Jurnal
Teknologi

Full Paper

HOLDING CAPACITY ANALYSIS OF AN EXISTING DRAINAGE CHANNEL USING THE SOFTWARE HEC-RAS 4.1.0

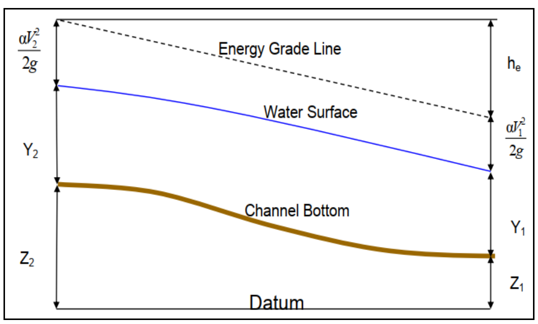
Ahmad Herison[a]*, Yuda Romdania[a], Cristiyanti, Lusmeilia Afriani[a], Arif Mahasin Sondani[b]

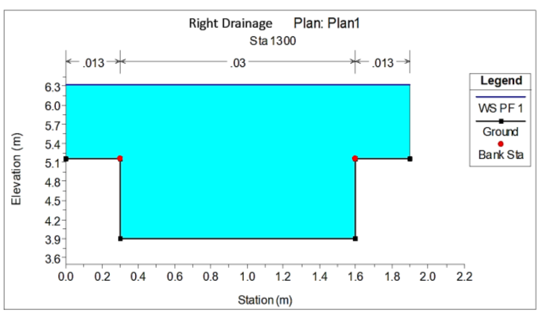
[a]Department of Civil Engineering, Faculty of Engineering, University of Lampung, Lampung, Indonesia
[b]Accounting Major, Polytechnic State of Sriwijaya, Indonesia

Article history
Received
4 September 2023
Received in revised form
18 August 2025
Accepted
28 August 2025
Published Online
16 June 2026

*Corresponding author
ahmad.herison@eng.unila.ac.id

Graphical abstract





Abstract

One of the factors of flood is resident's fast-paced growth rates. More than half of the world population currently live in urban areas. This growth has significant impacts on the hydrological cycle so that it will affect the drainage system a great deal. This research was aimed at analyzing the flood-related issues resulting from an insufficiently optimum holding capacity located on Pulau Sebesi Street, Sukarame Sub-District, Bandar Lampung City, Lampung Province, Indonesia. The methods on this research involved HEC-RAS 4.1.0 for the channel hydrologic analysis and the ministerial regulation PUPR No 1 of 2022 for the budget comparison. Based on the calculation, the water surface height of the right the part of the right existing drainage channel at the riverhead was 2,42 m and that at the river bottom was 2,21 m, whereas the height of the part of the left existing drainage channel at the riverhead was 0,71 m and that at the river bottom was 1 m, so the channels could not hold the flood debit. From the results of the budget comparison of the channel normalization, it is known that the cost variable was equivalent to Y, while the channel dimensional enlargement was equivalent to 333Y. The conclusion is that the drainage cannot hold the planned flood debit in a recurrence interval of 10 years. For this reason, such a solution as dimensional enlargement is of the essence so that no flood will reoccur. Drainage channel normalization is an alternative.

Keywords: Floods, drainage, HEC-RAS, budget, water surface

1.0 INTRODUCTION

One of the natural occurrences that threaten the sustainability of people's lives in some regions in Indonesia every time the rainy season comes is flood. [1], [2]. National Agency for Disaster Countermeasure, BNPB, recorded that during 2019, there had been 3.814 disasters, 784 of which were floods [3]. Flood causes severe damage to the social-spatial environments of the afflicted cities [4], like the breakdowns of the infrastructure and economic activities, and disturbs the social activities [5]. In the mitigation of flood in cities, it is highly important to take notice of the causal factors.

One of the factors of flood is resident's fast-paced growth rates. More than half of the world population currently live in urban areas [4]. This growth has significant impacts on the hydrological cycle [6], so that it will affect the drainage system a great deal.

The drainage channel system basically operates in order to lessen water excess in a city by directing water, which will then be directed towards a river, a pool, or the sea, either via surface channels or buried channels [7,8]. Nowadays, the drainage system has been one of the vital urban infrastructures [1]. The management quality of a city is reflected by the quality of its drainage system. Therefore, the evaluation of the drainage channels is of paramount importance.

Drainage channel evaluation plays a prominent part in the solution to the mitigation of issues in relation to rainfall-runoff [9, 10]. It is an attempt to measure the achievement of a drainage system plan in terms of channeling rainwater or wastewater from a riverhead to the river bottom [11].

Many previous studies may have discussed the analysis and evaluation of drainage channels, but this research employed a hydrologic analysis with a 1D model by the software HEC-RAS and the analysis was done in an area with different topographic characteristics, which makes this research unprecedented. In other words, this research offers a novelty.

This research was aimed at analyzing the flood-related issues resulting from an insufficiently optimum holding capacity located on Pulau Sebesi Street, Sukarame Sub-District, Bandar Lampung City, Lampung Province, Indonesia. This research had the output of the water surface profile of the channel cross section, so the holding capacity of the existing drainage could be figured out.

2.0 METHODOLOGY

2.1 Research Locations

The research was conducted in Pulau Sebesi Street, Sukarame District, Bandar Lampung City. Referring to the district administrative map, the width of the region is 493 Ha, which is used for places of residence and farms. The distance between the research location and the centre of Bandar Lampung City was ± 2 km only (see Figure 1).

