Study of Jetty Planning as Coastal Protection Structure to Address Estuary Sedimentation: Case Study of Lubuk Tukko Beach, North Sumatra Province

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Abstract

Sediment transport from the sea can cause estuary siltation. The effective jetty construction planning is one of problem addressing structurely. To derive the elevation and dimensions of an effective jetty structure in addressing the estuary sedimentation at Lubuk Tukko Beach, North Sumatra Province, wave analysis is conducted using the hindcasting method and tidal analysis using the admiralty method. The research analysis show that the wave height design is 0.676 meters and the highest water surface elevation is 0.616 meters. The type of jetty used is a long jetty with a left jetty length of 394 meters and a right jetty length of 426 meters. The elevation of the right jetty end is 2.5 meters, with a building top width of 1 meter, and the elevation of the left *jetty* end is 2.6 meters, with a top width structure of 1 meter.

Keywords: Jetty, hindcasting, tide, admiralty, elevation, dimension.

INTRODUCTION

The coast is one of the areas around waters that is directly influenced by tides. Coasts are divided into four zones or areas: offshore, inshore, foreshore, and backshore. The zone located from the breakpoint line toward the sea is called offshore, the zone located along the coastline from low tide to peak high tide is called foreshore, the zone located from the low tide line to the breakpoint line is called inshore, and the zone formed by the foreshore and coastline during storms coinciding with high tide is called backshore (Fadilah, 2021). On the other hand, the coast is an area continuously affected by water carried by currents, waves, and tides. These dynamic processes are perpetual and will generate problems that impact coastal conditions.

Problems occurring around coasts, particularly in Indonesia, are caused by two factors: natural factors and human factors. Tidal waves, sedimentation, and strong winds are examples of natural factors. Indonesia is a country with threats or high vulnerability to extreme wave disasters (Mardika, 2024). Meanwhile, human factors include examples such as improper waste disposal, mangrove deforestation, expansion of aquaculture areas, and so forth. Of both factors causing these problems, sedimentation is a highly critical issue in coastal areas, especially at river mouths/estuaries.

Sedimentation is the process of sediment deposition carried by ocean currents/sediment transport or deposition resulting from coastal erosion due to waves and ocean currents/abrasion. Furthermore, sedimentation can also be carried by river currents toward the coast (Suleman, 2023). Such cases are frequently encountered at estuaries, where sediment transport occurs from two directions: from the coast and from the river. This sediment transport meets at the estuary and can cause siltation.

Siltation occurring at estuaries can impact the surrounding environment because water flowing from the river will continuously move toward lower elevations. This situation is extremely hazardous if the estuary elevation becomes higher than the river elevation due to sedimentation. This can disrupt community activities, such as boat navigation routes, and even cause flooding in the areas surrounding the estuary. Therefore, protective structures at estuaries are essential to prevent siltation (Fahmi, 2022).

A jetty is a coastal protection structure perpendicular to the coast and placed on both sides of the estuary. This structure consists of three types, each with its own advantages. First, the short jetty type, whose end is located at the low tide level, functions to prevent estuary deviation. Second, the medium jetty, whose end is located between the low tide level and the wave breaking location, functions as a sediment transport barrier. Third, the long jetty type, whose end is located at the wave breaking point, functions as a barrier against large sediment transport, which will complicate ship navigation routes at the estuary (Triatmodjo, 1999).

Lubuk Tukko Beach, North Sumatra Province, is the case study location requiring a jetty as a coastal protection structure to address estuary siltation problems. Determining the elevation and dimensions of the jetty is crucial for achieving structural effectiveness because the root problems that can occur are inaccuracy and errors in the planning model during the processing of planning data (Fauzi, 2024). In determining the elevation and dimensions, this study aims to determine the design wave elevation for a specific return period, wave deformation, and tidal analysis.

