



INSTITUT PENYELIDIKAN HIDRAULIK KEBANGSAAN MALAYSIA (NAHRIM)
KEMENTERIAN SUMBER ASLI DAN SEKITAR (NRE)

Malaysia Water Research Journal



Vol. 1 (Bil. 1, 2018)





INSTITUT PENYELIDIKAN HIDRAULIK KEBANGSAAN MALAYSIA (NAHRIM)
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Print by

Ezrich Group Sdn Bhd

Printed year

July 2018

ISSN No

2232-0016



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HYDRAULIC STUDY FOR BELIBIS RIVER PUMP SUMP THROUGH PHYSICAL MODELLING

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ABSTRACT

A new Pekan, Pahang flood mitigation project was implemented by Malaysian Government due to frequent flooding event causing various losses of property and lives. A pump station was establish just downstream of the Belibis river for diverting the discharge to reduce the impact of the flooding. Hydraulic model studies are necessary for the pump sumps intake due to the operational requirements of the pumps in a limited space environment which can leads to hydraulic problems. The purposes of this hydraulic model study is to identify undesirable flow conditions in the pump sump model such as vortices, pre-rotation of flow and uneven distribution of flow and propose improvements to the pump sump prototype based on model testing results. Based on the optimum model discharge, an undistorted scale model of 1:10 was adopted. The model features four pumps (7.91 l/s for pump 1 and 2, and 4.74 l/s for pump 3 and 4) with different values of water depth in total of 4 cases of study. Pump flow rate measurements were obtained by ultrasonic flow meter and swirl angle in the suction intakes were measured by a vortimeter. No vortices were occurred near the suction intake for all cases but an uneven flow through the suction intake were detected in some cases. A minor modification is required by installation of buffer block directly under the pump intake column. During detail testing and analysis, the installation of the buffer shown a better flow distribution on all cases.

Keywords: Hydraulic, Physical Model, Pre-rotation flow, Pump Sump, Vortices.

1 INTRODUCTION

Pekan city in state of Pahang, Malaysia and its adjacent areas have been experiencing frequent flooding causing hardship to the local population and significant damages to the economy. To reduce the impact, a new Pekan Flood Mitigation Project was implemented by Malaysian Government whereby pump station was establish just downstream of the Belibis river for diverting the excess flows. Hydraulic model studies are necessary for the pump sumps intake due to the operational requirements of the pumps in a limited space environment which can lead to hydraulic problems such as attached surface vortices; submerged vortices; air entrainment from inflow; swirl and undulating flow; and dead flow regions.

Physical modelling is commonly used during design stages to optimize a structure and to ensure a safe operation of the structure (Chanson, 1999). When used in tandem with numerical modelling, this approach leads to great success for fluid-structure interaction studies and discharge capacity evaluation (Archembau et al., 2001; Piroton et al., 2001; Piroton et al., 2003). Physical model testing is recommended for pumping stations in which the geometry differs from recommended standards, particularly if previous experience with them is not available. Model testing can also be employed to seek solutions to problems in existing installations. If the source of a problem is unknown, it can be less expensive to determine the cause and find remedies by model studies rather than by trial and error at full scale. In this pump sump model test, the pumping station will house 2 nos. of submersible pumps (main pump) and 2 nos. of submersible pumps (jokey pump).

The objectives for this hydraulic studies of pump sump model are to identify undesirable flow conditions in the pump sump model such as vortices and pre-rotation flow and also to propose improvements to the pump sump prototype based on model testing results. The scope of work for the studies are design and construct the physical model of the pump sump; Carry out hydraulic studies on the pump sump model to confirm the suitability of the designed pump sump; Recommendation on modification of pump sump design based on the outcomes of the initial hydraulic studies on the pump sump model; And also carry out subsequent testing on modified pump sump model to confirm the performance of the modified pump sump design.

2 HYDRAULIC MODEL SIMULATION

Froude number modelling is typically used when friction losses are small and the flow is highly turbulent (Chanson, 1999). As the flows studied were mainly controlled by gravity and as the friction losses could be negligible, studies were adopted with the same ratio between inertia and gravity forces as on the prototype. This similarity results in the conservation between model and prototype of the non-dimensional number of Froude (Epicum et al., 2008). After thorough considerations for the most optimum configuration, an undistorted model scale of 1:10 ratio was decided. Thus, in compliance with the Froude Law the corresponding model and prototype relationships was summarised in Table 1.

Table 1. The relationship between prototype and model values.

Property	Scale Relationship	Model Value
Geometric	$S = L_m/L_p$	1/10
Velocity	$V_m = V_p S^{0.5}$	1/3.16
Flow	$Q_m = Q_p S^{2.5}$	1/316
Time	$T_m = T_p S^{0.5}$	1/3.16

A geometrically pump sump model with a scale ratio of 1:10 similar to the prototype had been constructed for this study. Figure 1 and 2 below shows the layout plan and side view of the study area.

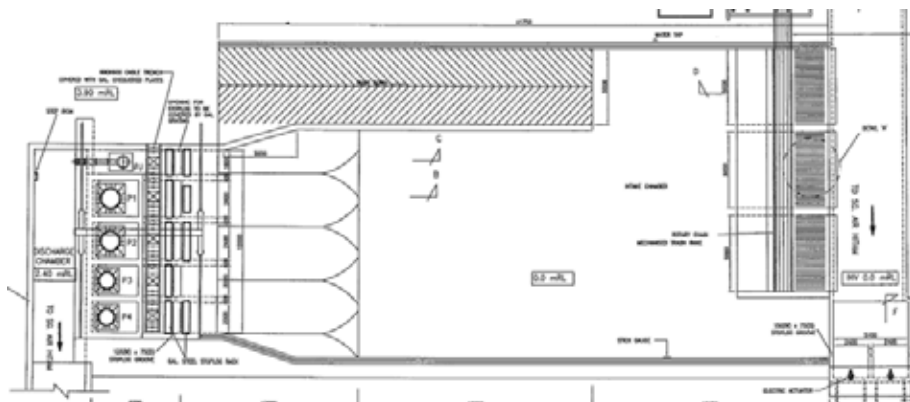


Figure 1. Detail plan view of the pump sump.

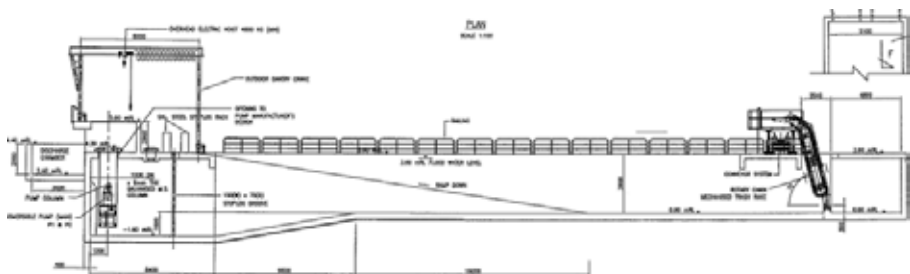


Figure 2. Overall side view of the pump sump.

In order for the pump sump model to maintain the water level at a constant value during observation, a closed loop system using a circulating pump had been built (please refer to Figure 3). This system also enables the regulation of water level in the pump sump model by changing the volume of water contained. In the closed loop of flow system, the flow will be drawn through the pump sump model to the suction side of the external circulating pumps and then into the intake structure of the pump sump model. All the flow for the model testing is measured using the ultrasonic flow meter. In order to control the regulation of flow and the delivery head, a butterfly valve is incorporated in each of the external circulating pumps as a control valves.



Figure 3. Water circulation system and scale model of the pump sump.

The assessment of the pump sump model test involved observing the approach flow pattern towards each operating pump intake with the aid of a blue dye tracer. This gave a good insight as to how the pumps would respond since any flow that departs significantly from the one of the steady flow is undesirable. This uneven distribution of flow approaching the pumps can cause swirl and vortex formation at the pump intake.

2.1 Test scenarios

Tests were undertaken under steady conditions with each duty pump operating at the specified design flow rate and water levels provided by client. Using Froude Number similarity, the model flow capacity for each major pump (P1 & P2) is 7.9 l/s and the capacity for minor pump (P3 & P4) is 4.74 l/s. The test scenarios (please referred Table 2) in this study are as follows:

- A single pump, the P4 (minor pump) in operation at MSL +0.40 which is equivalent to 260 mm from the bottom of the sump (minor pump)
- Two pumps, P4 (minor pump) and P1 (major pump) in operation at MSL +0.90 which is 310 mm from the bottom of the sump (minor and major pumps)
- Three pumps, P4 (minor pump), P1 (major pump) and P2 (major pump) in operation at MSL +1.50 which is 370 mm from the bottom of the sump (minor and major pumps)
- All pumps in operation at MSL +1.80 which is equivalent to 400 mm from the bottom of the sump

Table 2. The relationship between prototype and model values.

Test	Water Level MSL	Major Pump 1	Major Pump 2	Major Pump 3	Major Pump 4
Run 1	+1.80	√	√	√	√
Run 2	+1.50	√	√		√
Run 3	+0.90	√			√
Run 4	+0.40 (Min level)				√

3 RESULT AND DISCUSSION

3.1 Vortices

Ideally, the flow of water into a pump should be uniform, steady, without swirl and without air, either entrained from a free surface or released from local low pressure regions. Lack of uniformity can lead to reduction of efficiency. Unsteady flow will result in fluctuating loading of the propeller, leading to noise and vibration. Swirl in the intake can cause a change in flow, reduction in the pump efficiency and power. It may result in vortices leading from the free surface or from a bounding solid surface into the pump. These vortices can become strong enough for the cores to be air filled or cavitation. Vortices from the water surface can draw air continuously into the pump; solid surface vortices, often called submerged vortices, provide discontinuities in the flow around the propeller blades.

The free vortices are generated by rotation and separation combined with the effects of accelerated flow at the pump intake. Coherent subsurface swirl is often apparent, sometimes penetrating into the intake but again, since only gravitational and inertial forces are involved Froude Number similarity accurately predicts the magnitude of such effects. In the extreme, air core vortices will be generated which may have serious consequences. Such vortices, with air core extending near to a model intake, will be subjected to scale effects. In a full-size installation, due to the absence of scale effects, the transition from a surface dimple and coherent subsurface swirl to an air core will occur more readily than in the model.

In order to achieve the objectives, it is necessary to investigate the performance of the pump sump as designed and to make modifications to overcome any encountered problems. The assessment of the pump sump involved observing the approach flow pattern towards each operating pump intake with the aid of dye tracer. The tests were undertaken under steady conditions with each duty pump operating at the specified design flow rate and water levels. When a single pump in operation it was conducted at lower water level of 260 mm (from the bottom sump) and at 400 mm water level when all major and minor pumps are in operation. Observations were made from the dosing of the blue dye at the entrance and immediately downstream of the

slope intake showed that the dye had a tendency to flow towards the centre of the suction column for all test scenarios as in Figures 4. Thus, this indicates that the proposed pump sump design work well during all pumps operation cases.



Figure 4. The approach flow pattern towards pump intake with the aid of dye tracer.

3.2 Pre-rotation and swirl at pump intake

Swirl is a general term for any flow condition where there is a tangential velocity component in addition to a usually predominating axial flow component. Pre-rotation is a specific term to denote a cross sectional average swirl in suction line of a pump or, in case of a vertical wet pit pump, upstream of the impeller. Pre-rotation will influence the pump performance since the flow approaching the impeller already has a rotational flow field which may oppose or add to the impeller rotation, depending on direction. The design of the pump impeller vanes i.e. shape and angle assumes no pre-rotation and the existence of pre-rotation implies flow separation along one side of the impeller vanes. The degree that should be of concern is dependent on the type of pump and may not always be known. Pre-rotation should be quantified in a model by an average cross sectional swirl angle, determined by detailed velocity measurement or by reading on a swirl meter. Since swirl decays along a pipe as a result of wall friction, internal fluid shear and turbulence, the swirl meter in the model suction pipe should be located near the impeller.

The method to determine the degree of vortices is using Vortimeter (please referred Figure 5) developed by Aden Research Laboratory. The Vortimeter was constructed and mounted in each of pump column intake suction for swirl angle measurement. The swirl angle was determined using the following relationship:

$$\tan \theta = \left(\frac{V_R}{V_A} \right) \quad [1]$$

Where: V_R is the rotational velocity of the swirl meter vanes, and
 V_A is the axial velocity in the pump intake.



Figure 5. Vortimeter used to detect swirl or rotational flow at the pump intake.

The counting swirl angles are indicated that the results are within the acceptable limit with the installation of the proposed anti-swirl device on the bottom of intake column. As recommended by Ansar (1997), if the swirl angles are less than 5° , the model indicated that the incoming swirl is very minimum and insignificant. In sum, these results could be considered within the acceptable limit.

From Table 6 through 9, it was found 3 cases do not meet the requirement of the vortimeter angle. Pump 1 and pump 2 for the Case 1 (all pumps in operation), pump 2 for the Case 2 (3 pumps in operation) and finally for pump 4 in the Case 4. These mean that distributions velocity entered the inlet column with relative larger angle (more than 5°) that the limit suggested by Ansar (1997). There is possibility for the inflow angle leads to unbalanced pump impellers. Thus, it is recommended to install a triangle buffer block directly under the intake column.

Table 6. Pump configuration and swirl angle degree for case 1.

Test 1	PI	P2	P3	P4	Remark
swirl angle (°)	4.8	0.0	0.9	0.4	cw
swirl angle (°)	0	15.2	2.8	2.1	acw

Table 7. Pump configuration and swirl angle degree for case 2.

Test 2	PI	P2	P3	P4	Remark
swirl angle (°)	4.0	5.3		3.0	cw
swirl angle (°)	0	1.2		1.6	acw

Table 8. Pump configuration and swirl angle degree for case 3.

Test 3	PI	P2	P3	P4	Remark
swirl angle (°)	0.6			0.1	cw
swirl angle (°)	0.2			0.6	acw

Table 9. Pump configuration and swirl angle degree for case 4.