2.2 Data Colection

Rainfall duration is the period between the start of The types of data of this research were:

1. Primary Data

The primary data was the condition and dimension of the existing drainage channels, which had been found out through direct field observation and measurement.

2. Secondary Data

The secondary data was the daily rainfalls in the past ten years, from 2013 to 2022, which had been known from Rainfall Station 003 of Sukarame, whose records had been obtained from Balai Besar Wilayah Sungai Mesuji Sekampung (BBWS Mesuji Sekampung), and the number of residents, obtained from the website of the Central Bureau of Statistics, BPS, of Bandar Lampung City.

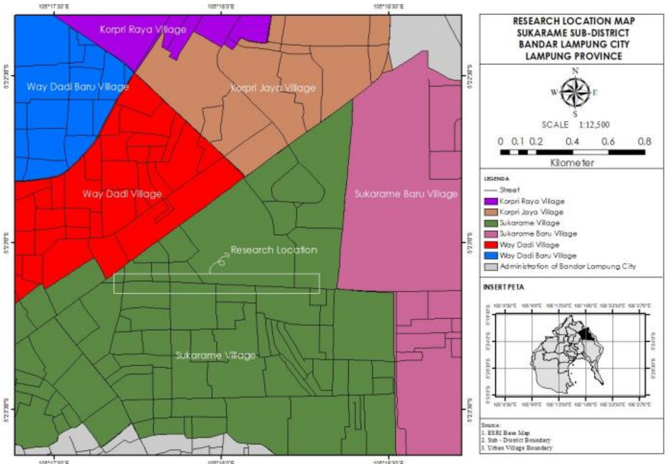


Figure 1 Research Locations

2.3 Data Analysis

Hydrologic Analyses

1. Frequency and Probability Analyses

The frequency analysis is an analysis for finding out the relation between the intensity of an extreme occurrence and the frequency probability of the occurrence [12].

A frequency analysis is based on the statistical nature of the available data for obtaining the probability of a rainfall intensity in the future. There are four types of distribution for analyses of flood probability in general, namely [13]:

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

The probability distribution closest to the data can be identified and used for frequency analysis [12] (see Table 1.

Table 1 Statistical Parameter Requirements

Type of Distribution	Requirement
Normal	$Cs = 0$ $Ck = 3$
Log Normal	$Cs = 3Cv + Cv^3$ $Ck = Cv^8 + 6Cv^6 + 15Cv^4 + 16Cv^2 + 3$
Gumbel	$Cs = 1{,}14$ $Ck = 5{,}4$
Log Person III	Other than the values above

Source: [12].

2. Rainfall Time of Concentration and Intensity

Time of concentration is the time raindrops need to flow from the farthest point to the control point subsequent to soil saturation [14]. The following is the formula of time concentration calculation:

$$tc = \left[\frac{0{,}87 x L^2}{1000 \, x \, s}\right]^{0{,}385} \quad \text{.......} \quad (1)$$

where:
tc = time of concentration (hours)
L = channel lengths (km)
s = skewness of channel area.

The nature of rainfall is that the shorter the period, the higher the intensity tends to be and the higher the recurrence interval, the higher the intensity. Rainfall intensity is obtained through rainfall data analysis either statistically or empirically [15].

Rainfall intensity can be calculated with the Mononobe formula [14]:

$$I = \frac{R_{24}}{24}\left(\frac{24}{tc}\right)^{2/3} \quad \text{.......} \quad (2)$$

where:
I = rainfall intensity (mm/jam)
R24 = 24-hour maximum rainfall (mm)
tc = time of concentration (hours).

3. Catchment Area

The catchment area is a rain garden where the water flowing on its surface is contained by the designated channel [16]. In order that the water flows optimally and effectively, it is necessary to determine the catchment area so that the channeling system is suitable for the previously determined catchment area.

4. Planned Flood Debit

The planned flood debit is the maximum flood debit or the maximum flood debit of a river or channel whose size is determined based on a certain recurrence interval. A planned flood debit is calculated by adding up the rainwater debit to the wastewater debit [17]. The following are the formulae of the calculation of rainwater and wastewater debits:

- Rainwater Debit

A rainfall or runoff debit is the water volume at a certain unit of time which does not experience infiltration, so it should be channeled through drainage [18]. The following is the equation of the rational method [12]:

Qah = 0,278.C.I.A .... (3)

where:
0,278 = conversion factor of planned flood debit to m3/second unit)
Qah = rainfall debit (m^3/second)
C = surface flow coefficient
I = rainfall intensity (mm/hour)
A = area width (km^2).

- Wastewater debit

For the estimation of wastewater to be channeled to a drainage channel, it is necessary to first figure out the average need for water and the population of the planning area [19].

It is estimated that the amount of wastewater getting into a collecting channel is 70% of the needed clean water. A debit of the wastewater discharged every km^2 can be calculated with the following equation [20]:

$$Q = \frac{(Pn \times Qkeb \times 70\%)}{A} \quad \text{.......} \quad (4)$$

where:
Q = wastewater debit (lt/sec/km^2)
Pn = population in the n-year
Qkeb = need for clean water (lt/day/individual)
A = area width (km^2).

As for the wastewater debit of each channel, it can be calculated by multiplying the debit of the water discharged every km^2 by width of the channel area.

Qak = Q x width of channel area...................... (5)

Hydrologic Analysis (HEC-RAS 4.1.0 Model)

The hydrologic calculation of a flow is basically intended to find out the depth and velocity of the flow along a channel caused by the flow going into the channel and the depth of the flow at the downstream limit.

The following are the equations involved by HEC-RAS [21].

1. Energy Equation

The profile of a water surface ranges from one cross section to another and is calculated by finishing the energy equation through an iterative procedure known as Standard Step Method. The representations of the terms in the energy equation can be seen in Figure 2.

