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Numerical Simulation of Breakwater Layout in Puger Beach Jember Due to Tidal Wave

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Abstract. Puger Beach is located in the south Coast of Java Island which is directly facing the Indian Ocean. The ease of the ship to sail is influenced by the conditions of water in the port which caused by tides. The elevation of the crest of a breakwater is determined by the maximum high water level, while the depth of the shipping lane / port is determined by the minimum low water level. The purpose of this research is to evaluate the breakwater performance by analyzing the surface water level and flow velocity caused by tidal wave in Puger Beach using Delft3D-Flow. The bathymetry data obtained from BIG (Geospatial Information Agency of Indonesia) were utilized for the simulations. Tidal observation results from the previous study were used to validate the model. Two scenarios of breakwater layout were simulated. Spatial variation of flow velocities in the highest tide, the minimum ebb, and representative time steps for both cases are presented. It is shown that using the modified layout of breakwater, the flow velocities become smaller and more stable than that of the existing layout. Using breakwater in the right and left hand side of estuary, high velocities are able to be reduced. Based on the flow velocity pattern in five observation points and the water level, it is confirmed the modified breakwater layout of is more effective than that of the existing one.

Keywords: Breakwater, Puger Beach, Delft3D-Flow

1. Introduction

Puger Beach, located in Puger District of Jember Regency, is the largest fish-producing area in Jember Regency. It is necessary to build the facilities and infrastructures that support fishing activities. One of the most important elements in infrastructure building in coastal area is the coastal safety factor.

There are several studies on breakwater structure in Puger. A study on the current flow patterns at Puger showed that the existing breakwater structure was still not optimal [1]. The structure allowed large waves to pass through the entrance of the channel, resulting in high current speeds. Another study at Puger Beach showed that the design and layout of breakwaters were not significantly effective



in reducing waves [2]. There are few studies on numerical simulation of breakwater layout due to tidal waves. The purpose of this study is to numerically model the alternative layout of a breakwater by considering the flow patterns and water levels caused by tidal wave. It is expected that the design of the breakwater's alternative layout will be useful to plan the location of breakwater.

2. Material and Methods

Research materials and methods were explained in this section. Flow chart of research methods was shown in Figure 1.

2.1 Research Material

The bathymetry data used in this simulation were extracted from the national bathymetry map published by BIG (*Badan Informasi Geospasial*, i.e. Geospatial Information Agency of Indonesia). The resolution of the data was 6 arc-seconds. The bathymetry data in coastal areas and shallow waters were provided by the Center for Marine and Coastal Environment, a division at BIG. The tidal data utilized in this study was obtained from the results of direct measurements and sampling in the field by a consultant company on April 26 to May 10, 2012 at Puger Beach.

2.2 Research Method

This research used descriptive research method. It examined a condition in nature with a systematic interpretation. Numerical modelling was conducted using Delft3D-Flow [3], a hydrodynamic module that solved shallow water equation that able to simulate unsteady flow that is the result of tides, e.g. [4]. In this study, the model was used to predict flows in shallow oceans and coastal areas.



Figure 1. Map of study location (source: Google Earth, 2019)

This study was located in Puger Beach, Jember Regency. Jember Regency covered an area of 3,293.34 km² with a coastline of approximately 170 km. Figure 1 showed a map of the research location in Puger Beach. The breakwater was located at the mouth of the Bedadung River (Plawangan Area).