Test 4	PI	P2	P3	P4	Remark
swirl angle (°)				9.1	cw
swirl angle (°)				0.0	acw

3.3 Modification and improvement at pump intake

Since the initial proposed design indicated the existence of some degree of uneven flow distribution in the intake column sections, modification tests are required. The retests were conducted with a triangle buffer blocks directly under all of the pump intake column.

The detail modifications of a triangle buffer block directly under the pump intake column are shown Figure 6. The buffers divide the under current flow on the pump sump and direct them straight to the intake column. These buffers prevent from the occurrences of cross current directly under the intake column. Therefore, these could lead to a better flow distribution.



Figure 6. Modification of a triangle buffer block directly under intake column at pumps.

The reading and the calculated angle values of vortimeter with a triangle buffer block are shown in Table 10 through 13. The modification minimize the angle of flow and provide a better flow distribution straight to the pump intake column. These buffers also prevent under current to cross each other. Further, the 45° face of the buffer provide a smooth transition of incoming front flows.

Table 10. Swirl angle degree with a triangle buffer block for case 1.

Test 1	P1	P2	P3	P4	Remark
swirl angle (°)	0.9	0.4	0.3	0.0	cw
swirl angle (°)	0.0	0.6	3.3	1.9	acw

Table 11. Swirl angle degree with a triangle buffer block for case 2.

Test 2	P1	P2	P3	P4	Remark
swirl angle (°)	0.9	1.4		0.7	cw
swirl angle (°)	0	0.8		1.0	acw

Table 12. Swirl angle degree with a triangle buffer block for case 3.

Test 3	P1	P2	P3	P4	Remark
swirl angle (°)	0.6			0.3	cw
swirl angle (°)	0.0			0.2	acw

Table 13. Swirl angle degree with a triangle buffer block for case 4.

Test 4	P1	P2	P3	P4	Remark
swirl angle (°)				3.0	cw
swirl angle (°)				0.0	acw

4 CONCLUSION

The results from the initial testing conclude that the proposed design for the Belibis River Pump Sump are not acceptable at three pump protocols. First when all 4 pumps in operations (Case 1 - 2 major and 2 minor). The occurrences of uneven flow distribution into the pump intake columns that indicated by the entrance degree of vortimeter greater than 5°. Second when 3 pumps in operations (Case 2 - 2 major and 1 minor). The occurrences of uneven flow distribution into the pump intake columns that indicated by the entrance degree of vortimeter greater than 5°. And the last one when only 1 pump in operation (Case 4 - 1 minor). The occurrence of uneven flow distribution into the pump intake column that indicated by the entrance degree of vortimeter greater than 5°. Thus, a minor modification is required by installation a triangle buffer block directly under all of the pump intake column as shown in Figure 6. During detail testing and analysis, the installation of the buffer shown a better flow distribution on all cases.

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BED LOAD CHANGES IN ESTUARY DUE TO SEA LEVEL RISE AND HYDRODYNAMIC REACTION

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ABSTRACT

The complexity of estuarine hydrodynamics is due to the continuous interaction between freshwater and saltwater. The hydrodynamic process is influenced by marine factors such as tides, waves, salt water intrusion, river discharge, sediment type and shape of the mouth. This research attempts to investigate the bed changes at Kuala Pahang due to sea level rise-induced hydrodynamic reactions. The study area experiences dominant mixed semidiurnal tides and is projected to be hit with sea level rise of 0.034, 0.144 and 0.307 m in years 2020, 2060 and 2100, respectively. The rate of sea level rising is the third highest, at the East Coast of Malaysia after Kelantan and Terengganu. The increase in sea level caused changes in water depth, bathymetry and hydrodynamic pattern. The increase in flow velocity promotes erosion and the reduction in flow velocity lead to sediment deposition. MIKE 21 software was used in the numerical modeling to analyse the hydrodynamic processes while Sand Transport (ST) module was used to study the sediment transport. The sea level rise factor was combined to obtain the changes in the study area. The study is limited to the southwest monsoon. Modelling work was conducted for a temporal scale of 14 days, taking into account a full tidal cycle. The historical analysis of bathymetry changes was done by comparing the bathymetry data in 1952 and 2014 using ArcGIS software. The analysis showed that the bathymetry has changed between -0.08 to +2.9856 m in the last 62 years. Hydrodynamic modeling results indicate that the flow velocity changed between -0.01 - +0.18 m/s on average, and maximum between -0.06 - +0.2 m/s. Sediment transport modeling shows the average rate of bed level change is between 0.2 to 0.6 m/dy and average bed load changes was between 0.0002 - 0.0008 m³/s /m. Analysis has shown the southern part of estuaries, meanders of rivers and near island experienced decrease in flow velocity which indicate sedimentation.

Keywords: sea level rise, hydrodynamic, bed load changes

1 INTRODUCTION

River and sea hydrodynamic processes such as tidal and waves are causing the erosion and sedimentation in the estuary. The tidal area (i.e. the intertidal zone), receives continuous alternating current which is vulnerable to changes due to sediment transport. The morphology of the river and the beach slope, roughness grooves, the type and shape of the grooves, the orientation of the coast, meeting creeks and vegetation also play a role in the sediment transport processes. In addition, sea level changes and vertical land movement is seen to contribute to the dynamics of sediment transport. Deposition and erosion resulting from the sediment transport process affect the navigation system, the flow of water, flora and fauna as well as the aesthetics value of the area.

Sea level is influenced by changes in the geoid, human activity, greenhouse effect, changes in the volume of water bodies and sea basin. Changes in the sea level increased the flood level, current velocity, longer seawater intrusion, loss of property and coastal areas, the risk of disease and adverse effects on agriculture, aquaculture, water quality and socio-economic development. The magnitude of the sea level rise impact is also affected by the vulnerability characteristics of the coastal area. Changes in environmental surroundings affect the tide and eventually the benchmark for the estuary. Sea level rise increases the depth of the sea water and the hydraulic and hydrodynamic processes of an area are subsequently modified.

Siti Waznah et. al. (2010) stated estuaries and rivers are often the high deposition area, serves as a trap for the minerals from river and sea materials from being transported to the shore. Furthermore, the tidal river upstream direction to play a role in determine the direction of sediment transport in shallow waters (Jing et. al., 2013). Tides and waves carries sediment from the ocean to the estuary resulting the dynamic and continuous events of filling and sediment entrainment within the area.

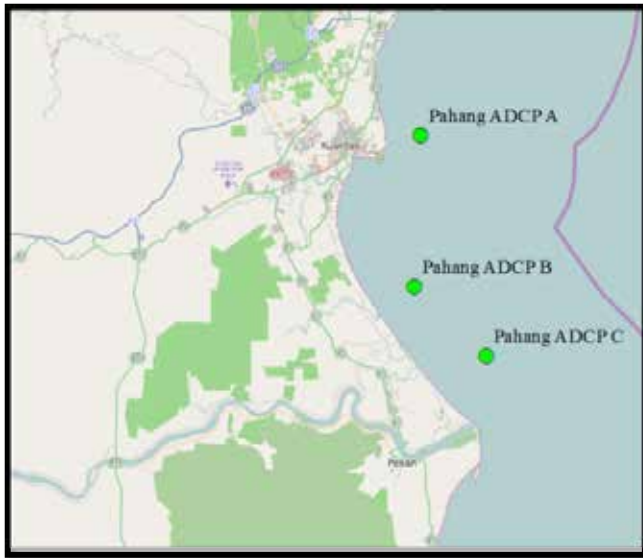


Figure 1. The map of Sungai Pahang and Kuala Pahang estuary.

2 STUDY AREA

Sungai Pahang is the longest river in Peninsular Malaysia with 459 km length and flows into the South China Sea, as shown in Figure 1. This meandering river carrying the water through the various cities like Jerantut, Temerloh, Maran, Bera and Pekan before flowing into the sea through the estuary, Kuala Pahang. Tasik Chini, the second largest natural lake in Malaysia has an area of 202 ha water bodies and 700 ha fresh water swamps with an averaged concentration of total suspended sediments (TSS) at 1755,242 tons/km²/year (Toriman et al. 2012). Deposition of sediments can be seen in the downstream part of the river with the formation of sand bars and small islands or delta. The presence of small islands is also feared to serve as a barrier to the flow of the river and in turn might trap more sediment.

River mouth or estuary is an area where freshwater meets saltwater and influenced by tides and wave action. Ishak et.al., (2001) stated that the formation of the mouth is affected by sediment transport process and this process is influenced by tides. According to Abd. Rahman et. al. (2005), the geomorphology at the Pahang coastal area can be divided into three parts, namely sandy beach (at the northern part), asymmetrical cone shape of Kuala Pahang estuary delta and the third part is dominated by mangrove plants and lagoon (at the south coast).

The Jabatan Pengukuran dan Pemetaan (JUPEM) tide table, (2014) has shown the tides in Tanjung Gelang and Tioman are dominantly mixed tidal semidiurnal where the tidal range is between 1.41 m and 1.56 m. Waves on the East Coast of Peninsular Malaysia during the South-West Monsoon season have heights which rarely exceed 1.8 m and a wave period of less than 6 s. Figure 2 shows a significant direction of the waves in the waters of Pahang is between 180° - 210° with dominant height between 1.75 - 2.75 m. JPM (1985) also indicated that the velocity of the wind during the South-West Monsoon rarely exceeds 15 m/s and the significant direction is from south to west. Keller & Richards (1967) also illustrate the dominant wind direction during South-West Monsoon for Peninsular Malaysia is about 22% from 135° direction as in Figure 3.

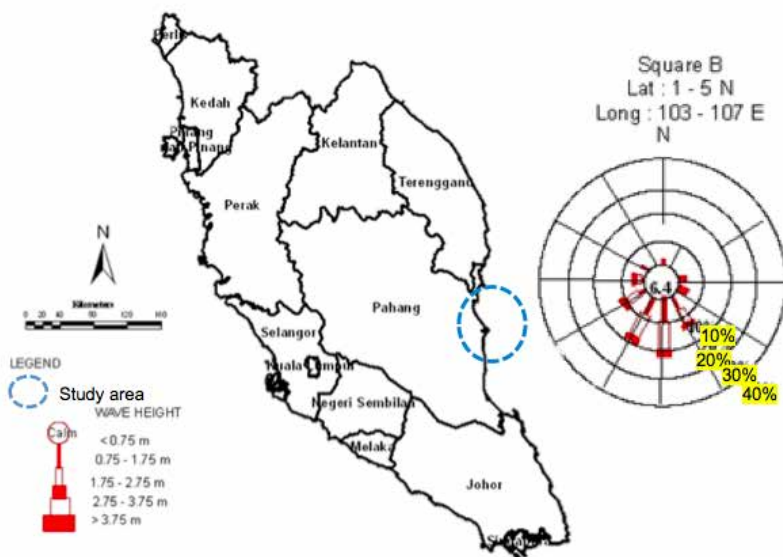


Figure 2. Annual Wave Height. Unit m. (JPM, 1985)

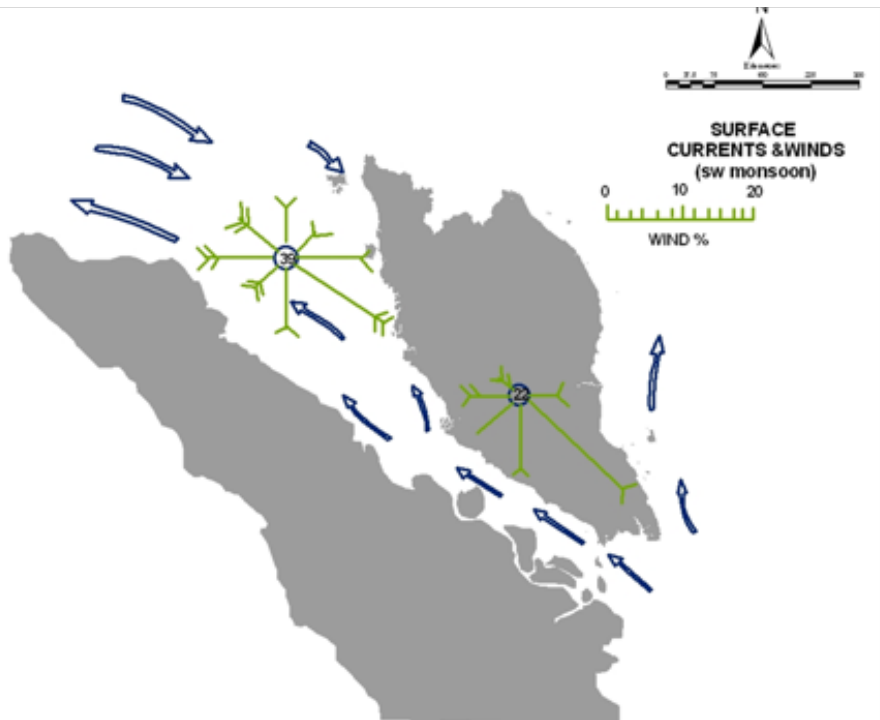


Figure 3. The Dominant Wind Direction at the Peninsular Malaysia during the South-West Monsoon. (Keller & Richards, 1967)

JPM (1985) stated that the east coast of Peninsular Malaysia mostly is sloping beach, shallow and notched lay of which 90% of the average median grain size, D50 was found between 0.17 to 0.48 mm. Study by Waznah et. al., (2010) reports that the sediment at upstream is coarser than the ones found at the downstream area for both seasons. The variation in sediment size along the stream is due to the gentler slope at downstream and resulted in much lower fluid velocity which promotes sediment deposition.

Pahang River originates from Mount Tahan and flows as far as 440 km from the height of 2187 m and is the main tributary of Sungai Jelai and Tembeling River (Tachikawa et. al., 2004). Other tributaries contributing to the flow and discharge of the river is Sungai Chini, Sungai Yap, Sungai Lubuk Paku and Sungai Temerloh. The average water discharge of Sungai Pahang from years 1980 to 2009 was 845.78 m³/s, measured at the Sungai Yap telemetry station, while in station in Temerloh produced 1008.50 m³/s discharge and about 1184.46m³/s flow was measured at Lubuk Paku (Pan et. al., 2011). Note that the telemetry stations of Sungai Yap, Temerloh and Lubuk Paku are located along the Sungai Pahang, with Sungai Yap station is located at the most upstream. The study area receives

an annual rainfall of 2170 mm, which mostly falls in the North-East Monsoon and the average annual temperature in Kuantan is 26.4 °C with an average relative humidity of 86%. The morphology of this area especially Sungai Pahang and Sungai Tembeling mainly composed of alluvial soils as bed material and has different depths of less than 1 m to over 18 m (Tachikawa et. al., 2004). The beach area is also mostly flat and swampy, with granite soil consists of coarse and fine sand and clay dominate this area.