METHODS

The research conducted on the Lubuk Tukko Beach case study, North Sumatra Province, employed a quantitative research method, where data and research components are expressed in the form of statistical data, as presented in tabular form. This study also applied a predictive analysis method, which estimates future wave height and period through regression analysis based on current available data (Purwanto, 2020). At this stage, hourly wind data processing was conducted over 11 years (January 01, 2012 – December 31, 2022) using the hindcasting method, and from this data, outputs were obtained in the form of design wave height analysis results using probability distribution functions. Subsequently, wave deformation analysis was performed, influenced by bathymetric data to obtain the height, depth, and distance of wave breaking, where the data used had intervals of 1 meter and 5 meters, obtained from the National Bathymetry (BATNAS). Following this, tidal analysis was conducted using 29-day tidal data from the Indonesian Geospatial Reference System (SRGI) and processed using the admiralty method to obtain the Highest High Water Level (HHWL) elevation, which was then used to determine the Design Water Level (DWL) elevation, as well as for scientific purposes and data in planning or managing infrastructure around the estuary (Mardika & Pratama, 2021). Finally, armor layer type data was used to determine the structure dimensions; this data was adjusted according to the armor layer types available in the environment surrounding the research location.

RESULTS AND DISCUSSION

1. Wind Data Processing Analysis

The process of determining the design wave height and wave height changes due to wave deformation requires first processing wind data into wave height and wave period using the hindcasting method, then calculating significant wave height and significant wave period to obtain the relationship between the wave height return period and wave period return period, analyzing frequency to obtain the probability distribution function method used in calculating design wave height, and finally calculating wave height due to wave deformation (Lestari, 2022)

a. Dominant wind and effective fetch

The recording location for wave height and wave period was obtained from Copernicus Climate Change Services at geographic coordinates Latitude (1.618138°) and Longitude (98.656868°). Furthermore, the recording location for dominant wind direction was obtained from the FL Tobing meteorological station at geographic coordinates Latitude (1.550000°) and Longitude (98.880000°). The data obtained were wind data from the last 11 years, starting from January 01, 2012 to December 31, 2022. The wind data processing results are integrated into the overall planning stage (Mardika, Mashuri, & Safaraz, 2024) and it was concluded that the largest or most frequent wind occurrence distribution was wind blowing from the Southeast direction (0.2111). However, because the research location is situated to the east of the wind data point location, the wind direction facing the research location itself was selected, namely between Southwest (0.0970), West (0.1079), and Northwest (0.0991). From these three wind directions, the dominant wind direction to be analyzed is derived, namely the West wind direction (0.1079). In summary, this distribution is displayed in **Figure 1**. Subsequently, for the effective fetch value, the effective fetch length in the west direction was obtained as 78.71 kilometers.

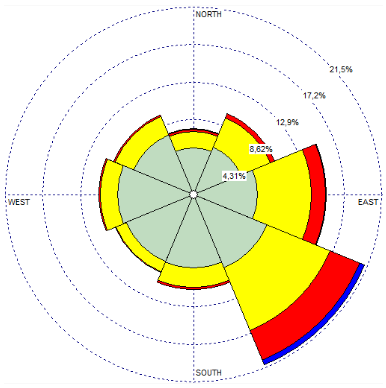

Figure 1. Windrose distribution of wind occurrences based on wind approach direction in 2012-2022

b. Wave height and period

Wave height and period were processed using the hindcasting method. Hindcasting is a forecasting method for future wave height and wave period based on historical data (CERC, 1984). This Coastal Engineering Research Center (CERC) standard is used as a reference in performing wave calculations.

c. Significant wave height and period

Significant wave is defined as the mean value of 33.3% of the highest observed wave heights. This wave height is used for planning flexible coastal structures, such as structures made of rock piles (Triatmodjo, 1999). The significant wave height and period for 2012-2022 are displayed in **Table 1**.