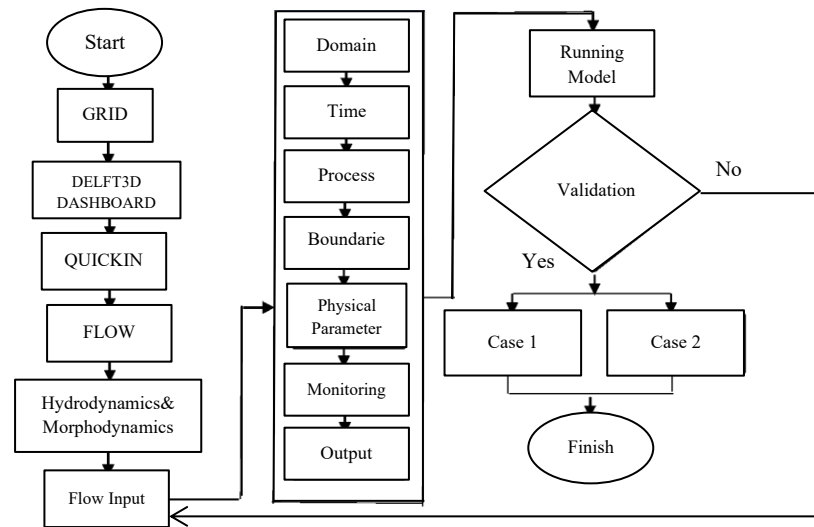


Figure 2. Framework of numerical modelling.

As seen in Figure 2, the implementation of this research was divided into several stages. The initial stage was preparation stage including the construction of simulation grid with 30.0-m resolution using Delft3D-Dashboard. Bathymetry data was interpolated into the grid using QUICKIN. Other input files were inserted in Delft3D-Flow including simulation parameters. Numerical simulations were conducted using Delft3D-Flow. After the simulation was validated using tidal data, the final stage which included the simulations of two cases was conducted.

Case 1 was designed to model the existing condition (Figure 3), while the proposed breakwater layout was included in Case 2 (Figure 4) with breakwater in the right and left hand side of estuary. The breakwater was modelled as “thin dam” in the numerical simulation. Five observation points were located in Figure 3 and Figure 4.

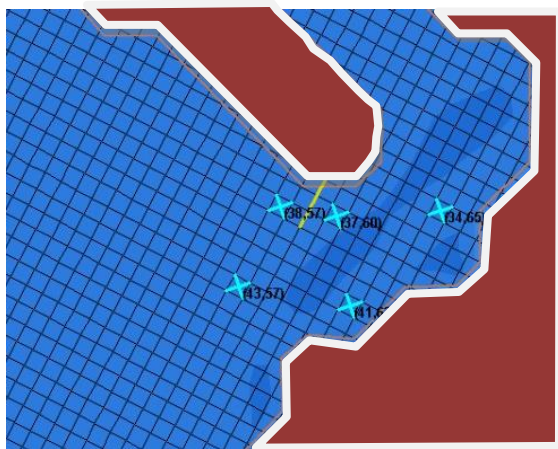


Figure 3. Breakwater Location for Case 1.

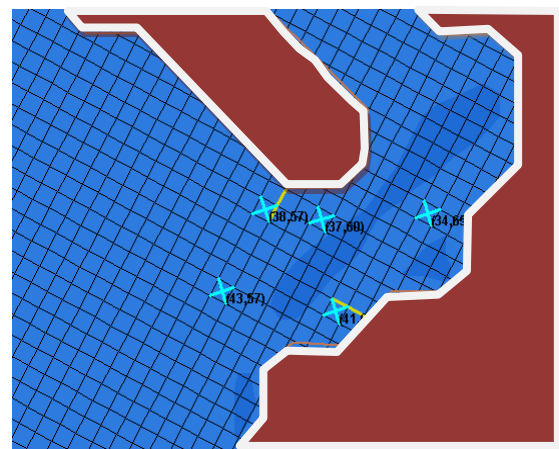


Figure 4. Breakwater Location for Case 2.

3. Result and Discussion

Results of validation and modelling of two scenarios were discussed in this section.

3.1. Model Validation

The validation process was conducted by comparing the tidal observation data with the simulated tidal results. Tidal validation was conducted by comparing the results of water level simulation with the tidal data resulted in 15-day field observation. Figure 5 showed comparison between the field observation data and the simulation results.