3 METHOD

3.1 Sea Level Rise

Rising sea levels caused the shoreline to retreat to the mainland while the decreasing sea level promotes bigger beach (JPM, 1985). The surface level of the world's oceans has increased by 1 mm/year due to the melted glaciers due to higher temperature, which subsequently increasing the volume of ocean water. Md Din (2014) based on altimetry and vertical-corrected tidal data concluded that sea level is rising at rate of 4.47 ± 0.71 mm/year for Malaysian region, where the average rate for South China Sea is at 3.77 ± 0.54 mm/yr.

NAHRIM (2010) has predicted sea level rise impacts of climate change in the locations along the coast of Malaysia for the years 2020, 2040, 2060, 2080 and 2100. The study was based on data from tide gauges and satellite altimetry for the years 1993 to 2009 and combined with the Atmosphere-Ocean coupled Global Climate Models or General Circulation Models (AOGCMs). The results of this study have shown some areas along the coast of Malaysia are experiencing projected sea level rise of between 0.2 - 1.1 m in year 2100. Areas which is expected to experience the highest sea level rise are Kedah, Kelantan, Sungai Sarawak estuary and east coast of Sabah with the range of increment is between 0.4 - 1.1 m. The study forecasted an average increase for the waters of Pekan and Pulau Tioman can reach up to 2.73 mm/year and 2.88 mm/year. Comparison with the year 2010 showed an increase sea level of 0.034 m, 0.144 m and 0.307 m in years 2020, 2040 and 2100, respectively, which is speculated to pose effects on the hydrodynamic process at the estuary. Figure 4 shows projections of sea level rise for Malaysian waters.

Table 2. Water level station

Station	Latitude	Longitude
WL A	3° 31' 50.400" N	103° 27' 44.000" E
WL B	3° 48' 35.600" N	103° 20' 09.800" E

3.3 Numerical Model

Analysis of the historical changes in bathymetry of the study area was carried out by making a comparison between the observed bathymetric data in 1952 and 2014. The bathymetric data for year 1952 is presented in fathoms unit, where 1.0 fathoms unit is approximately 1.83 m. Analysis of morphological changes were implemented by using Geographic Information System (GIS) of ArcMap version 10.1. Subsequent conversion unit was done to enable comparisons to be made. MIKE 21 software (DHI) was used in this study. The module hydrodynamic (HD), spectral wave (SW) was utilized for the analysis of wave, whereas sediment transport (ST) was used for sediment analysis. Hydrodynamic model was built in mesh generator and the boundary cover from Beserah to Kampung Alur Pasir where the bathymetry data were observed as plotted in figure 5. Bathymetric profile was generated by combining data extracted from MIKE C-Map and bathymetry data obtained from the field measurement campaign. Once the data is interpolated, the boundary conditions for the study area were determined by the Global Tide Model MIKE 21.

Modelling work was run for 14 days, that is similar temporal period done for the marine data campaign and covers a full tidal cycle. Simulations were carried out on 22 May – 8 June 2014 with a lag of 60 seconds and the concept of flood and dry was applied in this model. The seabed resistance value used is between 25 m^{1/3}/s - 60 m^{1/3}/s, depending on the sea depths. According to Lee (2011), coastline with the presence of plants such as mangroves have rough and high resistance values for the seabed.

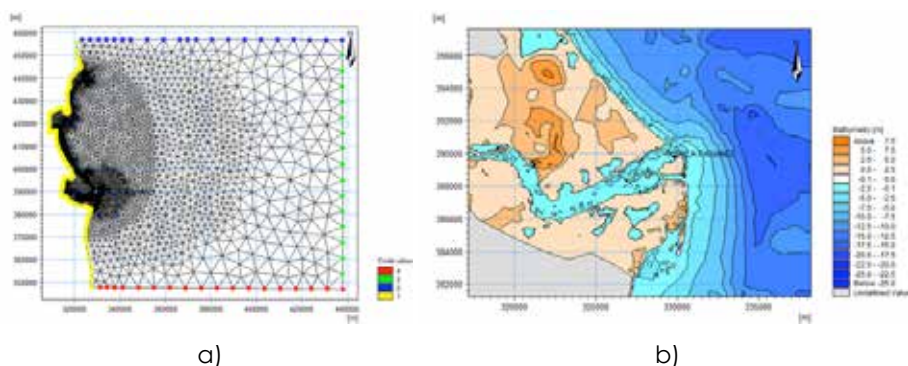


Figure 5. The numerical model developed showing a) mesh generator and b) topography and bathymetry data

4 RESULT AND DISCUSSION

4.1 Field Measurement

The highest speed measured during the measurement period at the observed location A and B, respectively were 0.33 m/s and 0.25 m/s. While the lowest reading for the measured current speed were 0.03 and 0.04 m/s. Analysis of the observed data for the current direction showed that the current flows in the 200° and 10° directions in location A and while the dominant flow is at 320° and 180° in location B. Wave data analysis showed the dominant wave direction is 80° to 120° in location A and 120° to 140° at location B. Location A indicates the wave height up to 0.4 m while the maximum wave height at location B exceeded 0.6 m. Contours show the mainland area has heights exceeding 7.5 m and the sea has depth exceeding 25 m towards offshore and become shallower when approaching the coastline.

4.2 Historical Bed Level Changes

Analysis of the 2014 bathymetry data found that the depth of the seabed in Pekan water is between 1.18 to 16.1 m according to the lowest chart datum. Comparisons between 62 years data (that is the 1952 and 2014 map) indicate the study area has experienced changing in depth between -0.08 to 2.9856 m. The majority of the areas experience increasing depth which indicates erosion while northern Kuala Pahang suffered reduction in depth which demonstrates deposition. Figure 6 illustrates the changes of bathymetry within the span of 62 years.

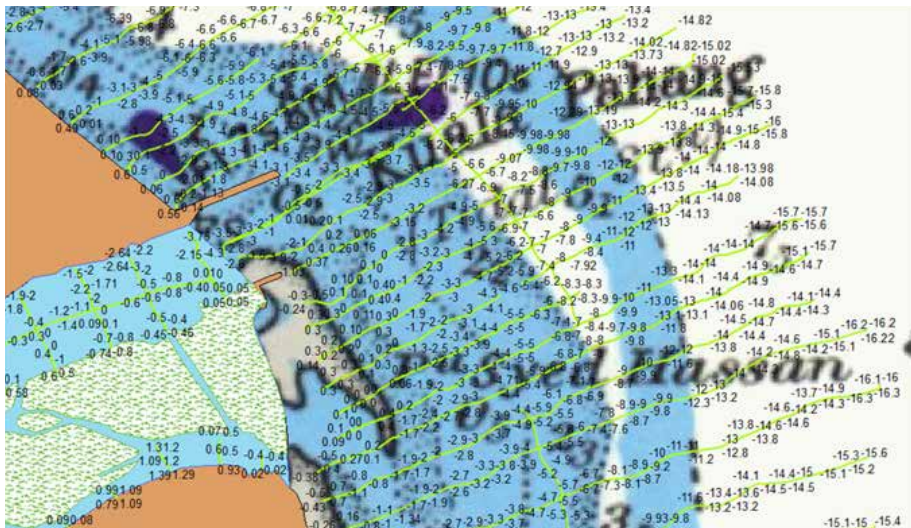


Figure 6. Changes of the bathymetry depth over 62 years

4.3 Hydrodynamic Model

The hydrodynamic modelling only considered the tidal effect. Based on the hydrodynamic modeling, the average current speed in the study area is 0.1 - 0.8 m/s which the highest speed occurred at the estuary. Results also showed that the maximum current speed for the study area is 1.8 m/s and along the river it reaches up to 0.7 m/s. The current speed was lower in the ocean compared to the river with a difference of 0.2 m/s. Modelling of the study area for the projected sea level rise for 2020 with 0.034 m show increment in current speed. The average current speed is expected to be 0.15 - 0.85 m/s. Analysis for 2020 showed the estuary has the highest current speed of 0.85 m/s while the river recorded an average flow velocity of 0.3 - 0.5 m/s. The maximum current speed in Kuala Pahang reaches up to 1.8 m/s and the river can go up to 0.9 m/s. Projected sea level rise of 0.144 m in year 2060 predicted that the study area will experience an increase of average current speed between 0.2 - 0.88 m/s. Statistical analysis showed the maximum current speed of the study area can reach between 0.8 - 1.9 m/s.

Projected sea level rise of 0.307 m in year 2100 showed the average current speed in Kuala Pahang ranges between 0.3 - 0.9 m/s and can reach a maximum speed of between 0.6 - 1.9 m/s. The maximum difference of current speed at the estuary showed increment of 0.1 - 0.2 m/s while the island area, meanders of the river and the southern estuary experienced a decline of 0.06 - 0.1 m/s. Models comparison for 2020 and 2014 show an increment in the average current speed to 0.02 - 0.04 m/s and the maximum speed can go to 0.02 - 0.1 m/s. The analysis also showed a reduction in the current speed between 0.001 - 0.02 m/s. Comparison of model years 2060 and 2014 show the average difference between the current speed 0.02 to 0.05 m/s and the maximum difference is 0.05 to 0.06 m/s. The analysis also showed that the area recorded a decrease in current speed between 0.02 to 0.05 m/s near the island and south of Kuala Pahang. Comparative analysis of model for years 2100 and 2014 shows the study area having increment in the average current speed between 0.03 to 0.18 m/s in the estuary and a slight decrease in the average current speed in the river and southern estuary of 0.01 m/s. See Table 3 for the predicted average and maximum current speed, residual current speed for the prospective years discussed here. The residual current speed at the estuary mouth for the future years was calculated based on the variation between the predicted sea level and the base level obtained on year 2014. The residual current speed profiles are shown in Figure 7.

Table 3. Current Speed of the study area

Year	Average Current Speed m/s	Average Residual m/s	Maximum Current Speedm/s	Maximum Residual m/s
2014	0.1 – 0.8	-	0.7 – 1.8	-
2020	0.15 – 0.85	+0.05	0.9 -1.8	-0.001 – +0.02
2060	0.2 – 0.88	+0.02 – +0.05	0.8 – 1.9	-0.06 – +0.05
2100	0.3 – 0.9	-0.01 – +0.18	0.6 – 1.9	-0.06 – +0.2

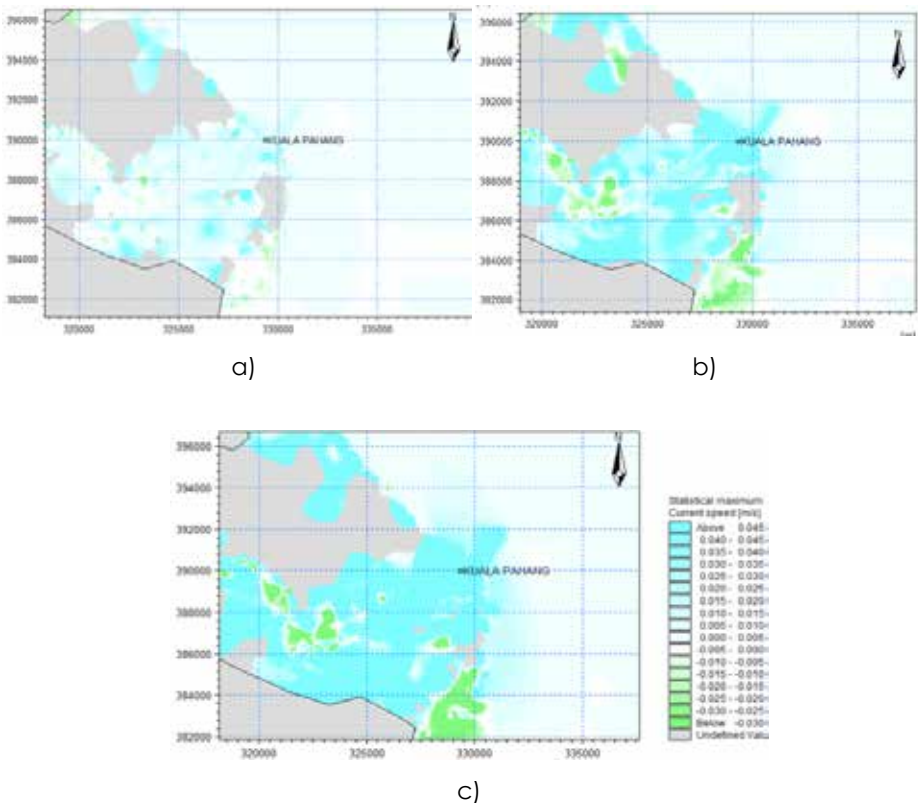


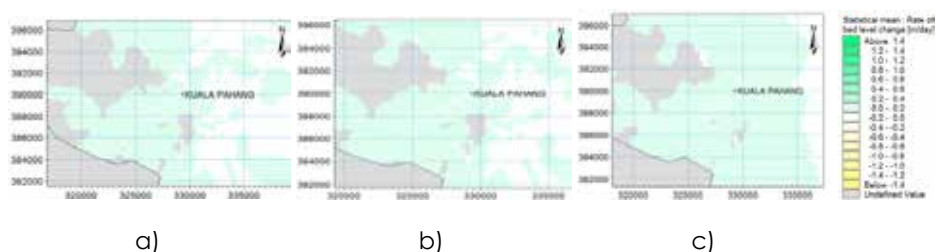
Figure 7. The residual maximum current speed for projection at year a) 2020, b) 2060 and c) 2100 at the Kuala Pahang estuary.