Table 1. Significant wave height and period for 2012-2022

Year	Number of Data	33.3% of Data	Hs (m)	Ts (s)
2012	991	331	0,299	2,776
2013	897	299	0,309	2,869
2014	1020	340	0,319	2,908
2015	1079	360	0,315	2,895
2016	742	248	0,397	3,210
2017	866	289	0,318	2,879
2018	1023	341	0,318	2,857
2019	1057	352	0,309	2,849
2020	1204	401	0,351	3,033
2021	1080	360	0,339	2,997
2022	1076	359	0,405	3,273

Based on **Table 1**, it is concluded that the highest wave height and wave period occurred in 2022, with a formed wave height of 0.405 meters and a period of 3.273 seconds. From this table, a graph showing the relationship between significant wave height and period from 2012-2022 was obtained, as shown in **Figure 2.**

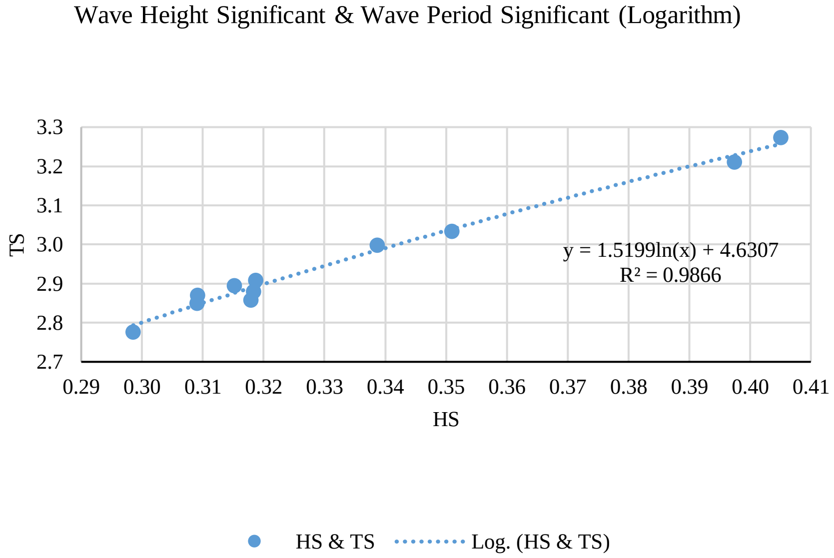

Figure 2. Graph of the relationship between significant wave height and period for 2012-2022

d. Frequency distribution

Frequency distribution is used to determine the probability distribution function that will be used to predict wave height. Two distributions were used: Gumbel Distribution for the Fisher-Tippett Type I probability distribution function and Log Pearson Type III Distribution for the Weibull probability distribution function. The frequency distribution conclusions are displayed in **Table 2**.

Table 2. Summary of Frequency Distribution Results

No	Distribution Type	Criteria		Value	Result	Final Conclusion
1	Gumbel	CS ≤ 1,1396	1,1396	1,328	Not Met	Not Met
		CK ≤ 5,4002	5,4020	4,344	Met	
2	*Log Pearson Type III*	CS ≠ 0	0,0000	1,230	Met	Met

Based on **Table 2,** it is concluded that the distribution type that meets the criteria is Log Pearson Type III, and the probability distribution function used is Weibull

e. Distribution goodness-of-fit test

The distribution goodness-of-fit test is a wave height data test performed on the significant wave data distribution. Two goodness-of-fit tests were used: the chi-square test and the Smirnov-Kolmogorov test. The chi-square test is a statistical test to determine the difference between observed and expected wave height data. The Smirnov-Kolmogorov test is a test conducted to examine distribution differences in different wave height data. The test results showed that the calculated chi-square value was smaller than the fixed chi-square value (met) and in the Smirnov-Kolmogorov test, the Δmaximum value was smaller than the Δcritical value (met) (Widyawati, Yuniarti, & Goejantoro, 2020).

f. Design wave height probability distribution function

Return period calculations are used to estimate the planned wave height to exceed approximately once in T years. A 50-year return period was used because structures in coastal engineering environments typically use a 50-year return period (Sorensen, 1993). This calculation was performed using the Weibull (k = 0.75), Weibull (k = 1.0), Weibull (k = 1.4), and Weibull (k = 2.0) methods. The design wave height for a 50-year return period based on shape parameters is displayed in **Table 3**.