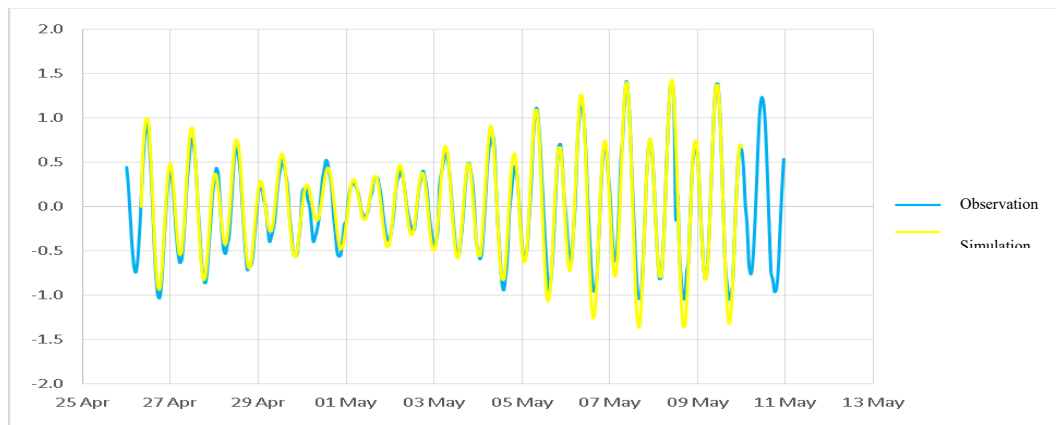


Figure 5. Model validation results.

The tidal height simulation results were compared with the observation data. The error value was calculated using the RMSE (Root Mean Square Error) formula as shown in Eq. (1). After calculation of water level error between modelling results and observational data, RMSE value of 0.125 was obtained. Because the RMSE value was close to zero, it is considered that the error value is small and the model was able to reproduce the observation data. The validation showed that the numerical simulation is validated well using the observation data.

$$RMSE = \sqrt{\frac{\sum_{i=1}^n (y_i - \hat{y}_i)^2}{n}} \quad (1)$$

3.2. Water level

The model was simulated for 15 days, from April 26 to May 10, 2012. The results of the simulation showed that the highest water level for Case 1 was 1.430 meters, while the lowest water level of the existing model was -1.362 meters, as seen in Figure 6 and Figure 7, respectively. The results of the simulation show that the highest water level for Case 2 was 1.432 meters, while the lowest water level of Case 2 was -1.364 meters, as seen in Figure 8 and Figure 9, respectively. The results show that the highest water level of Case 2 was slightly higher than the highest water level of Case 1.