Analysis showed that the deposition took place in the area where reduction or decreasing current speed occurred due to obstacles such as at the island. Erosion occurred in the estuary and the edge of the river bank and left bank at meander river due to higher velocity. This area receives direct current and an increase in speed causing an increase in friction thus resulting in the increasing suspended sediment and bed load. It is speculated that this incident makes the region more vulnerable to erosion.

4.4 Sediment Transport Model

Sediment transport modeling for the year 2014, 2020, 2060 and 2100 shows the average rate of bed level change range between -0.03 - 0.3, 0.2, 0.4 and 0.6 m/day respectively while analysis for bed load for the respective years shows an average of 0.0002, 0.0003, 0.0004 and 0.0008 m³/s/m. The overview of the model output are tabulated in figure 8. The result shows with low sea level rise the average rate of bed level change is lower compared to high sea level rise. Increase rate from 0.2 to 0.6 m/dy seem to cause deposition around the Kuala Pahang estuary. With sea level rise, the area experience increase in rate of bed level change also increases. The average bed load are significantly low with low rise in sea level and albeit increase with higher sea level rise. The sediment transport analysis for the respective years is in table 4. The changes indicate small increment with sea level rise without the river discharge input. These finding shows that increase in sea level only give small magnitude to bed load.

It is revealed that rise in sea level will give impact to existing condition of the area. The result shows that the rate of bed level change in the sea water increases as sea level rise and causing deposition along the river. Low ability for the sediment to transport either through rolls, slides or bounces along the bottom of the waterway will promotes more deposition to occur.



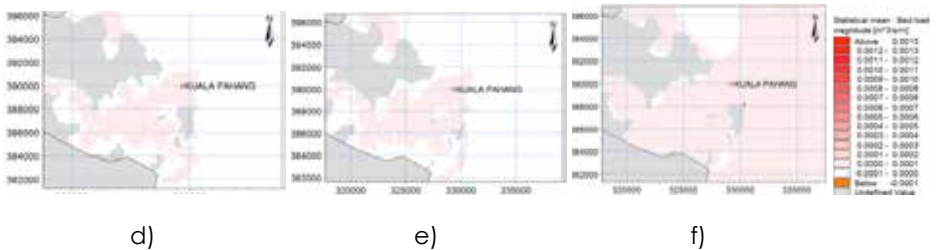


Figure 8. The average rate of bed level change for projection at year a) 2020, b) 2060, c) 2100 and average bedload for projection at year d) 2020, e) 2060 and f) 2100 the Kuala Pahang estuary.

Table 4. Sediment transport analysis

Sea Level Rise	No Sea Level Rise (2014)	Low Sea Level Rise (2020)	Moderate Sea Level Rise (2060)	High Sea Level Rise (2100)
	-	0.034 m	0.144	0.307 m
Average Rate of bed level change (m/ dy)	-0.03 – +0.3	+0.2	+0.4	+0.6
Average Bed load (m3/ s/m)	0.0002	0.0003	0.0004	0.0008

5 CONCLUSIONS

This study shows that rising sea levels obviously influenced the hydrodynamic pattern at the Kuala Pahang. Increasing sea level consequently produced faster current speed, in particular at the left bank of the estuary. This promotes sediment entrainment and increasing the erosion rate at this area. However, due to the complexity bed morphology of Kuala Pahang, where existing formed sand bars influenced the current speed to be much lower, in particular at the right bank of the estuary. Lower flow velocity promotes sediment deposition which subsequently a formation of new land may be expected. Therefore, it is concluded that rise in sea level will increase the water level at the river estuary due to backwater effect and changes in the hydrodynamic pattern change the place of sediment deposition which might lead to river flooding.

ACKNOWLEDGMENTS

The first author would like to express her gratitude to the Government of Malaysia for financial support during her Masters study. The authors are grateful to National Hydraulic Research Institute of Malaysia (NAHRIM) for their permission to use the data and information given on the study area.

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PHYSICAL HYDRAULIC MODELLING FOR THE DEVELOPMENT OF INNOVATIVE COASTAL PROTECTION STRUCTURE IN A 2-D WAVE FLUME

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ABSTRACT

NEXC Block is an innovative shore protection structure which is designed to protect the coast from severe erosion during monsoon season. While on a calmer period, the system encourages sediment accumulation thus expanding the beach naturally. NEXC Block is placed within Mean Higher High Water and Highest Astronomical Tide water level with sufficient toe protection for stability and efficiency, ensuring minimum adverse impact to existing coastal hydrodynamic. This paper aims to present the development process of NEXC Block in 2-D wave flume facility, where site measurement works are conducted for data validation and monitoring. Model dimension and hydrodynamic parameters are geometrically downscaled to 1:20. Two slope gradient values are used representing actual beach profiles at selected sites. Cohesive-form sediment size is 100 μ m. Three water levels; Mean Lower Low Water, Mean Sea Level and Mean Higher High Water with two significant wave height values H_s (irregular waves) are used for simulations. Beach profile is measured both before and after each simulation for erosion-accretion and stability analysis. Simulation on exposed beach without structure is also carried out as reference profile. The experiment shows that NEXC Block model structure installed with ample toe protection is a stable coastal protection structure. The structure is able to withstand strong wave impact during monsoon; no significant structural movement was detected. However scouring is prevalent at the toe of structure at unprotected structures. Some parts of NEXC Block and toe protection are exposed during monsoon season but less sediments are found eroded as compared to exposed unprotected beach; in which maintaining beach stability. Moreover during a longer calm period, the sediments are found to return back to the beach hence nourishing and expanding the beach naturally.

Keywords: Shore Protection, Hydraulic, Physical Model, Coastal Erosion, Natural Beach Accretion

1 INTRODUCTION

The main function of mostly available coastal protection structure is solely to protect shoreline from erosion. Some structures such as groin and offshore breakwater are more effective as those systems encourage sediment accumulation at leeside. However the installation of such systems are very costly and interfering with natural beach hydrodynamic processes. Wave breaking zone and the direction of longshore drift are directly affected hence creating adverse effects such as erosion to the other side of installation. Interference in this dynamic equilibrium leads to a change in sediment supply causing an increase or decrease in local sediment budget, resulted to accretion or erosion, respectively (Ghazali, 2006). In addition, the reflected wave energy back to offshore may cause beach profile changes due to resonant effects (US Army Engineers, 1984).

Seawall type of coastal protection can be considered as shoreline defense applications. Seawall is defined as an armoured revetment and an embankment with or without structural crest elements (Allsop, 1986). The protection wall is usually built parallel to the shoreline in order to prevent soil from sliding while offering protection from wave impact (Kamphuis, 2000). The construction material includes of concrete, rocks, steel, timber, rubber tyres, and sandbags (Pilkey, O. H.; Dixon, K. L., 1996). The shapes of coastal structures are determined by the use of the structure and generally can be categorised as vertical or nearly vertical, sloping, convex-curved, concave-curve, reentrant, or stepped. Shoreline defense structures, such as seawall and coastal blocks are situated beyond active hydrodynamic zone hence minimising adverse implication towards natural processes.

The aim of this study is to develop an innovative coastal block protection structure with minimum impact to existing coastal hydrodynamic processes. NEXC Block is designed and placed within Mean Higher High Water (MHHW) and Highest Astronomical Tide (HAT) water level to minimize the impact, and installed with sufficient toe protection for stability and efficiency. It protects the beach and acts as a wave breaker during monsoon season, while the system also encourage natural sediment accumulation on calmer period. NEXC Block, or NAHRIM Coastal Erosion Protection and Beach Expansion Block was developed by National Hydraulic Research Institute of Malaysia in 2015 – 2016. This applied research approach include conducting research and development using numerical models, experimental testing and pilot projects at selected sites.

Main objectives of 2D wave flume study include:

- a) To determine the ability of NEXC Block as a protection structure against coastal erosion,
- b) To assess the efficiency of NEXC Block as a coastal expansion mechanism in laboratory scale,
- c) To evaluate and verify the interaction between structures – water level – wave – sediment processes with NEXC Block site installations.

2 PROJECT SITES

Two locations are selected for the proposed NEXC Block pilot project development. The choices of sites are based on the differential exposures towards seasonal monsoon, morphological changes, hydrodynamic impacts and local socio-economic activities. Site A is located in Pantai Batu Lima, Port Dickson, Negeri Sembilan, within Regency Hotel compound which was severely eroded. The site is a sandy beach facing Malacca Strait, where wave impact is dominant during Southwest Monsoon (April – July). The bay is a popular attraction for tourism and water sport activities throughout the year.

Whereas Site B is situated at the east coast of Peninsular Malaysia, facing South China Sea. The shoreline are exposed to heavier wave impact during Northeast monsoon (November – February). The sandy beach of Pantai Rhu Muda, Marang, Terengganu is a traditional village community where fishing and small scale fishing related-industry are the main socio-economic activities. The straight coast off Marang is usually exposed and severely eroded during monsoon but eventually stable and naturally nourished during calmer period.



Figure 1. Site locations in Negeri Sembilan and Terengganu, Malaysia

Primary hydraulic data collection works have been conducted at both locations. Water level is measured with reference to the nearest tidal station at each site. Current and wave data are measured at 10m water depth using two ADCP units, approximately 3km from shoreline for a minimum 15 days. Bathymetry survey was done within 3km x 3km area, including sediment sea floor sampling and water quality monitoring. Beach profiling was done on monthly basis within few days after full moon event. Secondary historical wave and wind information are gathered from UKMO database. GIS survey was also conducted to verify existing physical, socio-economy and natural ecosystem at the selected sites. The data and information gathered are later analysed and used for numerical and experimental studies.

3 EXPERIMENTAL SETUP

Physical tests were conducted using a 50m - 2D wave flume facility located at Hydraulic and Instrumentation Lab, National Hydraulic Research Institute of Malaysia (NAHRIM). Model structure dimension and hydrodynamic parameters are geometrically downscaled to 1:20. Mildly sloping shore 1:15 and steep slope 1:3 are used representing actual beach profiles at selected sites. Cohesive-form sediment size is 100 μ m. Three water levels; Mean Lower Low Water (MLLW) 0.9m, Mean Sea Level (MSL) 1.0m and Mean Higher High Water (MHHW) 1.1m with two significant wave height values H_s (irregular waves) 0.1m and 0.2m are used for simulations. During the study, model structure is placed at a fix location along the flume, approximately at the centre of interest study area i.e. in active erosion / accretion zone. Beach profile is measured in the area of interest of 2.5m x 1.5m both before and after each simulation for erosion-accretion and stability analysis. Simulation run-time is 60 minutes referring to the duration of specific water level in each tide cycle. Simulation on empty unprotected beach without structure is also carried out as reference profile. Wave height and period is calibrated both manually and automatically using 3 units of wave probe located along the flume.

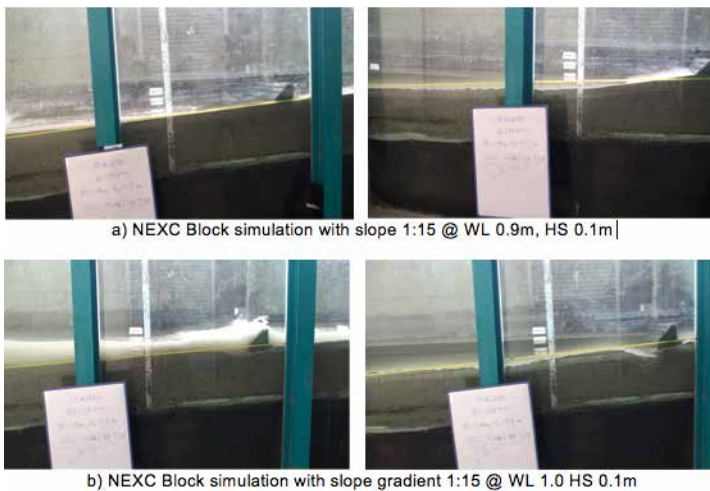


Figure 2. Experimental work in 2D wave flume facility

Interaction between waves and waves-structures due to variation of water levels and wave heights are studied and recorded. The scope of simulation works is simplified as below:

- Study on beach profile changes on accretion or erosion due to wave actions and sea levels on an exposed / unprotected beach (No Structure) as a control,
- Study on beach profile changes on accretion or erosion due to wave actions and sea levels with NEXC Block coastal protection structure model ,

In flume, the interest study area is marked as 0cm – 250cm with 10cm interval in Y direction (length of flume). Whilst in X direction the profile is marked from 0cm to 150cm (breadth of flume). Data sample for the first 30cm from both side walls in the flume is ignored and considered as unstable due to wave reflection effect. A datum or reference point is fixed at the top corner of flume wall. The distance between datum and beach surface is remarked as height in Z-axis at specific points. NEXC Block system is placed fixed along coordinate 0 – 150, 90 – 120 with single layer installation. Beach profile is measured manually each time before and after simulation to compare the differences, associated to erosion or accretion. In this paper, all erosion level is measured at the central of wave flume (x-axis) along the beach profile (y-axis) which is at cross-section 70, 30 – 160 (x, y). Figure 2 showed the experimental work conducted in wave flume with various hydrodynamic parameters on a stiff and mild slopes environment. Simulation equilibrium was set not more than 60 minutes.

Stability of coastal block as a coastal protection structure

The proposed coastal block aims to function as a natural solution to beach erosion and renourishment. Based on simple conservative economics, the structure can be used on either semi-permanent to permanent basis, contingent on the scope of the renourishment project. Coastal blocks work in two ways; firstly by breaking down the wave energy, therefore lessening erosion impact to the shore. Secondly, it encourages beach expansion or natural nourishment by permitting sea water which contains sediment and sand to pass through the tapered hole. During ebb flow, sediments are trapped behind the structure and unable to return to the sea. Over the time especially during a calm period, sediments are accumulated hence expanding the beach naturally. Hence the design of coastal block must be sufficiently stable against overturning and sliding failures. The calculation of wave forces on coastal block is according to EM 1110-2-1100 (Part VI), Goda formula for irregular waves (Goda, 1974; Tanimoto et al., 1976).