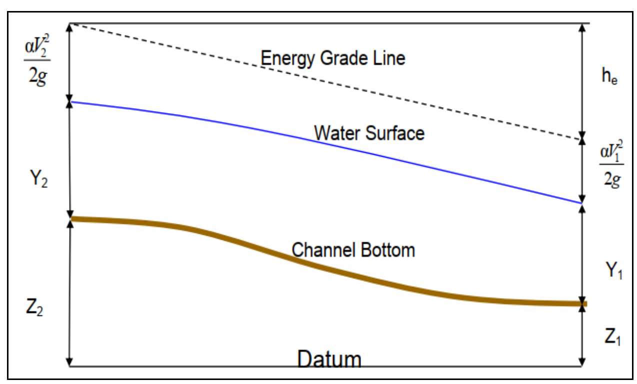


Figure 2 Representations of Energy Equation
Source: [21]

The energy equation is written as follows:

$$Z_2 + Y_2 + \frac{a_2 V^2_2}{2g} = Z_1 + Y_1 + \frac{a_1 V^2_1}{2g} + h_e \quad \text{.......} \quad (6)$$

where:
Z = channel basic elevations at cross sections 1 and 2 (m)
Y = water depths at cross sections 1 and 2 (m)
V = average velocities at cross sections 1 and 2 (m/sec)
α = energy coefficients at cross sections 1 and 2
g = gravitational acceleration (m/sec^2)
he = energy loss (m).

2. Energy Loss

The energy loss (he) between two cross sections is a friction loss or a contraction or expansion loss. The energy loss between cross section 2 and 1 is stated in the equation below:

$$h_e = L.SF + C\left|\frac{a_2V^2{}_2}{2g} - \frac{a_1V^2{}_1}{2g}\right| \quad (7)$$

where:
L = length of part of river between two cross sections given weight according to debit
SF = skewness of friction between two cross sections
C = coefficient of expansion or contraction loss

3. Holding Capacity

The calculation of the capacity and the average velocity at a cross section is done by dividing the cross section into several segments, where the velocity is distributed evenly at each segment.

The amount of the debit per cross section is calculated through the following Manning's equation:

$$Q = K.SF^{2/3} \quad (8)$$

$$K = \frac{1}{n}A.R^{2/3} \quad (9)$$

4. Contraction-Expansion Coefficient

The coefficient of contraction and expansion energy loss is calculate in the following way:

$$h_e = C\left|\frac{a_2V^2{}_2}{2g} - \frac{a_1V^2{}_1}{2g}\right| \quad (10)$$

Comparison of Budget Plans

The budget plan comparison refers to the regulation of the Ministry of Public Works and Housing of Indonesia Number 1 of 2022 on the Guidelines of the Cost Estimation of Construction Work in the Field of Public Works and Housing (the analysis of the unit prices of water resources work). As for the prices, this research referred to the material prices and the regional wage of Bandar Lampung City.

2.4 Frequency Analysis

Frequency analysis is a method for hydrologic data The flowchart (see Figure 3) consists of the several phases involved in this research. The first phase was the collection of the primary and secondary data. The second phase was the analysis of the planned flood debit of the rainwater and wastewater debits. The third phase was inputting the data into the HEC-RAS program so that the presentation of the simulation results was obtained. The fourth phase was providing the solution to the output of the previously obtained simulation results. The last phase was the analysis of the execution methods and budget plan.

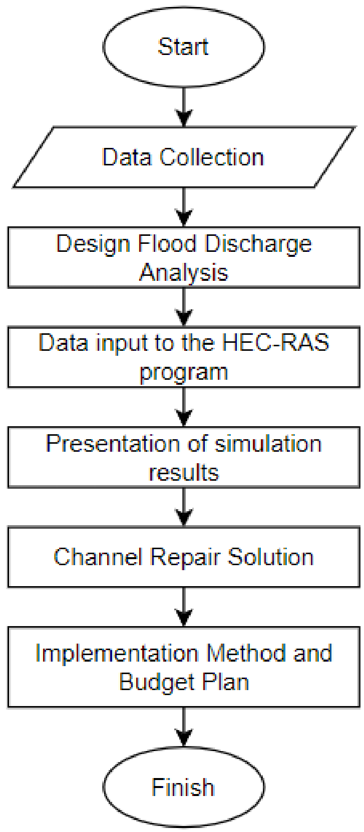


Figure 3 Research Flowchart

3.0 RESULTS AND DISCUSSION

3.1 Hydrologic Analyses

1. Frequency and Probability Analyses

The daily rainfall data was analyzed to obtain the annual maximum rainfall data, which was then referred to in the frequency distribution analysis. The frequency distribution analysis was done by sorting the whole data from the smallest to the largest (see Table 2).

Table 2 Annual Maximum Rainfall Data

No	Year	Maximum Rainfall (mm)
1	2015	52,3
2	2016	62
3	2014	66
4	2022	70
5	2017	78
6	2018	105
7	2013	107,2
8	2021	135
9	2020	160
10	2019	215

The analysis was carried out with four types of distribution, i.e. Normal, Log Normal, Gumbel and Log Pearson III. Each type of distribution was matched with its statistical parameters. The calculation results are presented in Table 3.

From Table 3, it is known that the type suitable for the planned rainfall was Log Pearson III, which met the requirement.

