# A Channel Drainage Slope Modified as Discharge Supply Chain in the Coastal Area

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***Abstract*—The slope in the drainage channel system is a primary factor that determines the quantity of water to be gravity discharged. By having a bigger slope, the flow velocity becomes larger and the capacity of water discharged becomes more so that there will be no surface runoff. Lhokseumawe is a city located in the coastal area with its height is only 0.95 m above sea level. It is difficult to build the appropriate slope of the channel. In general, the slope of the channel is made as a whole along the channel to get the required flow velocity, but in flat terrain areas such as in coastal areas, the slope of the required channel is difficult. For this reason, novelties as innovation are needed by modifying the slope of the channel in stages and tiered at each distance of 50 m, by which for channels along the 1,350 m will have 27 stages. The discharge capacity required to flow the rainfall is based on a 2-year return period of 5.10 m3/ sec. Slopes under normal conditions are 0.0013 to get a flow velocity of 1.27 m/sec. Using a section area of 1.32 m2 channel it can get a discharge capacity of 1.67 m3/ sec. Furthermore, the slope of the channel was modified to 0.005 at each distance of 50 m and this obtained a flow velocity of 2.46 m/sec so that it was able to drain a discharge capacity of 3.24 m3/ sec. By modifying this channel slope can increase the discharge by 1.57 m3/sec or 93.7% compared to the slope without modified. It has not been able to multiply the overall discharge but at least can reduce the discharge 93.7%.***

***Keywords*—** *Modification slope, velocity, discharge, coastal areas*

## 1. Introduction

The coastal area is an area that is close to the sea and very low to the sea level, while the disposal of drainage water is to the sea. Coastal land slope tends to be flat with a slope between 0°-30°. In the drainage system, the slope of the land is needed for a greater discharge; this is related to the discharge to flow. For an adequate slope of the channel, there must be a difference in altitude. The sloping the surface of the ground, the less likely it is to get the expected flow velocity. The slope-area relations in which slopes decay slower than roughly the cube root of the drainage area are insensitive processes that have shaped the landscape [1].

In coastal areas land conditions tend to be very flat with a very small difference in height so that it is difficult to get the slope needed to get the flow velocity needed to discharge, there is a tendency to cause inundation during the rainy season. Therefore, it is necessary to modify the slope of the channel as a solution flow velocity.

The flow direction is the most basic hydrological related parameter and forms the basis for all other parameters. The physics of pure flow driven by gravity states that water will always take the steepest descending path, so the flow line crosses the contour line at right angles. In a horizontal area with a small slope such as in the Coastal area, this cannot happen. In sloping terrain, such ambiguous flow direction is always a sub-grid effect that cannot be represented at the current resolution [2].

Besides, precipitation can vary significantly by geographic location based on elevation and slope aspect so that the resulting discharge will also vary, it needs an adequate slope to drain it [3]

In future development, construction material is also needed. Construction materials are an important part of building coastal drainage. Construction materials play a significant role in the construction industry which therefore requires full attention when creating a project plan.[4]

The city of Lhokseumawe in Aceh Province is an area that is in the coastal area, the slope of the land surface tends to be flat with a slope of 0.0013 and very close to the sea level which is 0.95 m above sea level so that floods often occur. Its city has an area of 181.10 $km^2$ covering 4 sub-districts; they are sub-district Muarasatu, sub-district Muaradua, sub-district Banda Sakti and sub-district Blang Mangat. The Lhokseumawe city is like an island separated by canals; called a Muaradua channel as shown in Figure 1.