Overturning failure is rotation of wall about its toe due to exceeding of moment caused due overturning forces to resisting forces. Eq. [1] show the factor of safety against overturning is given by:

$$F_0 = \frac{\sum M_R}{\sum M_0} > 1.5 \quad [1]$$

where,

$\sum M_R$ = sum of resisting moment about toe
 $\sum M_0$ = sum of overturning moment about toe

The factor of safety sliding may be expressed by Eq. [2] below, which generally a minimum factor of safety of 1.5 is required:

$$FS = \frac{\sum R_V \tan \delta + kc_2 B}{\sum R_H} \quad [2]$$

where,

$\sum R_V$ = sum of the vertical resisting forces

$\sum R_H$ = sum of the horizontal resisting forces

δ = angle of friction between the soil and base slab k = the range between 1/3 to 2/3

c = cohesion of soil R = resistance force B = bottom length of structure

Table 1. Overturning check on coastal block structure at point 0

Option	Θ	Moment at 0		$\frac{\sum M +}{\sum M -}$	Remark
		$\sum M +$ (kNm)	$\sum M -$ (kNm)		
1	30°	58.2	18.88	3.1	Ok
2	45°	18.1	7.58	2.4	Ok
3	60°	8.74	7.64	1.14	No (redesign 3a)
3a	60°	14.71	7.64	1.9	Ok

Table 2. Sliding check on coastal block structure at point 0

Angle (°)	ΣRV (kN)	ΣRH (kN)	B (m)	FS	Remark
30	68	18.45	1.85	3.7	Ok
45	40.2	15.36	1.2	2.8	Ok
60	31.75	12.34	1.05	2.9	Ok

The result of analysis is given as above. Table 1 showed the moment acting on coastal block for stability analysis on overturning check. The factor of safety for overturning is more than 1.5. The force acting on coastal block for stability analysis on sliding check are summarised as shown in Table 2. The stability of coastal block is analysed using full scale data collected from site and secondary data where necessary.

The structure is exposed to both forces from the wave action and lateral earth action specifically active earth pressure. As the block moves away from the backfill, there is a decrease in the pressure on the wall. The decrease continues until a minimum value is reach after which there is no reduction in the

pressure and the value will become constant. Coastal block must be designed to withstand wave actions in order to maintain its stability against overturning and sliding failures. Vertical distribution of pressure due to wave force acting on sea block is measured based on wave velocity and state (peak or trough) and water level (Hanbin GU et al., 2003).

There are three available designs of NEXC Block with different face angles; namely 30°, 45° and 60°. Due to technical, engineering and financial justification, the 45° angle is selected and suited in the physical experiment and site installations. Actual dimension of NEXC Block is 1.2m x 1.2m x 1.2m and each unit weighs approximately 2,500kg, made of reinforced concrete grade 40 which is suitable with marine environment. A laboratory scale model weighs about 5kg each and was found sufficiently strong and stable in providing protection against a downscaled wave and water level impacts in 2D wave flume simulations.

4 DISCUSSION ON LABORATORY DATA SAMPLING AND ANALYSIS

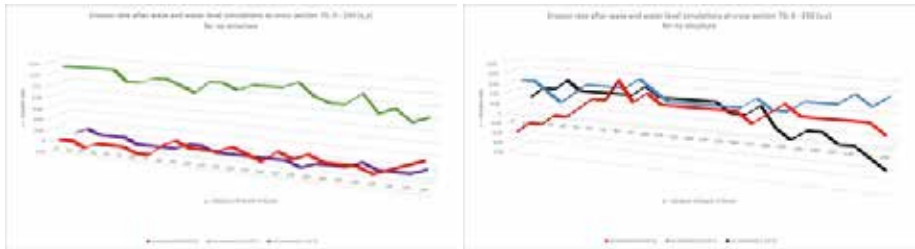
Figure 3 depicted the beach profiling on erosion-accretion analysis of selected simulations. No structure simulations as presented in Figure 3a shows that erosion occurred along the unprotected beach. Higher water level led to erosion farther landward. Higher wave magnitude caused greater erosion level or scarp height. Combination of high water and large wave magnitude caused catastrophic impact; total sand loss along the exposed shore. Rate of erosion is quicker and higher at a steeper slope and higher water levels conditions, until it reaches equilibrium. Data and information from this simulation is used as control and reference to the simulations with proposed coastal structure.

Figure 3b illustrated the erosion-accretion profile after simulations with NEXC Block are completed. It is found that with NEXC Block installation, erosion is controlled to occur only at the face of the structure, thus the backside of system is protected. Erosion was generally occurred at all water level scenarios MLLW, MSL and MHHW with H_s 0.2m. The protected beach at the backside of installation was found safe with the installation of NEXC Block. The system were also found stable against overturning and sliding failures.

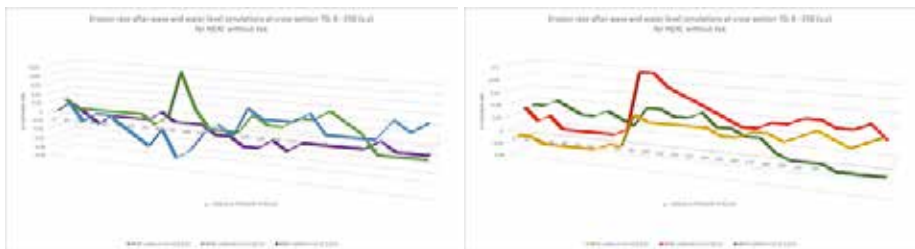
It is also found that lower water level with lower wave magnitude encouraged sediment accumulation at the structure face. This was found in simulation with MLLW 0.9m and H_s 0.1m. In a longer calm period or outside monsoon season, the accumulation process is repeated until the sediments are returned to the beach and trapped behind the structure installation, resulted of a natural beach nourishment.

Even though erosion is controlled to occur only at the face of the structure, this has led to scouring problem at the toe of structure. Scour is found maximum when water level is at the structure level; within wave breaking zone. Wave impact on structure caused wave overtopping and sediment deposition near structure toe. Higher water level which submerging the structure minimized the impact of toe scouring. However due to higher water level reaching farther landward, it left the area unprotected hence erosion is found more landward. Nevertheless the structure managed to block and control the sediment from

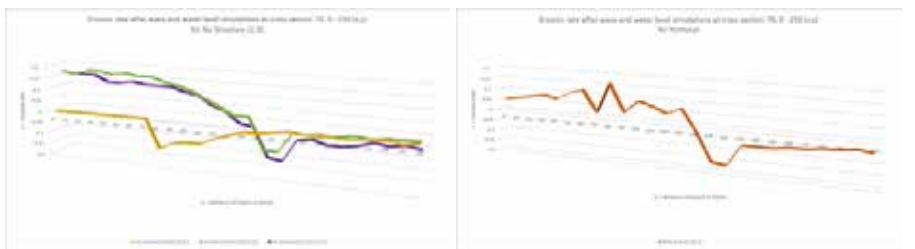
moving farther seaward from land.



a) Beach profiles at unprotected shores after simulations completed (control)



b) Beach profile changes with NEXC Block protection



c) Beach profiles with NEXC Block plus scour protection

Figure 3. Analysis of erosion-accretion after simulations completed

The simulations are also expanded by introducing scour protection installed at the toe structure to prevent structural failure. From Figure 3c, the analysis showed that sufficient toe protection had increased the stability of NEXC Block installation. Scour at toe structure can be minimized and naturally transferred at the bottom of the scour protector units. Scour protection units can be installed further seaward beyond breaking zone hence minimizing the impact of scour problem. However an increase in overtopping is expected with this type of installation due to the increase of height breaker.

Hollow slanting cone shape design at NEXC Block is the main feature that allows sea water to penetrate through structure during flood and ebb flows.

Suspended sediments from offshore are carried by water and wave, then penetrated the structure via hollow sections and trapped behind the installation as the water flows back out. Hollow sections on the structure must be designed less than 50% of surface area to ensure optimum sediment entrapment. The design was also able to distribute wave energy evenly during high tide, reducing overtopping and scouring impact.

5 ON SITE MEASUREMENT

Data collection works and beach profile measurement at sites are shown in Figure 4a. The measurements are generally done on monthly basis to monitor continuous beach profile changes throughout the beach cycle period in one year calendar. Measurement is done automatically using Real Time Kinetic device, however in case of no signal or connectivity issues, auto level is used. In Port Dickson, NEXC Block system is installed within the Mean Sea Level to Mean High Higher Water marks. Changes in beach profile in front of the structure is greatly influenced by the high tide event every month. Scour protection as in Figure 4a is exposed during full moon and high tide events but the area is usually recovered during neap tide. The process is relatively typical for each tidal cycle.



a) Beach profiling at Marang Beach

b) Natural beach nourished after monsoon

Figure 4. Beach profile measurement at sites

However in Marang Beach, Terengganu the shore is only exposed to extreme event during northeast monsoon season. The installation of NEXC Block, located within Mean High Higher Water to Highest Astronomical Tide marks successfully control and limit the erosion at the face of the installation, whereas the backside area are protected. During northeast monsoon, the beach was starting to erode and scour impact can be seen as in Figure 4a. However NEXC Block has managed to block sediment from reaching further seaward. Sediments are trapped right at the back of the structure hence expanding the beach naturally. Coastal vegetation are also found to start recovering and regrow as in Figure

4b after monsoon season ended. Figure 5 showed the analysis carried out to investigate the changes of profile at sites due to NEXC Block installation. Based on the analysis it is proven that water level, wave magnitude and beach profile are the main parameters in determining scour impact at the toe of structure, as per results in the study in 2D wave flume laboratory.

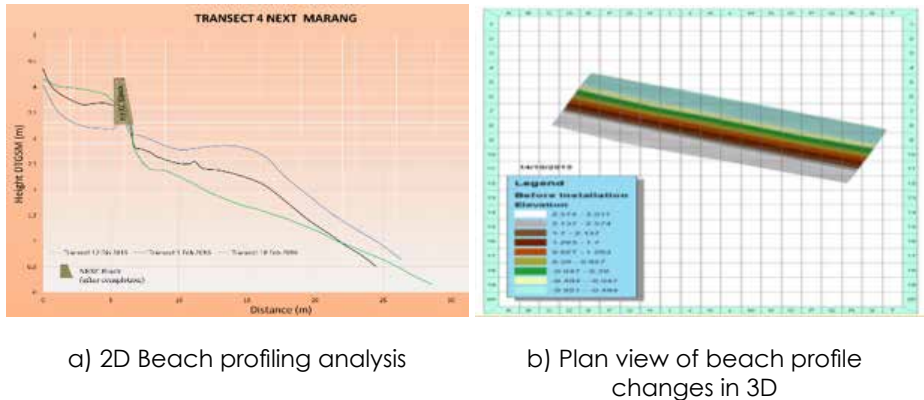


Figure 5. Analysis of beach profile changes.

6 CONCLUSIONS

In general, the rate of erosion is dependent of wave magnitude and water level. Higher wave breaks earlier and farther from the beach as compared to lower wave. However due to wave energy by higher wave, even after breaking, the wave propagates further inland with higher volume. Thus higher wave causes more erosion as compared to lower wave height, as same goes to water level parameter. As the water level increases, due to less bathymetry effect, the breaking point also moves to a point nearer to the beach thus leads to higher erosion rate. The experiment thus proves that the worst erosion occurs during the combination of high water level and high wave magnitude.

NEXC Block installation is found successfully controlled the erosion within the interest area. Sand loss due to erosion and scour is limited to only at the face of the structure, whereas the backside of installation is found stable. However during high water level and high wave event, scouring problem is prevalent. Hence scour protection units is proposed to be combined with NEXC Block installation. In a long term period, sediments are returned back to the beach hence naturally expanding the beach. Result of simulations in laboratory works in 2D wave flume has been validated using actual site measurement. Hence physical experiment approach is still considered as one of the best approaches in conducting study within coastal zones. For future work study, focus can be given to improve the efficiency of scour protection. It is also suggested that the material of NEXC Block is transformed into alternative environmental friendly materials.



a. NEXC Block installation in Marang, Terengganu



b. NEXC Block structure in Port Dickson, Negeri Sembilan

Figure 6. NEXC Block installation at sites

ACKNOWLEDGMENTS

All staff of Coastal and Oceanography Research Centre and Hydraulic and Instrumentation Laboratory, and National Hydraulic Research Institute of Malaysia is thanked for their assistance and contribution in conducting the experiment. Special acknowledgment also for the research team from Faculty of Civil Engineering, Universiti Teknologi Malaysia for their support. The Government of Malaysia is also thanked for the approval of research grant and related studies. NEXC Block has successfully accepted for registration approval in Industrial Design category MY16000690101 from MyIPO on January 2016.

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PHYSICAL MODELLING TESTING FOR RAM PUMP PERFORMANCE

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ABSTRACT

Provision of adequate domestic water supply for the scattered rural population is a major problem in many developing countries including Malaysia. Operation for the common pumping system had become an issue of water supply to rural and remote areas due to a few reasons such as power supply shortage, inaccessible infrastructure and new settlement. In order to minimize the problem, a study had been conducted in NAHRIM's laboratory using ram pump technology. The aim of the study is to identify the performance of ram pump for achieving sustained performance at optimum operational in laboratory scale. The testing component consists of the ram pump and inverter controlled pumping system with a series of pipelines. A total of 2 testing conditions had been conducted during the study, which are dam simulation and river simulation with 6 testing scenarios. It was found that the ram pump could transfer 10% of total volume of water sources from any open channel using kinetic energy only. The result of this study also found that the ram pump has a potential in supplying water for rural and remote settlement area especially near the river.

Keywords: Gravity Flow, Ram Pump, Water Resources, Testing Conditions, Kinetic Energy.