The maximum planned rainfall was calculated in the way below:

$\text{Log } X_T$ = Log X + (K_T x Sd) .... (11)
= 1,9786 + (1,3230 x 0,1990)
= 2,2419

X_T = $10^{2,2419}$
= 174,5414 mm

Table 3 Results of Statistical Parameters

Type of Distribution	Requirement	Calculation		Conclusion
Normal	Cs = 0	Cs	1,1735	Failed
	Ck = 3	Ck	4,7364	Failed
Log Normal	Cs Cv³ + 3 Cv = 1,5386	Cs	0,5001	Failed
	Ck = Cv⁸ + 6Cv⁶ + 15Cv⁴ + 16 Cv²+3 = 7,8824	Ck	3,2370	Failed
Gumbel	Cs = 1,14	Cs	1,1735	Failed
	Ck = 5,4	Ck	4,7364	Failed
Log Pearson III	Other than the values above (Cs ≠ 0)	Cs	0,5001	**Passed**

2. Time of Concentration and Rainfall Intensity

The calculation of time of concentration (tc) employed equation 1 and the intensity of the planned rainfall was calculated with the Mononobe formula in equation 2.

L : 1,3 km (lengths of reviewed channels)
s : 0,003

So, a time of concentration of 0,7599 hour and a rainfall intensity of 72,6644 mm/hour were obtained.

3. Catchment Area

The catchment area was determined by referring to the topographic map (see Figure 4). The catchment area obtained from the calculation of the right channel was 0,4742 km² and the catchment area of the left channel was 0,0431 km².

4. Planned Flood Debit

The planned flood debit was calculated with a recurrence interval of 10 years by adding up the rainwater debit to the wastewater debit. The calculation of the rainwater debit used equation 3 and that of the wastewater debit used equations 4 and 5.

The following is the data obtained from the Central Bureau of Statistics of Sukarame Sub-District:

Pn : 15.698 individuals
Q_{keb} : 144 lt/day/individual (0,001668 lt/sec/individual)
A : 4,93 km²
(The calculation results are in Table 4)

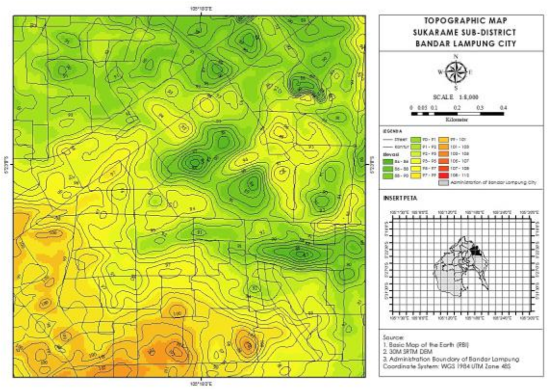


Figure 4 Topographic Map

Table 4 Results of Calculation of Planned Flood Debit with Recurrence Interval of 10 Years

Drainage Channel	Rainwater Debit (m³/sec)	Wastewater Debit (m³/sec)	Planned Flood Debit (m³/sec)
Right Channel	6,2814	0,00165	6,2830
Left Channel	0,6095	0,00016	0,6097

3.2 Hydrologic Analyses (HEC-RAS 4.1.0)

1. Existing Channel Geometry

The existing channel geometry was carried out by making a channel schema and inputting the dimensions of the cross sections. The schema was made by clicking the menu bar of the geometric data + river reach. Then, the channel flow schema was made on the editor screen of the geometry. Since the channels were straight, two dots at both ends of each channel were enough for the schema. The flow was made from the riverhead to the river bottom, not the other way around (see Figure 5).

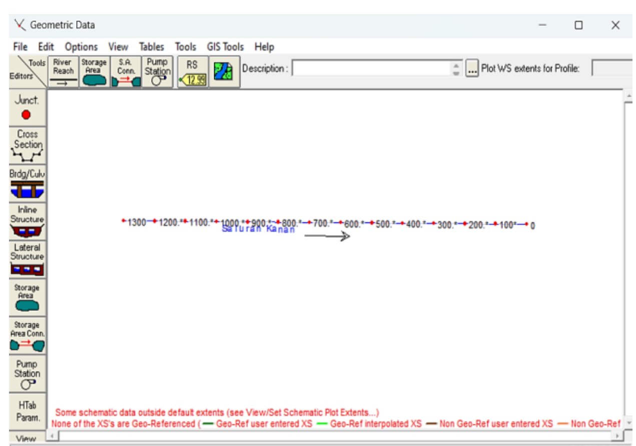


Figure 5 Schema of Existing Drainage Channels

Tables 5 and 6 present the input data obtained from the measurement of the channels (see Table 5 and Table 6). Figures 6 and 7 present the dimensions input with HEC-RAS for Sta 0 (riverhead) (see Figure 6 and Figure 7).

2. Inputting Debit Values

To input the debit value of a steady flow, click menu bar Edit + Steady Flow Data. The input debit values Q_{10} of the right and left drainage channels were 6,2830 m^3/second and 0,6097 m^3/second respectively (see Figure 8 and Figure 9).