***International Journal of Supply Chain Management***
IJSCM, ISSN: 2050-7399 (Online), 2051-3771 (Print)


**Figure 1.** Map of Lhokseumawe city

The city of Lhokseumawe has 4 drainage systems, namely system 1 the Panggoi area empties into a Muaradua channel, system 2 the Kandang and surrounding areas, system 3 to serve urban areas which empties into artificial reservoirs namely pusong reservoirs and system 4 which serves Banda Masen village, Hagu Teungoh village, Uteun Bayi village and Teumpok Teungoh village which empties into the Uteun Bayi pond. This research was carried out in drainage system area 4 on channels from Sp Besitua to BM Point 0.00 as shown in Figure 2

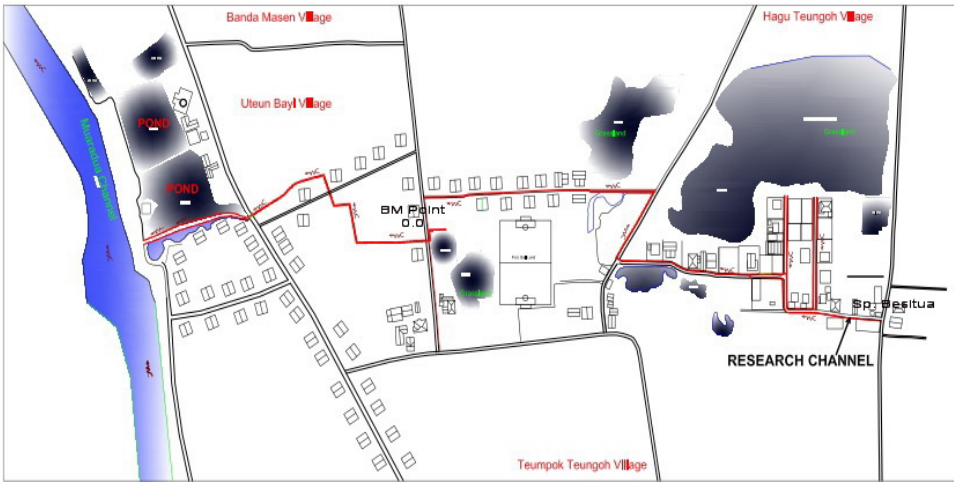


**Figure 2.** Research Area Layouts

The novelties as innovation are needed by modifying the slope of the channel in stages and tiered at each distance of 50 m, by which for channels along the 1,350 m will have 27 stages. Modification of the channel slope is done by making an adequate slope at each distance of 50 m in a multilevel and tiered manner, at 50 m the channel surface height is increased by 50% from the height difference between the base of the channel in the first part to the bottom of the second channel, as shown in Figure 3

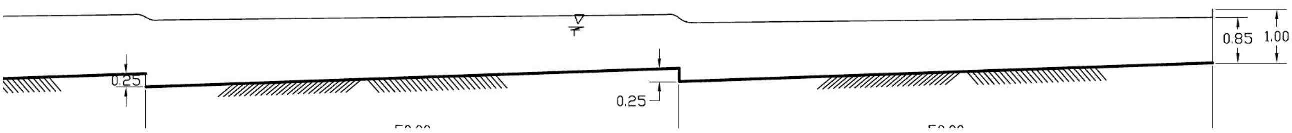


**Figure 3.** Slope Modification

This study aims firstly to know the flow velocity difference between the normal slope and the modified slope, secondly to know how much the discharge increases compared to the original slope before it is modified then finally to know whether the slope modification method can be used in areas with flat slopes such as coastal areas.

## 2. The Methods

The stages of the research were carried out by hydrological analysis based on 10-year rainfall series data to obtain the flow to be discharged. To ensure that the distribution of data is scattered in a normal distribution, a data match test with the Smirnov-Kolmogorov distribution method. The Smirnov Kolmogorov method compares the $D_0$ values obtained based on the table of critical values (Table 5) Smirnov-Kolmogorov Test with D values obtained from calculations. If the value of D is smaller than Do, the distribution used to determine the planned discharge is acceptable, whereas if the value of D is greater than Do, the distribution used to determine the planned debit is not acceptable [5]

Hydraulics analysis is carried out for the channel cross-section design and longitudinal planned by modifying the slope of the channel slope because the slope at the study site tends to be flat, this is done to get the adequate slope to increase the flow velocity as shown in Figure 3.