1 INTRODUCTION

A ram pump is a hydraulic pump that uses energy from a falling quantity of water to pump some of it to an elevation much higher than the original level at the source. No other energy is required as long as there is a steady flow, it will work continuously and automatically (De Carvalho et al., 2011; Filipan and Bergant, 2003; Herlambang and Wahjono, 2006). Provision of adequate domestic water supply for the scattered rural population is a major problem in many developing countries. Fuel and maintenance costs to operate conventional pumping system are becoming prohibitive. The ram pump is an alternative pumping device that is relatively simple technology that uses renewable energy and is durable. Ram pump has been used for over two centuries in many parts of the world (Suarda and Wirawan, 2008; Tessema, 2000; Inthachot et al., 2015). Their simplicity and

reliability made them commercially successful, particularly in certain Europe countries, in the days before automation technology become widely available (Maratos, 2003). As technology advanced and become increasingly reliant on sources of power derived from fossil fuels, the ram pump was neglected. Big had become beautiful and small-scale ram pump technology was unfashionable (Twort et al., 2000). In the United Kingdom, manually controlled precursor of the ram pump had been invented (Sakenian Dehkordi and Arshad, 2012; Shuaibu Ndache Mohammed, 2007; Atharva Pathak et al., 2016). Although hydraulic ram pump come in a variety of shapes and sizes, they all have the same basic component states of a ram pump are the main vessel, waste valve, delivery valve, snifter valve, air chamber and relief valve (Matthias Inthachot et al., 2015; Wallace and Warren, 1941). Ram pump's cycle had three phases; acceleration, delivery and recoil. A cyclic pumping action will produces characteristics beat during operation. The aim of the study is to identify the performance of ram pump for achieving sustained performance at optimum operational in laboratory scale. As to meet the objectives above, the scope of works include of: testing the performance of ram pump system with capacity of 40 meters a total maximum head and 30m³ volume water per day (equal to supplying water to 10 houses model); undertaking full scale test on the ram pump system inside NAHRIM's Hydraulic Instrumentation Laboratory; testing ram pump supply by river flow simulation; testing ram pump supply by reservoir/ lakes/ dam flow simulation and observing breakdown during testing for maintenance input.

2 METHODOLOGY

The setup component in the project are: a set of supply pump with inverter system, a unit of ram pump , water tanks (as water supply and receiving tank), a set of PVC pipeline system, flowmeters, valves and joints, structural component such as pump plinth and tank platform and some civil worked.

Inverter pump was used to supply water to the ram pump at certain flow rate. The flow rate was measured by two units of factory calibrated flow meter that located before and after the ram pump. The most common concept of surface water flow is cause by two types of sources, that is river water flow sources; and dam water flow sources from upstream to downstream. Based on the inconsistency flow rates, the ram pump testing was carried out in several scenarios. There will be two types of flow, which are (i) river flow simulation (by M1, By Passed using Inverter system); and (ii) dam simulation (by M2, Static Tank Method as illustrated in Figure 1 as below. There are six (6) testing scenarios during the testing period, which are pre-commissioning testing and pre-dam simulation testing, dam simulation testing, pre-river simulation testing, river simulation testing, the 24hrs simulation testing and the free flow simulation testing.

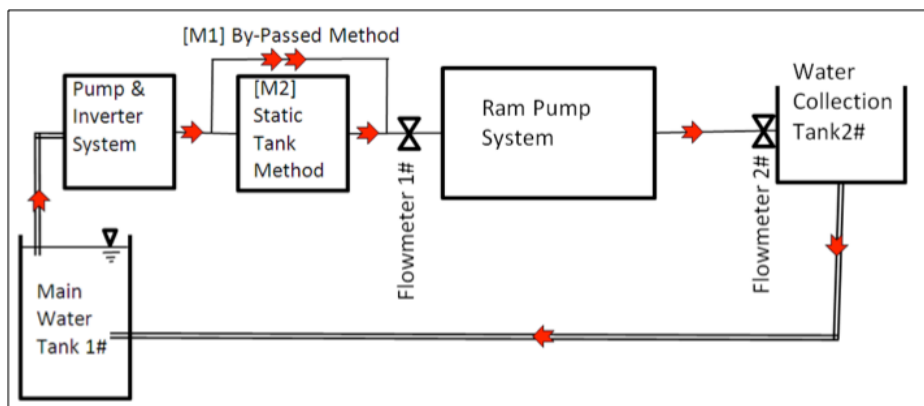


Figure 1. Schematic Diagram For Ram Pump Performance Test

2.1 Pre-Commissioning Testing

The Pre-Commissioning Testing purposes were to make sure the system operate smoothly. No testing data were recorded during the test run. This test served as a calibration for ram pump. All ram pump system components were assembled accordingly to design in order to have stable ram pump performance. Valves were controlled to assure optimum water supplied to the ram pump from dam tank.

Supplying water to ram pump 0 to 10 m³/hour for about 5-10minutes. Observe the ram pump optimum operation. Increase the flow rate if the ram pump does not operate, in interval of 10 m³/hr from 10 up to 50m³/hr. Close the valve after the ram pump works continuously. Try to start the ram pump by forcing up and down the choked for a few minutes; the ram pump was started to squirt the water. At certain forcing, ram pump choked automatically moving up and down. Stop forcing at when the ram pump squirting at constant flow. Increase inverter pump flow rate if the choked does not move up and down automatically. At this stage, start to monitor the pressure gauge at the top of the ram pump vessel. The pressure was increased its value from 0 to 1.5 bars (30psi). When the pressure had reached 1.5 bars, ram pump was started to squirt the water consistently. Slowly opening valve after ram pumps as to let the water, sending from ram pump to receiving tank. Remarks water sent recorded on flow meter after ram pump. From this moment, the ram pump is ready for further scenarios testing. After the ram pump had performed its pressure in a stable range of 5 - 10 minutes of each inverter pump flow rate setting, increase inverter setting in order to find out which setting make the ram pump shut down (unperformed), consistently performed and strongly performed. Now, the ram pump had performed in consistent inverter pump setting and ready for further testings. Let the scenario run for few hours continuously as to oversee any breakdown occurs.

2.2 Pre-Dam Simulation Testing

After the Pre-Commissioning Testing, ram pump was ready to be tested in further scenarios. This testing is purposely to find out the exact setting of inverter pump flow rate as to determine minimum workable flow rate for selected size of ram pump. Firstly, operate ram pump in steady performance at inverter pump setting begin at 39 Hertz. Then, record ram pump performance every hour until eight (8) hours continuously and also record water level decreasing at Dam Tank. Start again the ram pump after the ram pump stop operating. This time decrease it from 39 Hertz to 38 Hertz, repeat until at 30 Hertz. By this testing, ram pump designed factor at application site could be determined in terms of annual flow rate of the water body or by the necessity of having small size of water dam.

2.3 Dam Simulation Testing Methodology

This test is to evaluate whether the ram pump required a small dam for the effective and continuous operations during on-site application. During this testing, Inverter Pump was setting in interval of one (1) between 33 to 39 Hertz for this supplying water to the Dam Tank. There was a different level of water detention at Dam Tank. Firstly, operate the ram pump as usual; start at Inverter Pump setting at 39Hertz. Observe the water level of Dam Tank after few hours ram pump operating continuously. Repeat the test with Inverter Pump setting at 38 Hertz to 37.8 Hertz and observed the water level of the Dam Tank;

2.4 Pre-River Simulation Testing Methodology

This test is to confirm that ram pump could be operated without the assistant of Dam Tank. There are four (4) adjustable valves to control the flow to the ram pump to achieve a constant flow rate. Firstly, set the Inverter Pump constant flow rate at 33 Hertz. Then, adjust Valves A, B, C, D as follows: Fully Closed D-Valve, Fully Open A-Valves, $\frac{1}{4}$ Opened B-Valves and $\frac{1}{2}$ Opened C-Valves. After that, repeat Inverter Pump setting at more than 33 Hertz, followed by the setting, at less than 33 Hertz.

2.5 River Simulation Testing Methodology

This test is to confirm that ram pump could be operated without the assistant of Dam Tank at a certain setting of Inverter Pump. Firstly, set Inverter Pump constant flow rate at 43Hertz. Then adjust Valves A, B, C, D as to assure only one way flow to the ram pump produce. Repeat at Inverter Pump setting at more than 43 Hertz to 50 Hertz.

2.6 The 24 hours Simulation TestingMethodology

This is conducted to test whether the designed ram pump could operate continuously exceeding 6hours, 12hours and 24hours. This test was executed after the River Simulation Testing had completed.

2.7 Free Flow Simulation Testing Methodology

This test is to determine flow rate of Inverter Pump Setting to support the assembly factors of ram pump at the site. From this testing, correlation between Inverter Pump Setting and flow rate was produced as references to be used for application. The result was recorded only after the flow rate is stable. Based on standard operating procedure of current meter in-situ measurement and its tolerant factors, the test period had been selected for 300 second (5 minutes) of each testing.

3 RESULT AND DISCUSSION

A total of 17 test had successfully had been carried out as shown in Table 2. Some of the constraints during the experiments are: (i) modification of pre-commissioning test due to unstable pressure at the receiver tank; and (ii) rectification of inverter pump due to damage to the rubber bush coupling. A total of six (6) test scenarios was carried out during the testing period.

Table 2. Successful Test Recorded

Months	No. Of Test
May	9
June	3
July	5
Total	17

The Pre-Commissioning Testing and Pre-Dam Simulation Testing were to make sure the system operate smoothly, and no data were recorded for the test. Data were recorded for the rest of the testing scenarios for analysis. Table 3 showed the testing period for different testing scenarios.

Table 3. Testing Period According To Testing Scenario.

No.	Scenarios	Testing Period (Day)	Type of Data
i.	Pre-Commissioning Testing and Pre-Dam Simulation Testing	7	Visual Observation
ii.	Dam Simulation Testing	6	Flow Data
iii.	Pre-River Simulation Testing	4	Visual Observation
iv.	River Simulation Testing	6	Flow Data
v.	24hrs Simulation Testing	1	Flow Data
vi.	Free Flow Simulation Testing	1	Flow Data
Totals		25	

3.1 Result of Pre-Commissioning Testing

After the ram pump system had been set up, pre-commissioning test was run to determine ram pump steady operation. No flow data were recorded for this testing, but observations were recorded.

3.2 Pre-Dam Simulation Testing

Pre-commissioning testing has been successfully conducted as pre-requisite for further test. After the pre-commissioning testing, pre-dam simulation testing was run as to determine ram pump operation stability. Some of observation and flow were recorded as Table 4.

Table 4. Pre-Dam Simulation Testing At Different Setting of Inverter Pump.

Inverter Pump Setting (Hertz)	Impact of Water Level At Dam Tank	Ram Pump Operation Status
30.0	Decreased	Not Operating
30.5	Decreased	Not Operating
31.0	Decreased	Not Operating
31.5	Decreased	Not Operating
32.0	Decreased	Not Operating
32.5	Decreased	Not Operating
33.0	Decreased	Not Operating
33.5	Decreased	Not Operating
34.0	Decreased	Not Operating
34.5	Decreased	Not Operating
35.0	Decreased	Not Operating
35.5	Decreased	Not Operating
36.0	Decreased	Not Operating
36.5	Decreased	Not Operating
37.0	Decreased	Not Operating
37.5	Decreased	Not Operating
37.8	Constant	In Operation
38.00	Constant	In Operation
39.00	Constant	In Operation

3.3 Result of Dam Simulation Testing

From the testing, it was found that certain setting of Inverter Pump will cause to decrease its water level and subsequently interrupting the ram pump performance. However, at certain setting from 36 to 37Hertz, the ram pump operated intermittently for sometimes. These could be as an early warning sign of ram pump system failure and it show that the ram pump requires a certain amount of detention water above the ram pump to give constant water pressure to the ram pump. Results of dam simulation testing are shown in Table 5. This testing scenario had proved that ram pump requires a small size of retention water structure or mini dam, with a minimum capacity of 2,271 liter for continuous ram pump operation.

Table 5. Summary Result of Dam Simulation Testing.

Inverter Pump Setting Flowrate (Hertz)	Ram Pump Operation Status	Flowrate (m3/hr)
33.0	Not Operating	No Data
33.5	Not Operating	No Data
34.0	Not Operating	No Data
34.5	Not Operating	No Data
35.0	Not Operating	No Data
35.5	Not Operating	No Data
36.0	Intermittent Operation	151.16
36.5	Intermittent Operation	151.16
37.0	Intermittent Operation	161.7
37.8	In Operation	151.9
38.0	In Operation	137.8
39.0	In Operation	137.0

3.4 Result of Pre-River Simulation Testing

Generally, form this testing scenario; it is possible for a ram pump to operate without the assistance of retention water. However, it requires some complicated valves controlling procedures. Pressure gauge at ram pump does not give high reading, meaning that no additional pressure at ram pump during high and low peak of flows. However, by the assistance of two valve's setting, Valves C and Valves D, had given the possibility for the ram pump to continuously operate.

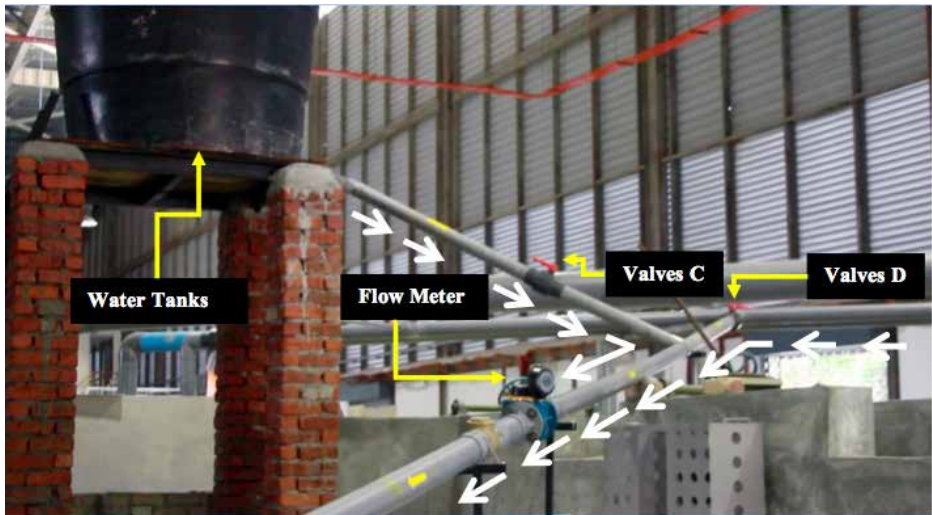


Figure 3. Water Flows and Valves Setting During the Ram Pump Testing.