Table 5 Results of Right Existing Drainage Channel

No	Channel Part	Size	
		Width (m)	**Height (m)**
1	Sta 0 - 750	1,42	1,1
2	Sta +750 - 1300	1,3	1,25

Table 6 Results of Left Existing Drainage Channel

No	Channel Part	Size	
		Width (m)	**Height (m)**
1	Sta 0 - 750	0,92	0,84
2	Sta +750 - 1300	1,04	0,7

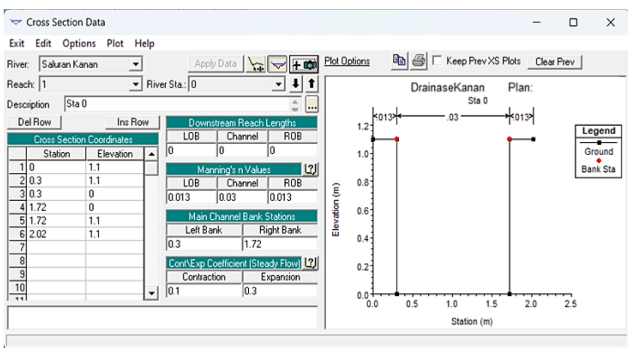


Figure 6 Editor Screen Displaying River Cross Section at Sta 0 (Right-hand Side)

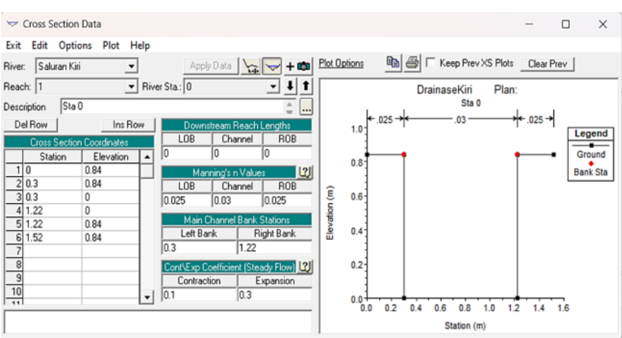


Figure 7 Editor Screen Displaying River Cross Section at Sta 0 (Left-hand Side)

3. Inputting Skewness Value

The skewness value was input in accordance with the skewness in the field, in the column "Downstream", i.e. 0,003. To do this, just click the icon "Reach Boundary Conditions" and choose "Normal Depth" and click "OK". This inputting stage was performed in the same way for the right channel and the left channel (see Figure 10).

4. Running Steady Flow Analysis

After the data was saved, it was analyzed with HEC-RAS by clicking Run + Steady Flow Analysis + Compute. The display in Figure 11 appeared when the analysis had been performed.

5. Presentation of Simulation Results

The presentation of the HEC-RAS 4.1.0 simulation results of one cross section was carried out by clicking View + Cross Section. "River Sta" in "Cross Section" was clicked to see the profile of the water surface. Figure 12 and Figure 13 present the profile of the water surface of the right channel (see Figure 12 and Figure 13), while Figure 14 and Figure 15 present the water surface of the left channel.