Table 3. Design wave height for 50-year return period based on shape parameter

k	H_{sr} - 1,96σ_r (m)	H_{sr} + 1,96σ_r (m)
0,75	0,240	0,676
1	0,307	0,585
1,4	0,341	0,522
2	0,354	0,482

Based on **Table 3**, it is concluded that the highest design wave height value is at shape parameter (k = 0.75), which is 0.676 meters in height

2. Bathymetric Data Processing Analysis

Bathymetric data are data used to derive seabed depth values in waters. This data is much potentially to use due to a construction planning. Seabed level is one of parameters to calculate transformation or deformation of wave from deep area propagates to shallow water. Bathymetry data is used to determine the location of the structure itself as well. Area of the planning location should have been known either it is gentle or steep slope. These data are collected from the National Bathymetry (BATNAS). The sea depth data are then used in wave deformation analysis. The sea depth data available in the map ranged from 40 m depth to 1 m depth toward the coast.

a. Wave deformation

Wave deformation calculations were performed because waves are affected by seabed depth when in transitional and shallow waters, resulting in wave height changes (T Rizal, 2021). Two types of wave deformation were used in this case study: shoaling and refraction, as well as wave breaking. This occurred because there were no obstacles such as islands from the wave center to the research location. Wave height and wave breaking depth are displayed in **Tables 4 and 5**.

Table 4. Wave height due to refraction and shoaling

D (m)	α_0 (°)	T (s)	d/L	L (m)	C (m/s)	A (°)	K_r	K_s	H (m)
20	3,699	4,036	0,787	25,404	6,295	3,699	1,000	1,000	0,676
15	2,310	4,036	0,591	25,374	6,288	2,307	1,000	0,996	0,673

Based on **Table 4**, it is concluded that the wave height value due to refraction and shoaling in deep water is 0.673 meters. This wave height is at a depth of 15 meters. These height and depth values were then reanalyzed to obtain the wave breaking height and depth, as shown in **Table 5**.

Table 5. Height, type, and depth of breaking wave

d_0 (m)	H_0 (m)	H'_0 (m)	Hb/H'$_0$ (Grafik)	Hb (m)	L_0	Breaking Wave Type	a	b	db (m)
15	0,673	0,673	1,135	0,764	25,404	Spilling	4,068	0,819	0,956

Based on **Table 5**, it is seen that the breaking wave height is 0.764 meters and the breaking wave depth is at 0.956 meters depth. This depth serves as a reference in determining the type of jetty to be used. The jetty type used is a long jetty with a left side length of 394 meters and a right side length of 426 meters, and both ends of these sides are at a depth of 1 meter.

3. Tidal Data Processing Analysis

Tidal calculations are considered to obtain effective elevation and design in planning a coastal protection structure, namely the highest high tide water level. The important level of tidal analysis result is one of parameter that is used to determine top elevation of coastal structure. One of the important level of tidal analysis result is heighest height water level. It is determine how high the water level design in the planning area. The method used in processing tidal data is the 29-day admiralty method, from December 01, 2021 to December 29, 2021. This method produces nine tidal components through eight different schemes to obtain high and low tide water elevations (Setyowati & Zahrina W, 2024). The admiralty analysis results with tidal component values are displayed in **Table 6**.

