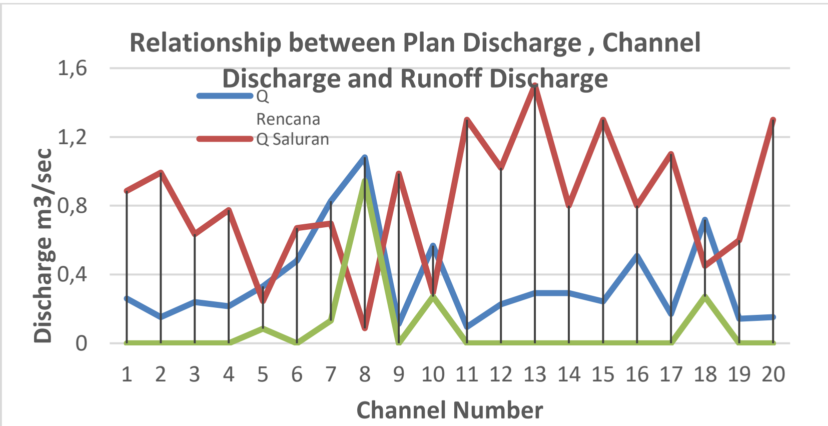

Figure 4.1. Chart Analysis of Plan Discharge , Channel Discharge and Runoff Discharge Condition Existing

P- ISSN: 2598-1277
E- ISSN : 2621-3966

4.3 Analysis Change Dimensions Channel Existing with H=0.45

The dimensions of channels in the research area generally have a height H that varies from 0.3m to 0.4m, rectangular channels with freeboard F=0.2m. The existing A21-A22 channel has B=0.3m, H=0.36m, y=0.16 and the channel capacity is only Q=0.2449 m3/second, while the runoff discharge is 0.3306 m3/second as a result the channel does not can accommodate so that runoff or overflow occurs. Handling to prevent runoff occurs is changing the dimension H=0.45m with the result that the channel capacity becomes 0.4347 m3/second, see table 4.4. on numbers 5,7,8 and 10.

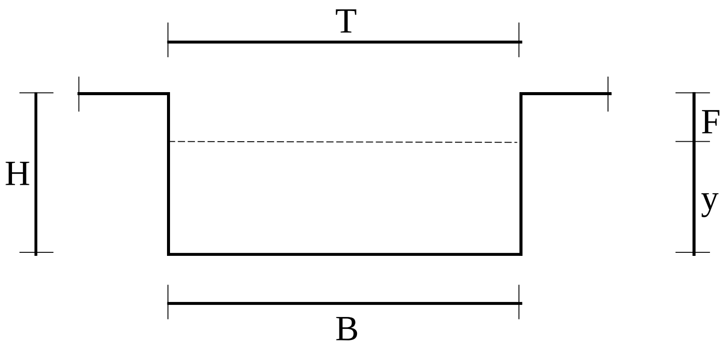

Figure 4.2. Dimensions Channel Existing

Table 4.4. Analysis Change Dimensions Channel Existing with H=0.45m

No	Channel	Care (F)	Dimensions B	Dimensions H	y	Cross-sectional area A(m2)	Capacity Discharge (m3/ sec
5	A21-A22	0.2	0.3	0.45	0.16	0.075	0.4347
7	A23-A24	0.2	0.3	0.45	0.12	0.075	1.8187
8	A24-A21	0.2	0.3	0.45	0.03	0.075	1.7621
10	A32-A33	0.2	0.2	0.45	0.10	0.050	0.9364

Source : Calculation results

From the results of changing the channel dimensions, it can be seen that if the planned discharge is smaller than the channel discharge then no runoff will occur. See table 4.5 results of changing dimensions

Table 4.5. Comparison Results of Planned Discharge with Channel Discharge Existing (H=045m)

No	Channel	Q Plan (m3/ sec) Q Runoff	Q Plan (m3/ sec) Additional Q	Q Plan (m3/ sec) Q Total	Q Channel Existing (m3/ sec)	Q Inundation (m3/ sec)	Condition Channel
5	A21-A22	0.3306	-	0.3306	0.4347	0	OK
7	A23-A24	0.5831	0.244	0.8271	1.8187	0	OK
8	A24-A21	0.7335	0.298	1.0815	1.7621	0	OK
10	A32-A33	0.2062	0.362	0.5672	0.9364	0	OK