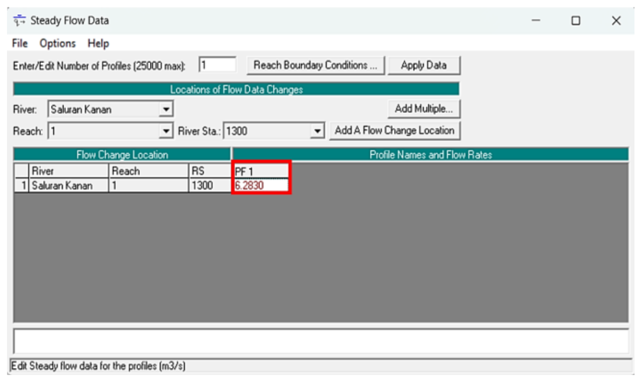


Figure 8 Input Values of Right-hand Side

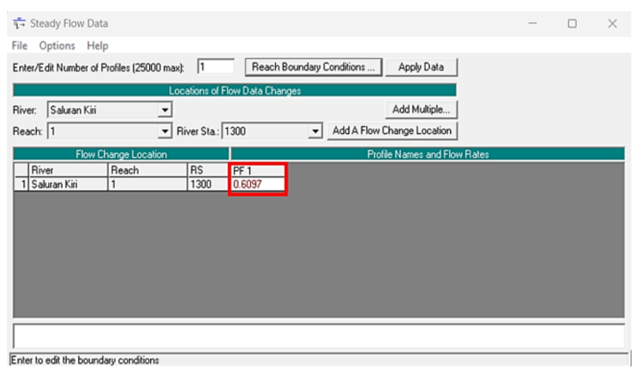


Figure 9 Input Values of Left-hand Side

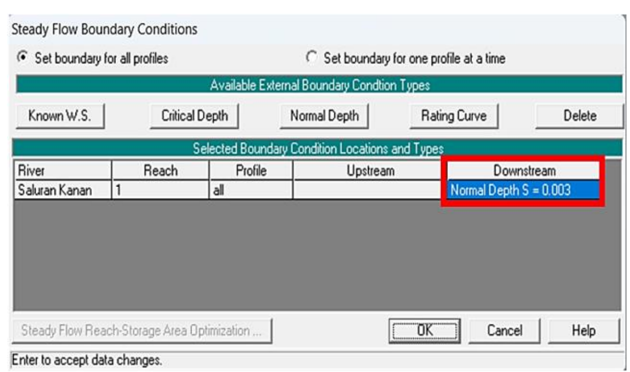


Figure 10 Input Skewness Value

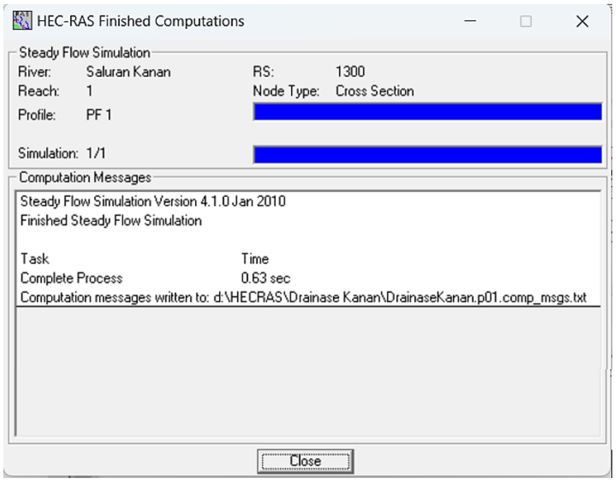


Figure 11 Steady Flow Analysis

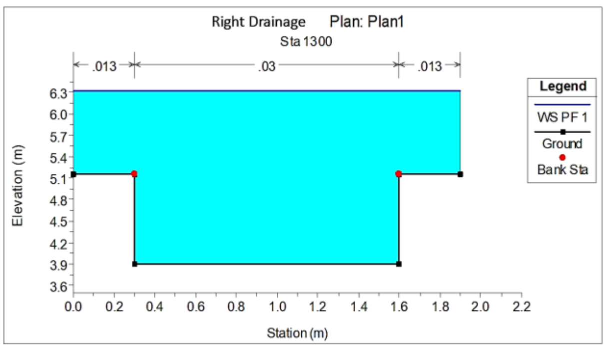


Figure 12 Profile of Riverhead Water Surface (Right-hand Side)

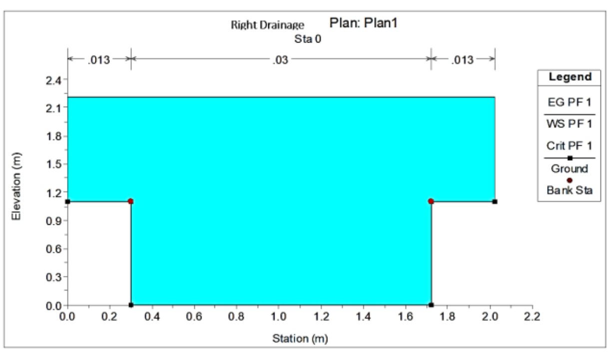


Figure 13 Profile of River-bottom Water Surface (Right-hand Side)

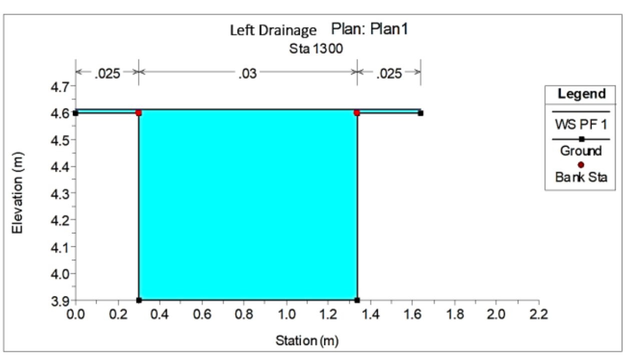


Figure 14 Profile of Riverhead Water Surface (Left-hand Side)

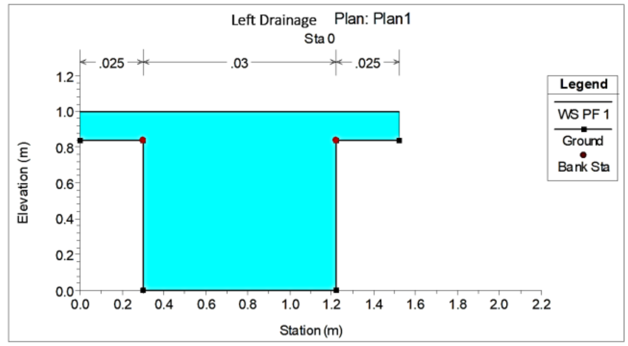


Figure 15 Profile of River-bottom Water Surface (Left-hand Side)

Referring to the simulation results, the right and left existing channels could not hold the debit in a recurrence interval of 10 years. The heights of the water surfaces in the simulation exceeded the height limits of the right and left channels (Bank Sta). The heights of the water surfaces are presented in Table 7.

Table 7 Results of Simulation of Existing Channel Water Surfaces

Channel	***Water Surface* (m)**	
	Sta 1300 (Riverhead)	**Sta 0 (River Bottom)**
Right Channel	2,42	2,21
Left Channel	0,71	1

3.3 Dimensional Enlargement Solution

In reference to the water surfaces in the simulation results in Table 7, one of the possible solutions was the dimensional enlargement of the drainage channels through the change in the heights and widths. The steps of changing the dimensions were just the same as those of the existing drainage channel modelling, but the dimensions had been enlarged through trial and error until they could hold the calculated debit. Table 8 presents the enlarged channel dimensions.

Table 8 Dimensions of Planned Drainage Channels

No	Channel	Size	
		Width (m)	**Height (m)**
1	Right Channel	1,5	1,8
2	Left Channel	1,1	1,1

Figure 16 and Figure 17 show the results of the water surface profile of the planned right channel, while Figure 18 and Figure 19 show the results of the water surface profile of the planned left channel.

Based on the simulation results, the planned left and right channels could hold the debit in a recurrence interval of 10 years with the dimensional enlargement. The water surface heights in the simulation did not exceed the heights of both channels (Bank Sta). Table 9 shows the heights of the water surfaces.