3.5 Result of River Simulation Testing

As the result of Pre-River Simulation Testing, some flow rate setting on Inverter Pump were found as stable testing methodologies. In this test, Inverter Pump had been set to six (6) setting which generate incoming water to ram pump from 34.4 to 40m³/hr(50 Hertz). The summary of testing result is as shown in Table 6.

Table 6. Summary of River Simulation Testing

No.	Inverter Pump Setting Flowrate (m ³ /hr)	Average Incoming Water Supply to Ram Pump Flowrate (liter/second)	Average Delivery Water Supply from Ram Pump Flowrate (liter/second)	Percentage Differences between Incoming and Delivery
1	34.4	2.022	0.079	3.89%
2	36.0	2.007	0.144	7.17%
3	37.6	1.933	0.103	5.30%
4	38.4	1.880	0.065	3.47%
5	39.2	1.852	0.072	3.87%
6	40.0	1.784	0.096	5.40%

At the setting from 36m³/hr till 38.4m³/hr of Inverter Pump setting, the average of incoming water supply to and from ram pump and from become deteriorated. From the Table 6, at the highest setting, 40 m³/hr and at the lowest setting, 34.4 m³/hr of Inverter Pump, the ram pump delivery also become deteriorated. This leads to the conclusion that optimum water flow rate to the ram pump would be a requirement. However, due to the river flows fluctuation, the ram pump is recommended to have water retention structure in order to ensure continuous performance.

3.6 Result of 24 Hours Simulation Testing

The ram pump could operate for more than 6 hours, 12 hours and 24 hours continuously. It also found that no breakdown occurs after 24 hours. No heat development that could lead to component ware and tare. Ram pump had successfully supplied 11.647 (~12m³) per day or 7.03% of incoming water. Conversion of the supplied water is: 12m³/day or 0.5m³/hours or 0.0083m³/minutes or 0.0001388m³/second or 0.1388liter/second (~0.14liter/second). As recorded by the pressure vessel of ram pump that the pressure decrease by 20% at the end of the testing from 10psi to 8psi, and the ram pump still could operate. In this test, 165 m³ (1.91 liter/second) had been supplied to ram pump delivering 0.14 liter/second. From these findings, it can be stated that flow rate ratio for at site (laboratory or river) and ram pump are 1.91:0.14 liter/second resulting in the ram pump deliverables at 7% water from the site. This means that, if the ram pump located near to the river of 1.91 liter/s, the ram pump require 4.5 hours to fill in 2,270 liter seized of water tank. From these findings, the numbers of selected sized ram pump could be determined by calculation. Tables 7 are the calculation for determining numbers of ram pump for certain residential usage.

Table 7. Ram Pump Calculation Determining Numbers of Ram Pump for Certain Residential Usage

Conditions/ Formula/ Input	Output
Resident Population/ Water Demands or Water Supplied	1000m ³ for 100 people
Water should be received by ram pump according to ratio formulation (1:10).	10,000m ³
Adjustment of Water Supply In Time of 12 hours or 720 minutes or 43,200seconds (Proposed 7.00PM to 7.00AM)	The River/ Dam Flow Rate Requirement: = 1000 m ³ / 12 hours or 720 minutes or 43,200seconds = 83m ³ /hour or 1.39 m ³ /minutes or 0.023 m ³ /s or 23 liter/ second.

No. of ram pump required based on early finding.

[Early finding: 5hours at 7.5liter/sec-
ond supplying 145m3 for a unit of ram
pump at 1.5Bar or 30psi]

Option 1: Three (3) unit of ram pump
at 1.5 Bar or 30psi

Option 2: One (1) unit ram pump with
achievement at 3.0Bar or 60psi

3.6 Result of Free Flow Simulation Testing

In this testing, the testing ranges of the Inverter Pump are from 36 Hertz to maximum 50 Hertz. From the testing, due to the distance from Inverter Pump to flow meter (before Ram Pump), the amount of the losses is about 27 - 34%. However, there are only small reading differences between flow meter reading and Inverter Pump manufacturer calculation. As such, this free flow testing result was used as references for acceptable tolerance between Inverter Pump Setting and real flow meter measurement. The summaries of free flow testing result are as shown in Table 8.

Table 8. Summary of Free Flow Simulation Testing.

Setting of Inverter Pump (Hertz)	Volume (liter)	Test Period (second)	Flowrate Recorded (liter/sec- ond)	Flowrate Calculated (liter/sec- ond)	Percent- age Differ- ences
36	1703	300	5.7	8.0	29%
37	1756	300	5.9	8.2	28%
38	1819	300	6.1	8.4	27%
39	1843	300	6.1	8.7	30%
40	1858	300	6.2	8.9	30%
41	1900	300	6.3	9.1	31%
42	2027	300	6.8	9.3	27%
43	1966	300	6.6	9.5	31%
44	1956	300	6.5	9.8	34%
45	2009	300	6.7	10.0	33%
46	2032	300	6.8	10.2	33%
47	2099	300	7.0	10.4	33%
48	2115	300	7.1	10.7	34%
49	2199	300	7.3	10.9	33%
50	2201	300	7.3	11.1	34%

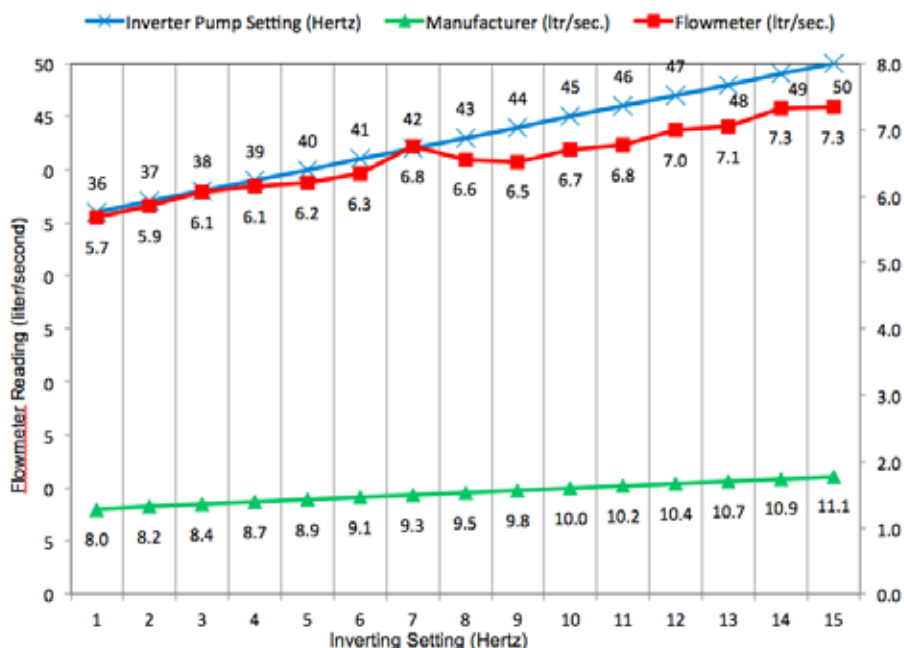


Figure 4. The Finding of Inverter Pump Setting and Flowrate Measurement.

4 CONCLUSIONS

This project had successfully evaluate and determine the ram pump performance. At the laboratory scale, it was confirmed that ram pump had its potential to provide an alternative means of water supply delivery system through not as efficient as conventional supply. For the study, we had found the Ram Pump Delivery Ratio (RPDR).

This study had met performance tested of ram pump. Results show that ram pump will deliver 10% of the incoming water injected to the ram pump to the end user. Water will be supplied by a ram pump in the ratio of 1:10. For example, if 10,000m³ go through the ram pump, only 1,000m³ of water will be the supply volume. This study also had to meet objectives of identifying optimum sustained water flow from ram pump through simplified calculation in Ram Pump Design Calculator (RPDC). The RPDC had been produced based on 1:10 ratio to determine the ram pump size, quantity and requirement according to water supply demand. Table 9 shows the RPDC.

From the test carried out, it was found as follows: (a) the ram pump could perform consistent with the mini dam concept; (b) factors affecting the ram pump performance are the consistency of water sources to start-up the ram pump. Although the ram pump does not use electricity, some additional accessories in controlling ram pump performance are recommended to be

installed, to assure its continued operation. It is recommended to be integrated with surveillance unit, semi or fully automated units for controlling purposes. This should be considered due to the engineering knowledge constraint of rural communities. Standard Operating Procedure documents should be prepared and deliver to the community as basic information to ram pump start up, monitoring and troubleshoots.

ACKNOWLEDGMENTS

The authors would like to thank the top management of National Hydraulic Research Institute of Malaysia (NAHRIM) for the research project's financing. Special thanks to NAHRIM's Research Assistances, Jegajeewan Naidu Balasundram, Zairi Jalaludin, Hafizul Husni Mohamed Sidin, Suriani Othman, Safarudin Salehuddin, Fardzir Johari involvement in this project.

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BIG DATA TECHNOLOGY IMPLEMENTATION IN MANAGING WATER ISSUES: NAHRIM'S EXPERIENCE

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ABSTRACT

Information explosion and data processing related to the water and environment have evolved in parallel with the development of data collection technologies that is done either automatically or manually. Water and environment data received should be used comprehensively and collectively through details analytics process by group of experts specialised in water and environment domains. A good data with a good analysis technique can support decision making with an aid from expert's insight, knowledge and experience. The implementation of Big Data Analytics (BDA) to the data received, collected and analysed are not solely an IT undertaking, but it is an alliance of various clusters of stakeholders such as Top Management, Subject Matter Expert (SME) and Information Technology (IT) Technical in determining the input, process and desired output or outcome. National Hydraulic Research Institute of Malaysia (NAHRIM) has been involved in the implementation of BDA technology by deliberating to Climate Change case study involving groups of SMEs consists of hydrologist, civil engineers, and researchers in the field of water resources, climate change and Information Technology. This paper is intended as an information sharing on NAHRIM experience in developing BDA projects with the aim to encourage water related agencies or parties to implement BDA in water and environmental management. We first briefly share on NAHRIM's first involvement and participation in BDA project and explain the mechanisms of implementing BDA through NAHRIM's experience. Then, we discuss the common issues that arose during the execution of BDA projects and finally, we conclude this paper by presenting several suggestions to carry out BDA projects.

Keywords: Big Data Analytics; water management; climate change

1 INTRODUCTION

Data as an asset is no longer a myth nowadays when the global evolution of data either in quantitative or qualitative forms carries not just monetary value, but also non-monetary value in almost every domain in the society. Understanding the value offered due to explosion of data, from water perspective overview those data could assist in preventing man-made disasters like overflowing rivers containing toxic waste, natural flooding, thus raising public awareness in water conservation and minimising the impacts of drought in arid regions (Cheung & Nuijten, 2014). Therefore, integration of exact tools with accurate algorithm to find a potential value from heterogeneous water datasets in a timely manner to support the decision on water resilient is challenging. As we enter the age of BDA, it is clear that we can take advantage from this phenomenon by engaging BDA technology in water domain. BDA projects are usually rigid and specific to the selected topic, but the results or outcomes generated and produced can be exploited and used by various parties depending on their level of understanding and critical thinking that are beyond the scope.

Like many terms used to refer to the rapidly evolving use of technologies and practices, there is no agreed definition of Big Data (Kitchin, 2013). However, researchers in this domain could conceptualise Big Data by looking at the perspectives of product-oriented, process oriented or cognition-oriented (Ekbja et al. 2015). The product-oriented perspective highlights the novelty of Big Data largely in terms of the attributes of the data themselves, the process-oriented perspective seeks to push the frontiers of computing technologies in handling Big Data structures and relations and the cognition-oriented perspective conceptualizes Big Data as something that exceeds human ability to comprehend and therefore required mediation through transdisciplinary work, technological infrastructures, statistical analyses and visualisation techniques to enhance interpretability. As a definition by Gartner, Big Data is a high-volume, high-velocity and/or high-variety information assets that demand cost-effective, innovative forms of information processing that enable enhanced insight, decision making, and process automation (Gartner, 2016). This definition covers all the perspectives mentioned before. Nonetheless the definition, BDA is predominantly associated with two ideas: data storage and data analysis (Ward & Barker, 2013). The aim is to minimise hardware and processing costs and to verify the value of Big Data before committing significant resources (Khan et al. 2014).

Due to forces like population growth and climate change, the water cycle and water availability are in time of flux. Some of the ways they are changing are predictable, enabling regions to plan for the changes and take action but some of these changes are more difficult to predict, requiring regions to be flexible and responsive (The Aspen Institute, 2015). From this, it shown that BDA in water related projects required involvement of ICT for data storage component and Subject Matter Experts (SME) for data analysis component.

Through effective used of data that is often already in place and available, BDA can provide various ways of achieving better water management, more adequate crisis management and even encouraging lower overall water

consumption (Cheung & Nuijten, 2014). BDA technology manage water related disaster by monitoring and detecting hazards, mitigate their effects, and assist in relief efforts where ultimately the goal is to build resilience so that vulnerable communities and countries as complex human ecosystems not only 'bounce back' but also learn to adapt to maintain equilibrium in the face of natural hazards (Data-Pop Alliance, 2015).

Comprehending the potential and value offered through BDA technology, NAHRIM in 2015 started to implement BDA project named NAHRIM Hydroclimate Data Analysis Accelerator (N-HyDAA) focusing on climate change case study which has an impact on managing water issues in Malaysia. Through N-HyDAA, NAHRIM has experienced processes involved in the project that NAHRIM would like to share with intention to encourage parties to maximise the usage of data through BDA Matrix Table and Gartner Analytics Ascending model which consist of i) Descriptive; ii) Diagnostic; iii) Predictive; and iv) Prescriptive Analytics. According to Lifescale Analytics (2015), descriptive analytics is the process of describing quantitatively what can be measured about a related domain. Diagnostic analytics look deeper into what has happened and seeks to understand why a problem or event of interest occurs. In predictive analytics, the analyst or Subject Matter Expert will combine current observations into predictions of what will happen in the related domain by using predictive modelling and statistical techniques. The last analytic approach, prescriptive analytics will address decision making and efficiency as soon as a good measure of accuracy on the predictive algorithm is achieved, and thus justify the prescriptive interventions.