### 2.1 Hydrological Analysis

The hydrological analysis aims to determine the discharge carried out through analysis of planned rain, rainfall intensity, and discharge. The planned rain is the maximum daily rainfall that will be used to calculate the rain intensity, and then the intensity of this rain is used to estimate the discharge plan. Rain plans can be calculated statistically based on previous rainfall data [6].

**Table 1.** Reduced Mean ($Y_n$)

| n | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
|---|---|---|---|---|---|---|---|---|---|---|
| 10 | 0.4952 | 0.4996 | 0.5035 | 0.5070 | 0.5100 | 0.5128 | 0.5157 | 0.5181 | 0.5202 | 0.5220 |
| 20 | 0.5236 | 0.5252 | 0.5268 | 0.5283 | 0.5296 | 0.5300 | 0.5820 | 0.5882 | 0.5343 | 0.5353 |
| 30 | 0.5362 | 0.5371 | 0.5380 | 0.5388 | 0.5396 | 0.5400 | 0.5410 | 0.5418 | 0.5424 | 0.5430 |
| 40 | 0.5436 | 0.5442 | 0.5448 | 0.5453 | 0.5458 | 0.5468 | 0.5468 | 0.5473 | 0.5477 | 0.5481 |
| 50 | 0.5485 | 0.5489 | 0.5493 | 0.5497 | 0.5501 | 0.5504 | 0.5508 | 0.5511 | 0.5515 | 0.5518 |
| 60 | 0.5521 | 0.5524 | 0.5527 | 0.5530 | 0.5533 | 0.5535 | 0.5538 | 0.5540 | 0.5543 | 0.5545 |
| 70 | 0.5548 | 0.5550 | 0.5552 | 0.5555 | 0.5557 | 0.5559 | 0.5561 | 0.5563 | 0.5565 | 0.5567 |
| 80 | 0.5569 | 0.5570 | 0.5572 | 0.5574 | 0.5576 | 0.5578 | 0.5580 | 0.5581 | 0.5583 | 0.5585 |
| 90 | 0.5586 | 0.5587 | 0.5589 | 0.5591 | 0.5592 | 0.5593 | 0.5595 | 0.5596 | 0.5598 | 0.5599 |
| 100 | 0.5600 | | | | | | | | | |

The return period is the average period expected to occur between two consecutive events. The period of return (T) is usually predicted from annual serial rainfall data. The annual maximum series, which is equivalent to the following formula for the return period, was adopted as a standard placement method by the US Water Resources Council [7] with the equation:

$$Tr = \frac{n+1}{m} \qquad (1)$$

Furthermore, the Return Period is calculated using the equation:

$$Y_{TR} = -(0.834 + 2.303 LogLog \frac{T}{T-1}) \qquad (2)$$

Déviation standard:

Where "$Y_{TR}$" is reduced variate; "T" is returned period; "SD" is deviation standard; "$R_{24}$" is a rain of return period; "Rr" is rainy plan; "K" is the frequency factor for the return period of the year is in accordance with the type of rainfall data distribution.

Where "Tr" returns period; "n" is recorded discharge data period; "m" is the number of events if arranged from the largest to the smallest.

Analysis of the return period depends on the amount of data (n) so that reduced mean (Yn) can be determined based on Table 1, as well as determining reduced deviation standard (Sn) determined from Table 2 based on the amount of rainfall data.

$$SD = \sqrt{\frac{\sum_{i=1}^{n} (R_{24_i} - \overline{R}_{24})^2}{n-1}} \qquad (3)$$