Table 6. Nine tidal components using admiralty method

Description	S_0	M_2	S_2	N_2	K_2	K_1	O_1	P_1	M_4	MS_4
A (cm)	-0,041	30,496	8,664	8,719	2,339	10,950	5,569	3,613	0,151	0,060
g (°)	156,000	206,416	6,057	232,879	6,057	-153,236	357,272	-153,236	386,497	204,582

Based on **Table 6**, it is given that the highest high tide water level is at an elevation of 0.616 meters, obtained by summing the components $S_0 + (M_2 + S_2 + K_2 + K_1 + O_1 + P_1)$.

a. Design water level elevation

Design water level elevation is water level planning aimed at being able to overcome water level elevation increases caused by periodic water level fluctuations. This elevation determination considers the highest high tide water level, sea level rise due to waves, and sea level rise due to global temperature increase. In addition to these factors, the maximum wave height after waves collide with the structure (run up) must also be considered. The calculation results for design water level and run up are displayed in **Table 7**.

Table 7. Calculation results for design water level and jetty run up

STA	DWL (m)		Run Up (m)	
	Left	Right	Left	Right
0	1,016	1,016	0	0
50	1,050	1,050	0,260	0,260
100	1,066	1,066	0,381	0,381
150	1,081	1,081	0,504	0,504
200	1,096	1,081	0,620	0,504
250	1,110	1,096	0,738	0,620
300	1,124	1,110	0,840	0,738
350	1,138	1,138	0,920	0,920
394	1,138	-	0,920	-
400	-	1,138	-	0,920
426	-	1,138	-	0,920

Based on **Table 7**, it is illustrated that the highest design water level on the left and right jetties is 1.138 meters with a run up height of 0.920 meters. To obtain the crest elevation, the design water level height and run up height must be summed and added with the freeboard value (±0.5 meters).

4. Armor Layer Data Analysis

The type of armor layer used in coastal structure planning is typically the armor layer type available in the vicinity of the planning location. The armor layer type used in this planning is crushed rock with a rough angle.

a. Jetty dimensions

In coastal structure protection planning, dimensions are measurements designed to be effective in dampening waves. Jetty dimensions include structure crest elevation, armor layer stability, structure crest width, armor layer thickness, number of protective rocks per 10 m², and toe protection. To calculate the weight of protective rock grains, it is first necessary to adjust the number of rock grains, slope, stability coefficient on the structure arm and head, considering its placement, whether at wave breaking or not at wave breaking, and the structure slope. Then, to calculate structure crest width, armor layer thickness, and number of protective rocks per area, it is first necessary to adjust the number of rock grains, slope, layer coefficient, and porosity. Finally, to calculate toe protection, the main thickness must be adjusted. Armor layer stability can be determined from the protective rock weight. The jetty dimension planning results are displayed in **Tables 8 and 9**.

Table 8. Calculation results for rock grain weight, crest width, layer thickness, and number of protective rocks per area of jetty

Jetty	No	STA	H1	W 1st Layer	B	t 1st Layer	N 1st Layer
		m	m	kg	m	m	
Left	1	50	1,9	1,362	0,300	0,200	3571
	2	100	2,0	4,597	0,500	0,300	1588
	3	150	2,1	10,898	0,600	0,400	893
	4	200	2,3	21,284	0,700	0,500	572
	5	250	2,4	36,780	0,900	0,600	397
	6	300	2,5	58,405	1,000	0,700	292
	7	350	2,6	87,181	1,200	0,800	224
	8	394	2,6	62,272	1,000	0,700	280
Right	1	50	1,8	1,362	0,300	0,200	3571
	2	100	1,9	4,597	0,500	0,300	1588
	3	150	2,0	10,898	0,600	0,400	893
	4	200	2,1	10,898	0,600	0,400	893
	5	250	2,2	21,284	0,700	0,500	572
	6	300	2,3	36,780	0,900	0,600	397
	7	350	2,5	87,181	1,200	0,800	224
	8	400	2,6	87,181	1,200	0,800	224
	9	426	2,5	62,272	1,000	0,700	280

Table 8 presents the calculated requirements for rock armour along the left and right jetty structures, covering rock weight, crest width, armour layer thickness, and the estimated number of protective rocks for each station. The values show a gradual increase in armour size and quantity as wave conditions intensify along the jetty alignment.