Source : Calculation results

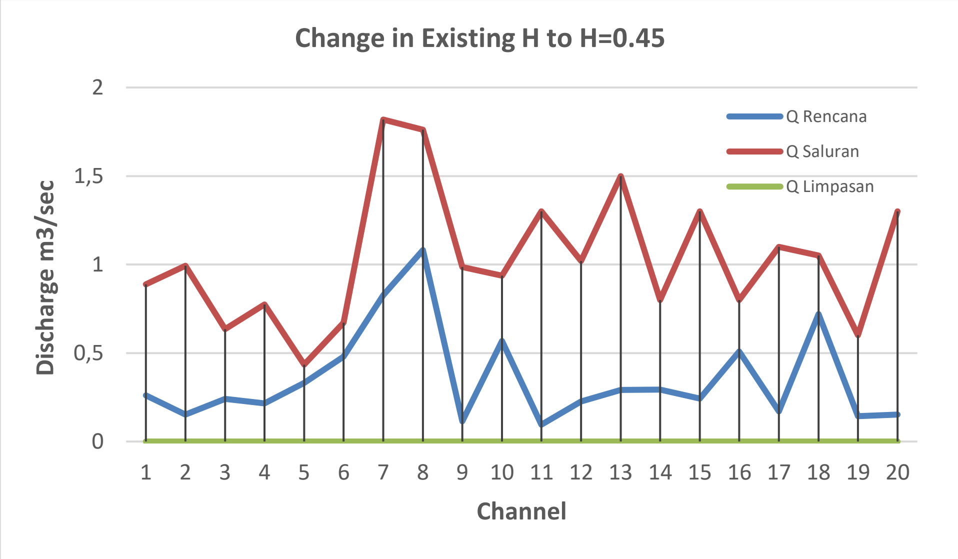

Figure 4.3. Chart Analysis of Plan Discharge, Channel Discharge and Runoff Discharge Condition Change Dimension H=0.45

In figure 4.3. on state that after done change dimension H=45m power accommodate channel can accommodate planned debits without There is runoff .

4.4 Analysis Handling With Well Absorption

In figure 4.3. The above states that after changing the dimensions of H=45m, the channel's capacity can accommodate the planned discharge without any runoff.

Table 4.6. Recapitulation of channels experiencing runoff after H=0.45

No	Channel	Q Plan (m3/s)	Q Capacity (m3/s)	Q Puddle (m3/s)	Q Total (m 3.s)
26	A7.2-A7.3	0.987	0.515	0.472	Overflow
32	A8.4-A8.1	0.907	0.515	0.391	Overflow
40	A.10.4-A10.1	0.971	0.592	0.378	Overflow
48	A.12.4-A12.1	1,117	0.541	0.577	Overflow
56	A.14.4-A14.1	1,246	0.65	0.595	Overflow
60	A.15.4-A15.1	1,336	0.399	0.937	Overflow
63	A.16.3-A16.4	0.598	0.541	0.057	Overflow
64	A.16.4-A16.1	0.970	0.437	0.534	Overflow
65	A.17.1-A17.2	2,030	0.684	1,347	Overflow
79	A.20.3-A20.8	0.521	0.456	0.065	Overflow
97	A.20.1-A20.2	1,952	1,065	0.887	Overflow
98	A.20.2-A20.3	0.495	0.194	0.301	Overflow
109	B3.1-B3.2	1,007	0.999	0.008	Overflow
114	B4.2-B4.3	0.390	0.367	0.023	Overflow
129	B8.1-B8.2	1,283	0.282	1,001	Overflow
130	B8.2-B8.3	1,410	0.354	1,057	Overflow
134	B9.2-B9.3	1,483	0.779	0.703	Overflow
135	B9.3-B9.4	1,577	1,345	0.231	Overflow
142	B11.2-B11.3	0.510	0.312	0.198	Overflow
152	B13.4-B13.1	0.676	0.595	0.081	Overflow
158	B15.2-B15.3	0.396	0.209	0.187	Overflow
166	B17.2-B17.3	0.915	0.645	0.27 0	Overflow
182	B21.2-B21.3	1,184	0.636	0.548	Overflow