Daily rainfall

$$R_T = \overline{R}_{24} + K.S_d \qquad (4)$$

**Table 2.** Reduced Standard Deviation ($S_n$)

| n | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
|---|---|---|---|---|---|---|---|---|---|---|
| **10** | 0.9496 | 0.9676 | 0.9833 | 0.9971 | 1.0095 | 1.0206 | 1.0316 | 1.0411 | 1.0493 | 1.0565 |
| **20** | 1.0628 | 1.0696 | 1.0754 | 1.0811 | 1.0864 | 1.0915 | 1.0961 | 1.1004 | 1.1047 | 1.1080 |
| **30** | 1.1124 | 1.1159 | 1.1193 | 1.1226 | 1.1255 | 1.1285 | 1.1313 | 1.1339 | 1.1363 | 1.1388 |
| **40** | 1.1413 | 1.1436 | 1.1458 | 1.1480 | 1.1499 | 1.1519 | 1.1538 | 1.1557 | 1.1574 | 1.1590 |
| **50** | 1.1607 | 1.1623 | 1.1638 | 1.1658 | 1.1667 | 1.1981 | 1.1696 | 1.1708 | 1.1721 | 1.1734 |
| **60** | 1.1747 | 1.1759 | 1.1770 | 1.1782 | 1.1793 | 1.1803 | 1.1814 | 1.1824 | 1.1834 | 1.1844 |
| **70** | 1.1854 | 1.1863 | 1.1873 | 1.1881 | 1.1890 | 1.1898 | 1.1906 | 0.1915 | 1].1923 | 1.1930 |
| **80** | 1.1938 | 1.1945 | 1.1953 | 1.1959 | 1.1967 | 1.1973 | 1.1980 | 1.1987 | 1.1994 | 1.2001 |
| **90** | 1.2007 | 1.2013 | 1.2026 | 1.2032 | 1.2038 | 1.2044 | 1.2044 | 1.2049 | 1.2055 | 1.2060 |
| **100** | 1.2065 | | | | | | | | | |

Rainfall data before being used to estimate the maximum discharge is first carried out by the suitability test for rainfall data distribution, with the Smirnov Kolmogorov Test [6]. The Smirnov-Kolmogorov distribution compatibility test is often also called the non-parametric test because the test does not use a particular distribution function.

The procedure for the Smirnov-Kolmogorov test is done by sorting data from large to small and determining the opportunities and each of these data using the equation [8]:

$$p=\frac{m}{n+1}\,x100\% \tag{5}$$

Then determine the theoretical opportunities for each of these data based on the distribution equation:

$$P'=\frac{1}{T} \tag{6}$$

Where "P" is the probability (%); "m" is the sequence number of data after being sorted from large to small; "n" is the number of years of data

From the two opportunity values, the biggest difference is determined between the opportunities of observation with theoretical opportunities:

$$D_{\max imum}=\left[P(Q_{\max})-P'(Q_{\max})\right] \tag{7}$$

Based on the table for the critical value of Smirnov-Kolmogorov (Table 5), the value of "Do" is determined, if "D" is smaller than "Do", the distribution used to determine the planned discharge is acceptable, whereas if the value "D" is greater than "Do", the distribution is used to determine the planned discharge not accepted.

Water availability determined based on rainfall data is then calculated as the rainfall intensity. The calculation of the rainfall intensity with Mononobe equation [6] as follows:

$$I_T=\frac{R_{24T}}{24}\left(\frac{24}{T_c}\right)^{2/3} \tag{8}$$

Time-concentration (Tc) by Kirpich:

$$T_c=0.00013\frac{L^{0.77}}{S^{0.385}} \tag{9}$$

Where "I" is rainfall intensity (mm/hours); "$R_{24}$" is maximum rainfall in 24 hours (mm) and "Tc" is the duration of rainfall.

Some of the measures of runoff concentration could be expressed in terms of slope length and gradient [9], From the results of the rainfall intensity, the maximum amount of discharge is calculated. There are many rational formulas made empirically that can explain the relationship between rain and runoff including:

$$Q=0,278.C\,.I\,.A \tag{10}$$

Where "Q" is discharged ($m^3$/ sec); "C" is run off coefficient; "I" is rain intensity during a time of concentration (mm/ hour); "A" is catchment area ($km^2$)

## 2.2 Flow Discharge

The velocity will increase with the hydraulic radius so that for a given wet look area the discharge will be maximum if the maximum value of R = A / P or minimum wet circumference. To determine the channel cross-section, the channel discharge equation can be used:

$$Q=F.V \tag{11}$$

While to determine the flow velocity (V) the Manning equation can be used:

$$V = \frac{1}{n}.R^{(2/3)} I^{(1/2)} \quad (12)$$

Where "Q" is discharged ($m^3$/ sec); "F" is a cross-sectional channel; "V" is velocity; "n" is the manning coefficient; "R" is the hydraulic radius and "I" is the slope of the channel.