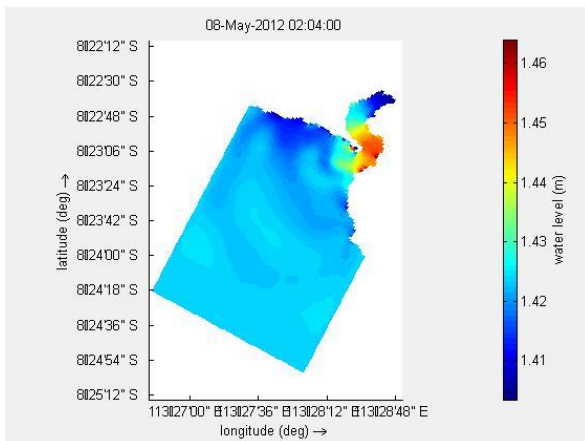


Figure 6. Water level condition of the existing model at the highest tide

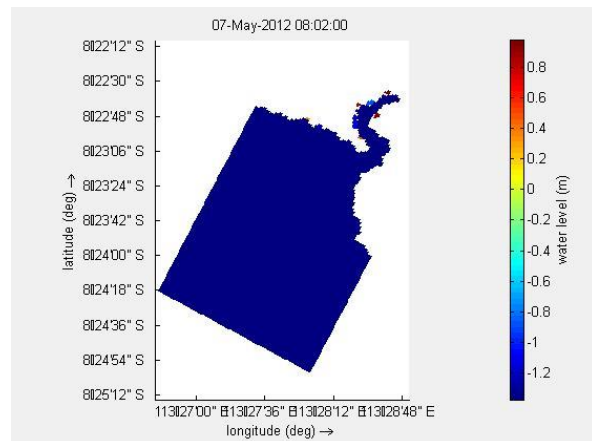


Figure 7. Water level condition of the existing model at the lowest ebb

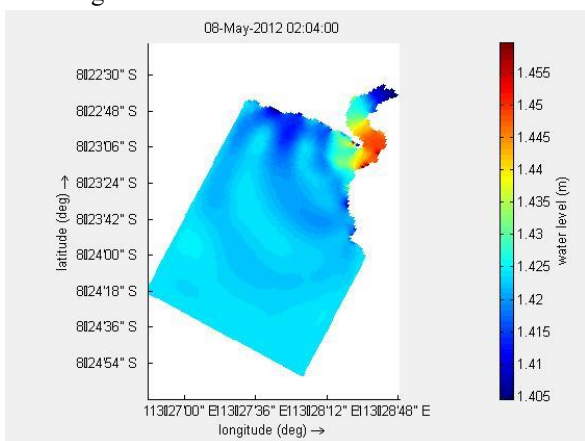


Figure 8. Water level condition of the modified model at the highest tide

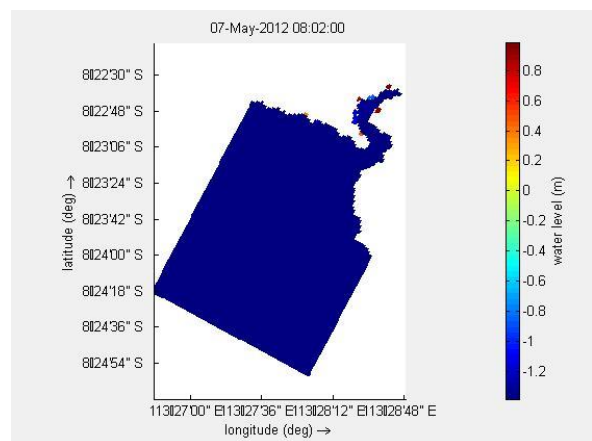


Figure 9. Water level condition of the modified model at the lowest ebb

3.3. Flow velocity at observation points

The maximum and minimum values of the flow for each case were shown in Table 1 and Table 2. It was shown that the average of maximum velocity at observation points of Case 1 (0.0715 m/s) was slightly higher than the average of maximum velocity at observation points of Case 2 (0.0599 m/s).

The results showed that the modified layout of breakwater (Case 2) gave slightly smaller velocity than that of the existing one (Case 1). With smaller velocity, the flow condition of the modified case would be safer for the fisherman than the existing one.

Table 1. Flow Velocity at Observation Points of Case 1

Observation Point	Maximum (m/s)	Minimum (m/s)
(38,57)	0.0641	0.000031
(37,60)	0.0549	0.000009
(41,63)	0.0487	0.000082
(43,57)	0.0971	0.000335
(34,65)	0.0928	0.000031
Average	0.0715	0.000098

Table 2. Flow Velocity at Observation Points of Case 2

Observation Point	Maximum (m/s)	Minimum (m/s)
(38,57)	0.0448	0.000074
(37,60)	0.0526	0.000041
(41,63)	0.0149	0.000055
(43,57)	0.0947	0.000061
(34,65)	0.0928	0.000034
Average	0.0599	0.000065

3.4. Spatial variations of flow velocity

The spatial variations of depth-averaged flow velocity at the highest tide and the lowest ebb for Case 1 and Case 2 were shown in Figs. 10-13. It is shown that at the highest tide, the flow velocities in Case 1 was higher (see Fig.10 and Fig. 12) compared to the flow velocities in Case 2 (Fig. 12) especially in the river mouth. Flow velocities, represented by arrows, were shown longer implying higher values in the river mouth of Case 1 (existing breakwater layout). While for Case 2 (modified breakwater layout), flow velocities in the river mouth were shown as shorter arrows implying smaller values. Using two breakwaters in both sides of estuary (Case 2), flow velocities were able to be reduced. Low velocities would ease fisherman to cross the estuary and minimize turbulences caused by flow.

In the case of the lowest ebb (Fig. 11 and Fig. 13), the differences between both cases were not shown clearly. Higher flow velocities were shown in the upstream area for both cases. This could be occurred due to reflection caused by land boundary in the river upstream.