No	Channel	Q Plan (m3/s)	Q Capacity (m3/s)	Q Puddle (m3/s)	Q Total (m 3.s)
191	C1.3-C1.4	1,732	1,163	0.57 0	Overflow
195	C2.4-C2.1	1,600	0.772	0.828	Overflow
196	C3.1-C3.2	0.443	0.375	0.069	Overflow
200	C4.1-C4.2	1,633	1,001	0.633	Overflow
204	C5.1-C5.2	0.555	0.543	0.012	Overflow
208	C6.1-C6.2	0.66	0.519	0.141	Overflow
252	D1.2-D1.3	1,478	1,444	0.034	Overflow
261	D1.4-D1.1	1,310	0.708	0.602	Overflow
269	D1.4-D1.1	1,781	1,118	0.663	Overflow
270	D1.1-D1.2	1,775	0.581	1,194	Overflow
311	D1.2-D1.3	1,105	0.486	0.619	Overflow

Source : Calculation results

The infiltration well discharge is calculated using the Sunjoto method to reduce runoff. Infiltration wells are placed in canals that experience flooding. The calculation results conclude that 378 infiltration wells are needed in 34 canals for a flooded area of 34,838 ha.

4.5 Relationship between Area Size, Inundation Area and Flood Free

The initial condition of the existing channel 317, for 90 overflow channels, the presentation of channels that do not overflow is 77.89%, the overflow is 22.11%. The size of the overflow area varies as in table 4.7 below at the initial time

Table 4. 7. Initial Area

Areas	Area (Ha)	Inundation Area (Ha)	Free Area Flood (Ha)
1	27,083	23,169	3,914
2	16,370	15,072	1,298
3	12,010	12,010	0,000
4	12,883	8,707	4,176
	68,346	58,958	9,388

Source : Calculation results

After carrying out identification and analysis of the area, it is necessary to handle surface flow in order to reduce inundation at the research location. From table 4.7, areas 1, 2 and 4 experienced inundation which is quite worrying, the existing channel cannot accommodate the surface flow entering the channel so that almost all The area was inundated with water, only area 3 did not experience inundation. Based on these results, flood management was carried out, namely by changing the initial height H to H=0.45m. After the changes were made, it was seen that there was a change in the flood inundation area, which was initially 58,958 ha, reduced to 23,127 ha, so that the flood free area increased to 35,831 ha, which was originally 9,388 ha, see comparison of table 4.7 and table 4.8.

Table 4.8. Area After Change in H=0.45m

Areas	Area of Flooded Area (Ha)	Inundation Area (Ha)	Free Area Flood (Ha)
1	23,169	13,992	9,177
2	15,072	5,054	10,018
3	12,010	0	12,010
4	8,707	4,081	4,626
	58,958	23,127	35,831

Source : Calculation results

P- ISSN: 2598-1277
E- ISSN : 2621-3966

Based on table 4.8. states that there is still inundation of 23,127 ha, so there needs to be a second treatment, namely by making absorption wells. After the analysis is carried out, it is known that the absorption wells can suppress inundation in Areas 1,2 and 4 as shown in table 4.9, that the remaining area of inundation 0.060 ha and the flood-free area is 23,087 ha.

Table 4.9. Area After Use Well Absorption

Areas	Area of Flooded Area (Ha)	Inundation Area (Ha)	Free Area Flood (Ha)
1	13,992	0.010	13,982
2	5,054	0.020	5,034
3	0	0	12,010
4	4,081	0.010	4,071
	23,127	0.060	23,087

Source : Calculation results

The total flood free area from the initial identification and analysis results is 9,388 ha, then the first flood treatment with H=0.45 results in a flood free area of 35,831 ha for the second treatment the flood free area is 23,087 ha so that the total flood free area is 63,346 ha with the remaining still inundated 0.040 ha

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

Based on the calculated inundation that occurred at the research location for Areas 1, 2, 3 and 4, efforts are made to overcome flooding due to runoff in two ways.