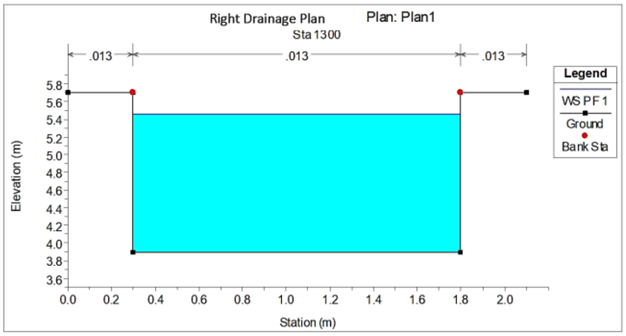


Figure 16 Water Surface Profile at Riverhead (Planned Right Channel)

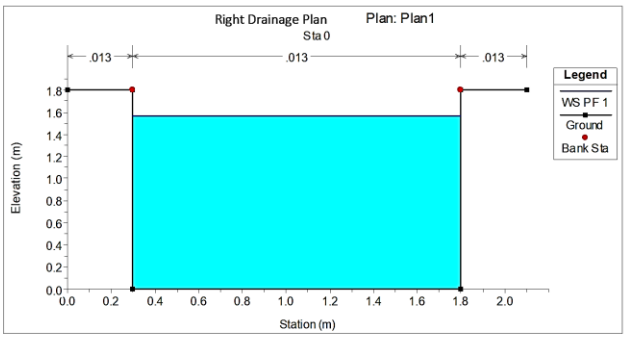


Figure 17 Water Surface Profile at River Bottom (Planned Right Channel)

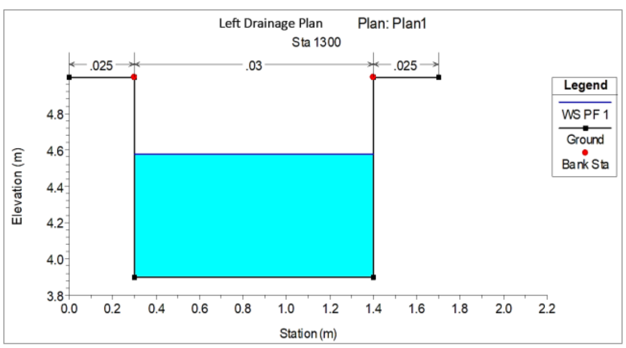


Figure 18 Water Surface Profile at Riverhead (Planned Left Channel)

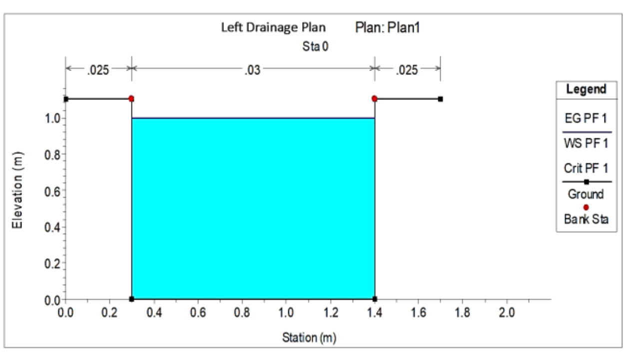


Figure 19 Water Surface Profile at River Bottom (Planned Left Channel)

Table 9 Results of Simulation of Planned Channel Water Surfaces

Channel	*Water Surface* (m)	
	Sta 1300 (Riverhead)	Sta 0 (River Bottom)
Right Channel	1,56	1,56
Left Channel	0,67	0,67

The simulation performed with the software HEC-RAS did not take into account issues relating to the sediment and rubbish in the drainage channels. The measurement of the heights of the existing channels was carried out above the sediment, not at the bottoms of the channels. It is essential to pay special attention to the sediment and rubbish in the channels since they affect the capacities of the drainage channels a great deal.

3.4 Methods of Work Execution

Taking the thickness of the sediment in the channels, the solution should still be executable by neutralizing the channels from the sediment and rubbish so that the heights added to the channels through the enlargement will not change. It will ease the execution as digging too much in the channels will not be necessary.

As for this research, the sediment and rubbish in the channels were not taken into account, so the heights of the channels resulting from the dimensional enlargement caused the channels to be deeper. The following are the stages of the renovation of the drainage channels:

1. The first step was manually digging in the existing drainage channels with certain equipment until reaching the depths and elevations to be enlarged. In order that the holes suited the planned dimensions, they were measured with a certain measuring device.
2. The earth resulting from the digging was carried out of the research location by dump truck so that it would not pile up on the project site and cause any disturbance to the process of the work execution.
3. The next step was the fixing of riverstone. Before it was put on the first layer, the hole in the ground was filled and the ground was then flattened and cemented.
4. The riverstone was fixed on the right and left drainage channels and was glued with mortar until no gap between stones was found.
5. After that, it was plastered, but before starting the work, the surfaces that would be plastered were checked to make sure that they were dirt-free. This was done by evenly applying cement on the walls and grounds of the channels. The use of the plaster was to form strong bonds of the riverstone in order that it was firm enough.
6. The sixth step was coating. Before getting started on this work, the provider had to get the permission from the directors to decide which parts would be coated.

3.5 Comparison of Planned Budgets

The planned budgets were compared according to the offered solutions to the drainage channel issue, i.e.:

1. The cost is lower if it is only cleaning the sediment since only the operational cost needs to be taken into account, which is a variable cost of Y.
2. If the construction with the enlargement of the width and height is carried out, the variable cost is 333Y

4.0 CONCLUSION

The conclusion is that the drainage channels cannot hold the flood debit in a recurrence interval of 10 years, so such a solution as dimensional enlargement is prominent as it can prevent flood from occurring. Normalization of the drainage channels is an alternative. From the comparison of the two solutions, it is known that the cost coefficient of channel normalization is lower than the cost coefficient of dimensional enlargement.

Acknowledgement

Our deepest gratitude goes to Balai Besar Wilayah Sungai Mesuji Sekampung (BBWS-MS).

Conflicts of Interest

The authors declare that there is no conflict of interest regarding the publication of this paper.