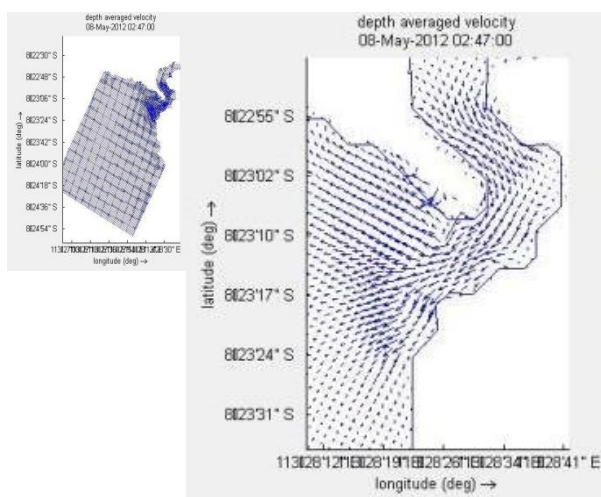


Figure 10. Depth average velocity at the highest tide for Case 1

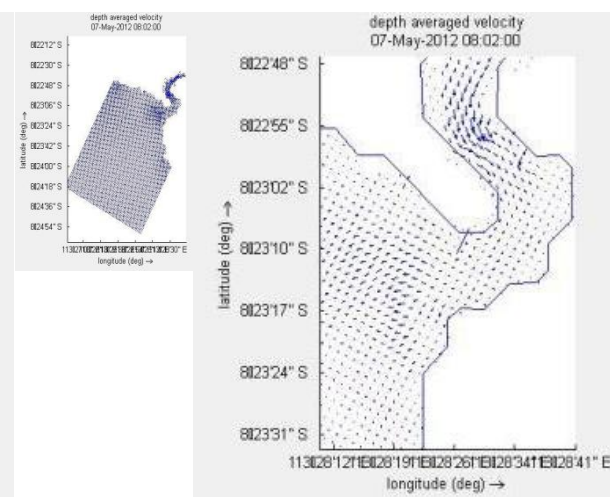


Figure 11. Depth average velocity at the lowest ebb for Case 1

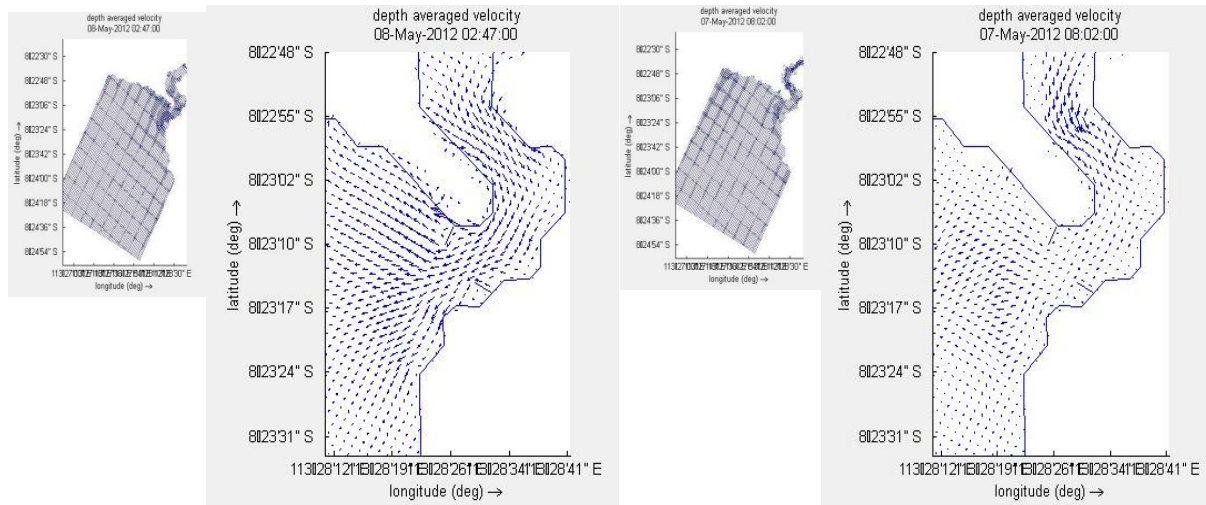


Figure 12. Depth average velocity at the highest tide for Case 2

Figure 13. Depth average at the lowest ebb for Case 2

Spatial variations of flow were also observed in three consecutive minutes (02:46, 02:47 and 02:48) as the comparison between both cases (Figs. 14-19). Figs.14 and 15 showed flow distribution at 02:46 of Case 1 and Case 2, respectively. At this condition, the differences between both cases are not clearly visible. However, one minute later, at 02:47, there are significant differences between both cases. Fig. 16 showed flow velocities of Case 1 which was higher than the condition at 1 minute before. The flows were represented by longer arrows in the middle part and also in the river mouth. At the same time, Fig. 18 showed flow velocities of Case 2. Different from Case 1, flow velocities in Case 2 showed shorter arrows implying smaller values of velocity. At 02:48, both cases show smaller values of velocities. However, still the velocities of Case 1 (Fig. 18) shows higher values than that of Case 2 (Fig. 19).

From spatial variation of flow velocities in the highest tide, the minimum ebb, and representative time steps, the effectiveness of modified breakwater (Case 2) to reduce flow velocities are confirmed. It is shown that using the modified layout of breakwater, the flow velocities become smaller and more stable than that of the existing layout (Case 1). Using breakwater in the right and left hand side of estuary, high velocities are able to be reduced.