# 3 Result and Discussion

## 3.1 Rainfall Data

The regional drainage service area IV is 0.65 $km^2$ and the longest ($L_1$) long service is 1,300 m, with the nearest long service ($L_0$) 500 m and Slope 0.0013. The 10-year rainfall data from 2007 to 2016 was obtained from Meteorology and Geophysics Lhokseumawe [10]. Rainfall data processing as shown in Table 3

**Table 3.** Rainfall data 10 years from 2007 to 2016

| Years | R | R average | R – R average | (R – R average)$^2$ |
|---|---|---|---|---|
| 2007 | 167.4 | 116.7 | 50.7 | 2,570.5 |
| 2008 | 145.3 | 116.7 | 28.6 | 818.0 |
| 2009 | 72.3 | 116.7 | (44.4) | 1,971.4 |
| 2010 | 115.6 | 116.7 | (1.1) | 1.2 |
| 2011 | 99.9 | 116.7 | (16.8) | 282.2 |
| 2012 | 130.5 | 116.7 | 13.8 | 190.4 |
| 2013 | 91.5 | 116.7 | (25.2) | 635.0 |
| 2014 | 117.1 | 116.7 | 0.4 | 0.2 |
| 2015 | 102.4 | 116.7 | (14.3) | 204.5 |
| 2016 | 125.0 | 116.7 | 8.3 | 68.9 |
| **Total** | **1,167.1** | | | **6,742.3** |
| **Average** | **116.7** | | | |

The amount of deviation standard based on equation (3) is obtained SD is 27.4, so Daily rainfall $R_{24}$ becomes 113.14 mm. The frequency of extreme rainfall events shows significant inter-annual and inter-decadal variations in addition to a statistically significant long term trend [11]

## 3.2 Data Distribution Test for Smirnov Kolmogorov

Kolmogorov-Smirnov test conducted for compatibility analysis of the data distribution whether it is acceptable or not acceptable. Rainfall data 10 years are sorted from the largest to the smallest, then analyzed opportunities for observation and theoretical opportunities. There is a difference in the maximum values used for the comparison. Probability for the analysis as shown in Table 4.

From Table 5 based n = 10 with a degree of confidence α = 5%, the obtained value of Do is 0:41. $D_{Max}$ is smaller than Do the Gumbel Type I Distribution acceptable. To get accurate results on the data an approach to classification modulation is performed based on the Kolmogorov-Smirnov test [12]

Based on the table as shown in Table 4, it shows that the maximum value of "Dmax" is in 2 rows, m = 3 and m = 10, where the Dmax value is 0.01, this value will be compared with the critical value for the Kolmogorov Test-Smirnov (Do) which is obtained from Table 5.

**Table 4.** Kolmogorov-Smirnov Test

| m | R | P=m/(n+1) | T | P' = 1/T | D=\|P-P'\| |
|---|---|---|---|---|---|
| 1 | 167.4 | 0.09 | 11.08 | 0.09 | (0.00) |
| 2 | 145.3 | 0.18 | 5.45 | 0.18 | 0.00 |
| 3 | 130.5 | 0.27 | 3.48 | 0.29 | **0.01** |
| 4 | 125.0 | 0.36 | 2.97 | 0.34 | (0.03) |
| 5 | 117.1 | 0.45 | 2.40 | 0.42 | (0.04) |
| 6 | 115.6 | 0.55 | 2.31 | 0.43 | (0.11) |
| 7 | 102.4 | 0.64 | 1.69 | 0.59 | (0.05) |
| 8 | 99.9 | 0.73 | 1.61 | 0.62 | (0.10) |
| 9 | 91.5 | 0.82 | 1.37 | 0.73 | (0.09) |
| 10 | 72.3 | 0.91 | 1.09 | 0.92 | **0.01** |