1. Change Dimensions Channel

y making changes to the dimensions of the channels in the field, the height H was increased to 0.45m, but after this was done, there were still 34 channels that were still experiencing flooding, this was caused by differences in elevation which caused flooding in the channels, the changes obtained from the results of raising height H, the change in flood area from 58,958 ha was reduced to 35,831 ha

2. Making Well Absorption

The second step to overcome the problem of inundation that still occurs after raising the height of H, which is 23,127 ha, is carried out by making infiltration wells, the number of infiltration wells is calculated at 378 spread across 34 channels that experience inundation, the results show that the area of inundation has decreased by 23,087 and still remains. The area still inundated is 0.040 ha.

3. The success rate of flood management efforts reached 99,908% failure 0.092% from an area of 68,346 ha with a flood-free area reaching 68,306 ha and a side area of 0.040 ha

5.2 Suggestion

1. Further research needs to be carried out regarding infiltration well trials to determine the effectiveness of infiltration wells in absorbing rainwater
2. It is necessary to carry out regular studies on sustainable urban drainage systems following city developments

BIBLIOGRAPHY

[1] Syaifuddin, A. Hamire, and Dahlan, “Hubungan antara Jumlah Penduduk dengan Alih Fungsi

P- ISSN: 2598-1277
E- ISSN : 2621-3966

Lahan di Kecamatan Somba Opu Kabupaten Gowa," *J. Agrisistem*, vol. 9, no. 2, pp. 169–179, 2013.

[2] A. Tribhuwana and A. Prasetyo, "Analysis of Infiltration Relations With Land Physical Properties," *J. Green Sci. Technol.*, vol. IV, no. 3, pp. 105–112, 2020.

[3] Manggau M A, Alam B Y C S, and Iskandarsyah T Y W M., "Simulasi Pengaruh Perubahan Tataguna Lahan Dan Temperatur Terhadap Potensi Cadangan Airtanah Pada Perhitungan Neraca Air Das Cijulang, Pangandaran," *Padjadjaran Geosci. J.*, vol. 4, no. 4, pp. 322–337, 2020.

[4] A. Tribhuwana, F. Rohman, and O. Farhan, "Analysis of The Carrying Capacity of Urban Drainage Dimensions," *J. Green Sci. Technol.*, vol. V, no. 1, pp. 11–20, 2021.

[5] Muharomah R, "Analisis Run-off Sebagai Dampak Perubahan Lahan Sekitar Pembangunan UnderPass Simpang Patal Palembang Dengan Memanfaatkan Teknik GIS," *Tek. Sipil dan Lingkung.*, vol. 2, pp. 424–433, 2014.

[6] P. J. Ward *et al.*, "The need to integrate flood and drought disaster risk reduction strategies," *Water Secur.*, vol. 11, no. September, 2020, doi: 10.1016/j.wasec.2020.100070.

[7] C. S. Silvia and M. Safriani, "Analisis Penanganan Masalah Banjir Dengan Sumur Resapan," *Civilla J. Tek. Sipil Univ. …*, 2020, [Online]. Available: http://www.jurnalteknik.unisla.ac.id/index.php/CVL/article/view/410

[8] E. C. Edwards, T. Harter, G. E. Fogg, B. Washburn, and ..., "Assessing the effectiveness of drywells as tools for stormwater management and aquifer recharge and their groundwater contamination potential," *Journal of …*. Elsevier, 2016. [Online]. Available: https://www.sciencedirect.com/science/article/pii/S0022169416303365

[9] S. Richard Chikabvumbwa and D. Worku, "Rainfall frequency analysis using Gumbel distribution," *Int. J. Creat. Innov. Res. All Stud.*, vol. 1, no. 1, 2017.

[10] A. Tribhuwana, "Perbandingan Pengukuran Luas Area antara Theodolit dan Global Positioning System (GPS)," *Logika*, vol. 22, no. 3, pp. 58–64, 2018.