References

[1] Rahman, M. H. 2022. A Study on Determining Land Use/Land Cover Changes in Dhaka over the Last 20 Years and Observing the Impact of Population Growth on Land Use/Land Cover Using Remote Sensing. *Malaysian Journal of Civil Engineering*. 34(2): 1–9. https://doi.org/10.11113/mjce.v34.17812.

[2] Wigati, R., T. Soedarsono, and T. Mutia. 2016. Analisis Banjir Menggunakan Software HEC-RAS 4.1.0 (Studi Kasus Sub DAS Ciberang HM 0+00–HM 34+00). *Fondasi: Jurnal Teknik Sipil*. 5(2). http://dx.doi.org/10.36055/jft.v5i2.1261.

[3] Sulaksana, N., P. P. R. Rendra, and M. Sulastri. 2021. Sosialisasi Mitigasi Bencana Longsor dan Banjir Secara Virtual di Masa Pandemi COVID-19. *Kumawula: Jurnal Pengabdian Kepada Masyarakat*. 4(3). https://doi.org/10.24198/kumawula.v4i3.35516.

[4] Badan Nasional Penanggulangan Bencana (BNPB). 2020. Infografis Update Bencana Tgl. 13 Maret 2020 Pk. 10.00 WIB. *Aspirasi*. *11*(1).

[5] Chandraseana, D. C. N., K. W. Yusof, M. R. U. Mustafa, and S. I. Umar. 2015. A Theoretical Discussion and Case Study on the Development of Smart Storm Drainage Unit for Compact Cities. *Jurnal Teknologi (Sciences & Engineering)*. 78(5–3): 85–90. https://doi.org/10.11113/jt.v78.8517.

[6] Svm, B. 2019. Prediksi Kejadian Banjir dengan Ensemble Machine Learning Menggunakan BP-NN dan SVM. *Jurnal Teknik Sistem dan Komputer*. 7(3): 93–97. https://doi.org/10.14710/jtsiskom.7.3.2019.93-97.

[7] Efrizal, E., Y. A. Saputro, and N. Hidayati. 2022. Implementasi Software HEC-RAS 4.1.0 dan EPA SWMM 5.1.0 pada Efektivitas Analisis Saluran Drainase (Studi Kasus Desa Kelet, Kecamatan Keling, Kabupaten Jepara). *Jurnal Civil Engineering Study*. 2(1): 7–16. https://doi.org/10.34001/ces.02012022.2.

[8] Riswanto, S., A. Yunianta, and A. Huddiankuwera. 2023. Analisis Sistem Saluran Drainase di Kawasan Perumahan BTN Bawah Kamkey Abepura dengan Menggunakan Program HEC-RAS 4.1.0. *Civil Engineering and Planning Journal*. 1(1): 42–53. https://doi.org/10.55098/.v1i1.336.

[9] Arifin, D. 2019. Studi Analisa Kapasitas Drainase terhadap Banjir di Jalan Anggana Kota Samarinda. *Kurva Mahasiswa*. 1(1): 43–55.

[10] Wicaksono, B., P. T. Juwono, and D. Sisinggih. 2018. Analisa Kinerja Sistem Drainase terhadap Penanggulangan Banjir dan Genangan Berbasis Konservasi Air di Kecamatan Bojonegoro Kabupaten Bojonegoro. *Jurnal Teknik Pengairan: Journal of Water Resources Engineering*. 9(2): 70–81.

[11] Astika, M. N., and O. H. Cahyonugroho. 2020. Evaluasi Sistem Drainase di Wilayah Kecamatan Waru, Kabupaten Sidoarjo dengan Software HEC-RAS. *EnviroUS*. 1(1): 55–64. https://doi.org/10.33005/envirous.v1i1.19.

[12] Triatmodjo, B. 2008. *Hidrologi Terapan*. Yogyakarta: Beta Offset.

[13] Harto, S. 1993. *Analisis Hidrologi*. Jakarta: Gramedia.

[14] Suripin. 2004. *Sistem Drainase Perkotaan yang Berkelanjutan*. Yogyakarta: Andi Offset.

[15] Yulius, E. 2014. Analisa Curah Hujan dalam Membuat Kurva Intensity Duration Frequency (IDF) pada DAS Bekasi. *Bentang: Jurnal Teoritis dan Terapan Bidang Rekayasa Sipil*. 2(1): 1–8.

[16] Niglar, N., M. A. Samaila, and E. Tuwanakotta. 2018. Evaluasi Existing Saluran Drainase Jalan Kilang Kelurahan Sawagumu Kota Sorong. *Jurnal Karkasa*. 4(2): 84–92. https://doi.org/10.32531/jkar.v4i2.239.

[17] Suadnya, D. P., J. S. F. Sumarauw, and T. Mananoma. 2017. Analisis Debit Banjir dan Tinggi Muka Air. *Jurnal Sipil Statik*. 5(3): 143–150.

[18] Harahap, M. A., and D. S. Harahap. 2021. Evaluasi Saluran Drainase pada Jalan Tangguk Kecamatan Medan Denai. *Jurnal Teknik Sipil*. 16(2): 94–102.

[19] Wicaksono, D. H., R. Anwar, and Suroso. 2014. *Evaluasi dan Perencanaan Ulang Saluran Drainase pada Kawasan Perumahan Sawojajar Kecamatan Kedungkandang Kota Malang*. Doctoral diss., Universitas Brawijaya.

[20] Suhardjono. 1984. *Drainase*. Malang: Universitas Brawijaya.

[21] US Army Corps of Engineers. 2010. *HEC-RAS River Analysis System: User's Manual Version 4.1*. Davis, CA: Hydrologic Engineering Center.