The results in **Table 8** confirm that both jetties require the largest armour rock sizes near the head section where wave energy is greatest. The left jetty head requires a crest width of 1 meter at an elevation of 2.6 meters, whereas the right jetty head applies the same crest width at a slightly lower elevation of 2.5 meters. These findings highlight the importance of tailoring armour design to localized wave forces, ensuring structural stability and long term performance of the jetty system.

Table 9. Calculation results for jetty toe protection

Jetty	No	STA	H	W	2r	Toe Protection Width (B)	d1/ds	Ns^3	S_r	W Toe Protection	Check
		m	m	kg	m	m				kg	
Left	1	50	0,2	0,136	0,3	0,90	0,500	112	2,573	0,049	Ok
	2	100	0,3	0,460	0,5	1,35	0,667	230		0,080	Ok
	3	150	0,4	1,090	0,6	1,80	0,500	112		0,389	Ok
	4	200	0,5	2,128	0,8	2,25	0,333	50		1,703	Ok
	5	250	0,6	3,678	0,9	2,70	0,286	37		3,976	Ok
	6	300	0,7	5,840	1,0	3,15	0,250	30		7,787	Ok
	7	350	0,8	8,718	1,2	3,60	0,333	50		6,974	Ok
	8	394	0,8	6,227	1,1	3,60	0,100	14		24,909	Ok
Right	1	50	0,2	0,136	0,3	0,90	0,500	112	2,573	0,049	Ok
	2	100	0,3	0,460	0,5	1,35	0,667	230		0,080	Ok
	3	150	0,4	1,090	0,6	1,80	0,500	112		0,389	Ok

4	200	0,4	1,090	0,6	1,80	0,200	23	1,895	Ok
5	250	0,5	2,128	0,8	2,25	0,333	50	1,703	Ok
6	300	0,7	5,840	1,0	3,15	0,250	30	7,787	Ok
7	350	0,8	8,718	1,2	3,60	0,333	50	6,974	Ok
8	400	0,8	8,718	1,2	3,60	0,200	23	15,162	Ok
9	426	0,8	6,227	1,1	3,60	0,100	14	24,909	Ok

Table 9 provides the calculated design parameters for toe protection along both the left and right jetties. The results include the required toe protection width, stability number, and corresponding rock weight to ensure resistance against local scour and hydraulic loading at the jetty base.

In general, the results demonstrate that the most critical zone requiring enhanced toe protection is located near the jetty heads, where the toe elevation is 1.1 meters. At this point, a toe protection width of 3.6 meters and a minimum stability number of 14 are required, accompanied by an average rock weight of 24.909 kilograms. These findings emphasize the importance of increasing toe protection capacity toward the seaward end to mitigate the risks of scour, structural displacement, and long term degradation of the jetty foundation.

CONCLUSION

Based on the research results that have been conducted, the researchers can provide the following conclusions: the design wave height for a 50-year return period is 0.676 meters. The wave deformation height due to refraction and shoaling is 0.673 meters, and the wave deformation height due to wave breaking is 0.764 meters. The highest high water level elevation from tidal analysis using the 29-day admiralty method is 0.616 meters. Therefore, the effective jetty elevation and dimensions obtained are a long jetty with a right jetty length of 426 meters and a left jetty length of 394 meters. The left jetty head elevation height is 2.6 meters with a structure crest width of 1 meter, and the right jetty head elevation height is 2.5 meters with a structure crest width of 1 meter. The toe protection elevation height at the left and right jetty heads is 1.1 meters, with a toe protection width of 3.6 meters, a minimum design stability number of 14, and an average rock grain weight of 24.909 kilograms.

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