**Table 5.** Critical Value ($D_0$) for Kolmogorov-Smirnov Test

| N | α | | | |
|---|---|---|---|---|
| | 0.20 | 0.10 | 0.05 | 0.01 |
| 5 | 0.45 | 0.51 | 0.56 | 0.67 |
| 10 | 0.32 | 0.37 | 0.41 | 0.49 |
| 15 | 0.27 | 0.30 | 0.34 | 0.40 |
| 20 | 0.23 | 0.26 | 0.29 | 0.36 |
| 25 | 0.21 | 0.24 | 0.27 | 0.32 |
| 30 | 0.19 | 0.22 | 0.24 | 0.29 |
| 35 | 0.18 | 0.20 | 0.23 | 0.27 |
| 40 | 0.17 | 0.19 | 0.21 | 0.25 |
| 45 | 0.16 | 0.18 | 0.20 | 0.24 |
| 50 | 0.15 | 0.17 | 0.19 | 0.23 |
| n>50 | $1.07/n^{0,5}$ | $1.22/n^{0,5}$ | $1.36/n^{0,5}$ | $1.63/n^{0,5}$ |

## 3.3 Rain Intensity

The intensity of rainfall in the 2-year return period based on equation (8) is 40.31 mm/hour. From the service area 0.65 $km^2$ with a channel length of 1,350 m and Based on daily rainfall and rain intensity the maximum discharge return period can be calculated as 5.1 $m^3$/ sec. Based on this, it simulated the channel dimensions as a channel dimension with a minimum width of 110 cm and a height of 120 cm. For Manning purposes, the coefficient of roughness (n), and for hydraulics of this flow, that Manning roughness coefficient varies inversely with depth; the Manning roughness coefficient varies directly with slope [13]

## 3.4 Channel capacity

Based on channel properties and slope of existing conditions 0.001333, distance station (STA) 50 m, many stations 27 STA and Manning coefficients (n) 0.015, it can be calculated flow velocity and discharge.

*3.4.1 Existing condition*

From the channel width and the height channel, it can be determined cross-sectional area 1.32 m2 and wet circumference 3.5 m. Hydraulic radius 0.377143 m. Based on the slope of the land of 0.001333, resulting in the flow velocity is 1.27 m/sec. The hydraulic properties of soil flow are the average velocities for channel slope. Many hydrological and hydraulic models require a component of velocity for flow [14]. Then the discharge can be calculated is 1.67 $m^3$/ second.

*3.4.2 Utilizing a Slope Modification*

Based on Station (STA) distance with the number of Station (STA), the longitudinal cross-section can be illustrated as shown in Figure 3. In the downstream, the planned slope height is lower than 0.25 m at each station 50 m distance, slope channel is 0.005. The channel cross-section becomes 1.32 $m^2$ with a wet circumference of 3.5 m. The hydraulic radius is 0.377143 m which produces a velocity of 2.46 m/ sec; this resulting in a discharge capacity of 3.24 $m^3$/ second. Thus, it can be stated that modifying the channel slope will increase the discharge capacity.

# 4 Conclusion and Recommendations

## 4.1 Conclusion

The novelty in this study is the modification of the slope of the channel by making a stage at every 50 meters the length of the channel with a height difference of 25 cm made as many as 27 stages. In the present study, the results of modifying the slope of the channel can be conclusions as follows:

1. The flow velocity of the existing condition was 1.27 m/sec, by doing slope modifications the flow velocity becomes 2.46 m/sec. There is an increased velocity of 1.19 m/sec or 93.7%
2. Discharge on the existing condition was 1.67 $m^3$/sec while by having the slope modification the discharge becomes 3.24 $m^3$/sec. This indicates that there is an increase in channel capacity to flow discharge of 1.57 $m^3$/sec or 93.7%
3. To control flooding on the plains or areas tend to be horizontal; the slope modification technologies can be used.

## 4.2 Recommendation

This study recommends that in areas that have a tendency to land that is likely to be floodplains such as coastal areas, modification of channel slope to increase flow velocity which ultimately increases the discharge is one of the recommended solutions

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