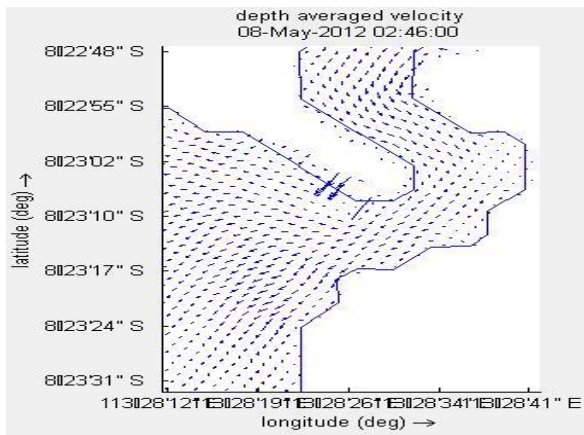


Figure 14. Velocity distribution at 02:46 for Case 1

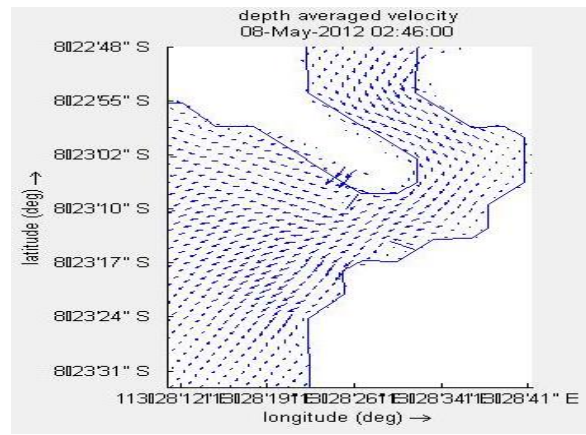


Figure 15. Velocity distribution at 02:46 for Case 2

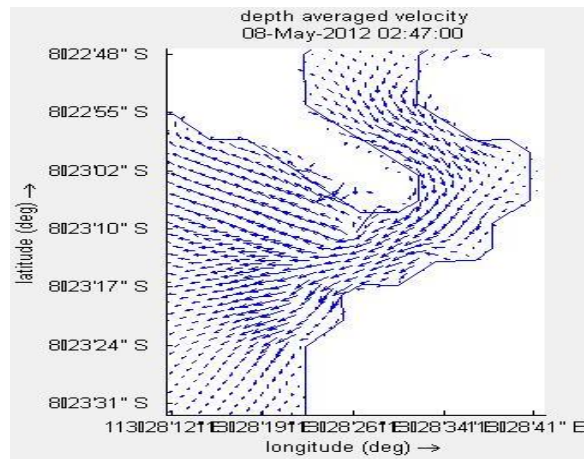


Figure 16. Velocity distribution at 02:47 for Case 1

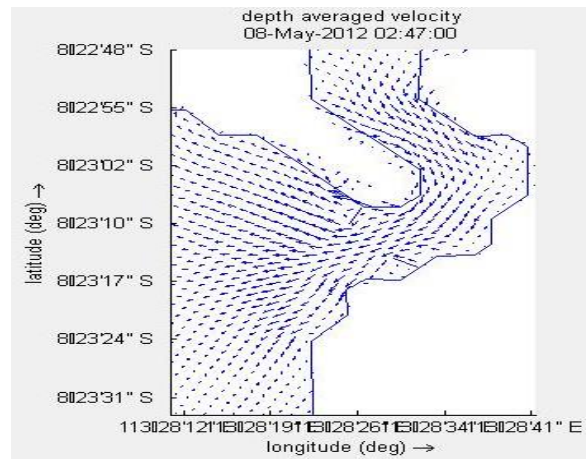


Figure 17. Velocity distribution at 02:47 for Case 2

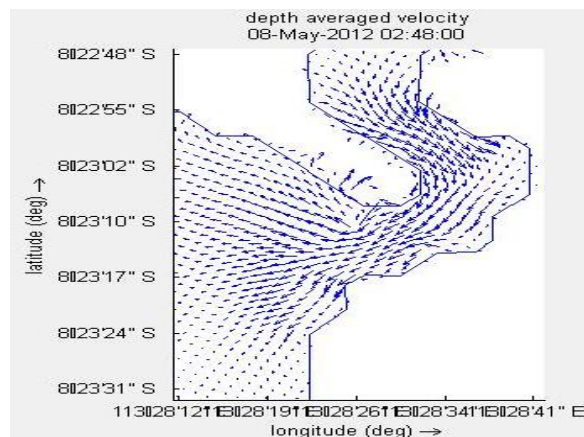


Figure 18. Velocity distribution at 02:48 for Case 1

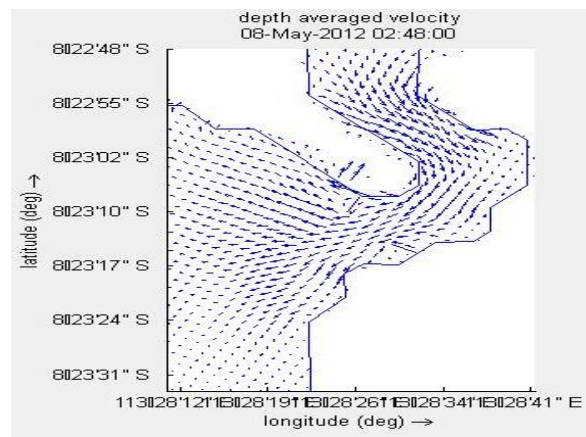


Figure 19. Velocity distribution at 02:48 for Case 2

4. Conclusions

Two cases of numerical simulations are conducted for 15-days in Puger Beach, Jember. The simulations are validated well with the tidal data. Based on the simulations, the results show that the highest water level of Case 1 was slightly higher than the highest water level of Case 2. In the case of depth average velocity, it is shown that the average of maximum velocity at observation points of Case

1 (0.0715 m/s) is slightly higher than the average of maximum velocity at observation points of Case 2 (0.0599 m/s).

Spatial variation of flow velocities in the highest tide, the minimum ebb, and representative time steps for both cases are presented. The effectiveness of modified breakwater (Case 2) to reduce flow velocities is confirmed. It is shown that using the modified layout of breakwater, the flow velocities become smaller and more stable than that of the existing layout (Case 1). Using breakwater in the right and left hand side of estuary, high velocities are able to be reduced. Therefore, it can be concluded that based on the flow velocity pattern and the water level, the breakwater layout of Case 2 is more effective than that of Case 1.

Acknowledgements

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