The rest of this paper is organised as follows. Section 2 presents NAHRIM involvement in BDA project. Section 3 will discuss the implementation of BDA project and explains the process of implementing BDA in water management. Section 3 discuss some common issues that arose during the execution of Big Data projects and finally, we conclude the paper by presenting several suggestions to carry out BDA projects.

2 NAHRIM INVOLVEMENT

NAHRIM as a research institute focusing on R&D for water and environment, holds numerous water related and climate change data for Malaysia either primary or secondary data, collected through sampling activities, modelling, simulation and other R&D activities. Those data are being used for water and environment planning, supporting decision making and identified new potential R&D areas that can be diversified into various domain such as data projection analysis, climate change impact, sea level rise projection, hydro-climate and water resources related issues (Zulkifli et al., 2015).

Malaysia government has acknowledged the big data's potential by specifying BDA project as one of the national agenda. In 2013, the Prime Minister of Malaysia has officially announced the Malaysia BDA initiatives.

Malaysian Administrative Modernisation and Management Planning Unit (MAMPU) has been mandated to implement the BDA pilot project in government agencies in 2015. This initiative is joined by Malaysia Digital Economy Corporation

(MDEC) and MIMOS Berhad as a technology provider for this BDA project. Four public agencies with five pilot projects were selected to develop Malaysia BDA Proof-of-Concept (POC) and NAHRIM was one of them. NAHRIM's BDA POC project titled "Visualizing 90 Years of Projected Rainfall corresponding runoff after-effects based on river basin Malaysian Map" was develop to assist NAHRIM in visualising and analysing almost 1450 simulation-years of projected hydro-climate data for Peninsular Malaysia based on 3888 grids. Other projects were "Islamist Extremist Amongst Malaysians" by Department of Islamic Development Malaysia (JAKIM), "Flood Knowledge Base from a Combination Sensor Data and Social Media" by Department of Irrigation and Drainage (DID), "Data Analytics to Analyse and Build Fiscal Economic Models" and "Sentiment Analysis on Cost of Living gathering from Social Media" by Ministry of Finance (MOF) (MAMPU,2014)

Module involved in this BDA POC were Drought, Drought & Temperature, Rainfall & Runoff, Storm Centre and Streamflow. We successfully proved the concept of implementing big data analytics using NAHRIM hydroclimate datasets, comprises of time-series historical, current and projected data, acquired through the modelling of historical data. We were able to visualise 3,888 grids for Peninsular Malaysia, detected extreme rainfall and runoff projection data for 90 years, identified flood flow for 11 river basins and 12 states in Peninsular Malaysia, and traced drought episodes from weekly to annual rainfall data for 90 years.

NAHRIM BDA POC project was completed in September 2015, and later in August 2016, NAHRIM BDA project has turned to full project that catered three more modules; Climate Change Factor, Water Stress Index, and Water Stress Index Simulation. With the automated and systematic BDA project, later called NAHRIM Hydro-Climate Data Analysis Accelerator (N-HyDAA) system, will reduces the current manual process by humans and improves the quality and visual of data hence saving time and cost. It will also have benefited in discovering the vast potential data, sharing information and producing more effective decision making in timely manner.

There are two teams involved in this BDA project which were BDA Technology Team and SMEs Team to ensure the successful of the project. BDA Technology Team is responsible to provide technology consisting of the hardware, software and customisation services to develop the system. Meanwhile, SMEs Team for this project are the backbone or the brain of the project responsible for the solution, methodology, algorithm regarding the domain of the chosen business case. SMEs Team for the project was a mixture of various background of education and experience that come from hydrologist, engineers and IT researchers of NAHRIM's Water Resources and Climate Change Research Centre and Information Management Division.

3 NAHRIM BDA PROCESS

BDA project implementation in managing water issues by NAHRIM has been through a systematic and thoroughly process to ensure result and outcome produced provide a great impact on managing water issues. Viewing from ICT standpoint, BDA implementation is focusing on resolving dedicated business

issues by integrating main key players in the proposed business project. The key players in this context referring to SME who are well verse and expert about the business/domain choose for the project. Opinions from SMEs will be the core of how the project should be understand and developed to cater the issues arise from the business.

NAHRIM's BDA project is a combination of SME from various background named hydrologist, engineers and IT researchers. The SMEs play their part in every phase of the development process. Hydrologists and engineers focusing on the arithmetic calculation, parameters selection, analysis and data source while SMEs of IT researchers are dealing with the system development, data management and visualisation. This collaboration ensures the project developed met the purpose and objective with a result and outcome that benefit the users.

The complete process of implementing NAHRIM's BDA shows in Table 1 comprises of six phases; BDA Matrix Process, Requirement Analysis, System Architecture, Data Flow Diagram, System Development and Deployment. Details on every phase of the process are explained in Table 1. In the next section, we will go in depth on three crucial phases of this process; BDA Matric Process, System Architecture and Data Flow Diagram.

Table 1. Detailed process of NAHRIM's BDA project.

NO	PROCESS	EXPLANATION
1	BDA Matric Process	a. Identifying Business Direction and Business Problem Definition for proposed domain and project.
2	Requirement Analysis	a. Develop Requirement Analysis Book to highlight scope of work for the proposed project. b. Itemised scope of work based on importance of features (Must Have, Should Have, Could Have and Won't Have).
3	System Architecture	a. Designing System Architecture of the project based on Requirement Analysis Book.
4	Data Flow Diagram	a. Designing Data Flow Diagram of the system to determine the input data, processing and output data.
5	System Development	a. Development of BDA System including system testing.
6	Deployment	a. Deploy the System for user practice.

3.1 BDA Matric Process

NAHRIM's BDA project started by defining Business Direction and Business Problem Definition which are the main activities that guide the whole development of the project based on MAMPU assessment. In the Business

Direction activity, the tasks were to identify Project Objectives of the proposed project based on identified business case or domain are. The Key Measures as indicator that reflect the performance of the project also has been identified. In the Business Problem Definition activity, the tasks were to identify Business Function/Problem Area, Business Challenges, Problem Statements, Impact of The Problem, Business Questions and Data Sets Usage. The proposed of Business Function/Problem Area is to identify the specific domain or business area for the project. The Business Challenges is to explain the current challenges faced that must be tackled. Statement that will guide through the implementation of the project is created in Problem Statements to ensure the project is on-track with the business direction. Impact or issues based on business challenges were listed in Impact of the Problem task. In Business Questions, potential question that could be asked from the project was prognosed and finally data sets that are required in the project were identified in Data Sets Usage.

Table 2 shows the essence of NAHRIM's BDA project info over NAHRIM requirement analysis. From Table 2, it shows hydrologist and engineers are the main pillar of this project that lead to the usage and direction of the project to cater issues under Climate Change domain.

Table 2. NAHRIM's BDA Matrix Table.

ACTIVITY		TASK	ROLE
Business Direction	1. Project Objectives	• Domain Climate Change has been identified in this project, by focusing on:- a. Water Security (Water Stress & Water Availability); b. Weather Extreme Events (Floods & Droughts); c. Disaster Risk Reduction; • To prepare early data & information of potential water related disaster to related Agency for:- a. Possible issues arise in identified domain; b. Risk management planning; • To ensure an effective & efficient asset and resource management in risk management plan and issues in related domain; • To avoid, reduce and safe guard a high-risk area due to water related disaster and climate change;	NAHRIM's SME

Business Problem Definition	1. Project Objectives	- To identify and reduce lost (life, properties and ecosystem) due to disaster happened. - Number of lives that can be saved from hydrometeorology disasters; - Number of loss that can be saved from hydrometeorology disasters;	NAHRIM's SME
	2. Key Measures	- Number of risk management plan that has been implemented; - Size of potential area affected from hydrometeorology disaster; - Amount of area, infrastructure, life, properties and ecosystem that can be save from hydrometeorology disasters.	NAHRIM's SME
	1. Business Function/ Problem Area	- Water related disaster risk management & climate change impacts; - A systematic management and decision making for potential disaster related to hydrometeorology.	NAHRIM's SME
	2. Business Challenges	- To highlight, use and disseminate disaster information related to climate change extensively in Malaysia such as flood, drought, sea level rise etc. - Citizens concern on disaster related to hydrometeorological (pre & post), especially flood which has a high frequency and a large magnitude. Similarly, the occurrence of flash floods in major cities across the country in Malaysia where the cost to fix is very high.	NAHRIM's SME
	3. Problem Statements		NAHRIM's SME
	4. Impact of the Problem	- Loss of life; - Loss of properties (Government, Business and People); - Loss of income and jobs; - Loss of ecosystem.	NAHRIM's SME

5. Business Questions	• How many loss of life in the events?	NAHRIM's SME
	• How much loss from the event (Government, Business, and People)?	
	• What are preparation taken by Government?	
	• What are policies would be taken by Government to face potential disaster?	
	• Hydrometeorology and weather data;	
6. Data Sets Usage	• Climate change projection data;	NAHRIM's SME
	• Landuse data;	
	• Department of Stastical's data;	
	• Catchment data;	
	• Waterbodies & Water Treatment Plan data;	
	• Satellite data;	
	• Water Intake data;	
	• Infrastructure & Water Resource Facilities data (Groundwater & Surface);	
	• Socio economy data.	

3.2 System Architecture

N-HyDAA is a web-based information system that uses internet web technologies to deliver information and services. In N-HyDAA project, the design of System Architecture and Data Flow Diagram had been prepared based on the Requirement Book gathered during Requirement Analysis phase. Figure 2 shows NAHRIM System Architecture that represents the IT and non-IT components identified and involved in the project. IT component deals with data management, data authorization & authentication, data storage, operating system, web server and ICT infrastructure to run the system. The non-IT component in NAHRIM's BDA project involves module that required Data Accelerator for accelerating data processing and the modules are (i) Drought/ Storm Center/ Streamflow/ Rainfall/ Runoff (ii) Climate Change Factor (iii) Water Stress Index (WSI) and (iv) WSI Simulation.

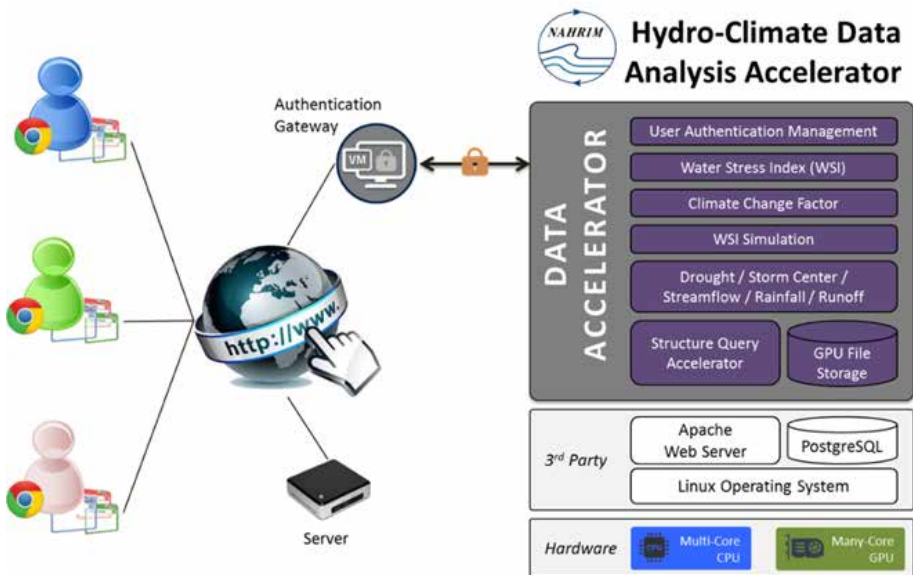


Figure 2. N-HyDAA System Architecture.

3.3 Data Flow Diagram

Data Flow Diagram showed in Figure 3 explain the overall flow of data for BDA project that are required in the project. We divided the flow into 5 layers; (i) Data Acquisition (ii) Data Cleaning & Integration (iii) Data Repository (iv) Analytics and (v) Presentation. Each data layer performs a particular function. Data Acquisition consists of components to gather raw, pre-process or unclear input data from all sources, such as rainfall, runoff, streamflow, and so forth. Data Cleansing & Integration consists of integration and process components in amending or removing data flow from the sources to the data repository layer in the architecture. Data Repository stores data in a columnar storage format for accelerating data parallel processing and improving query performance and extensibility. In Analytics layer, the queried data will be extracted from the repository to make it easier for users to perform big query processing and what-if analysis. Presentation layer gives access to different set of users where it consumes the data through web pages that are defined in the reporting tool (NAHRIM, 2016).

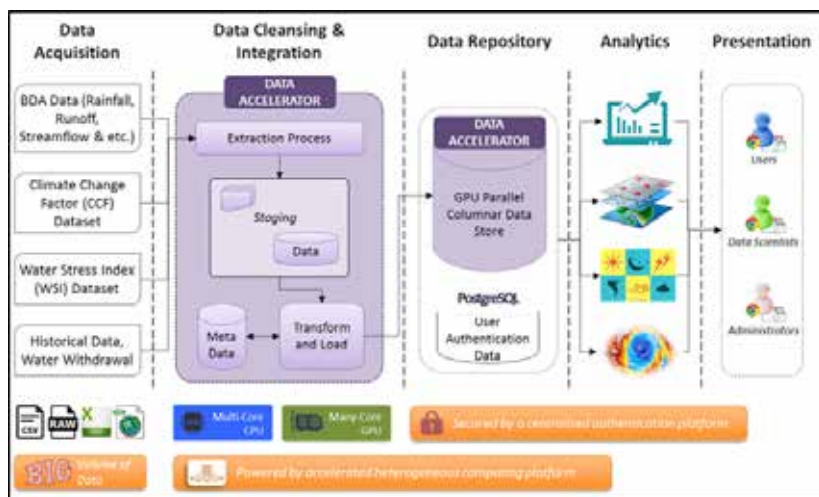


Figure 3. N-HyDAA Data Flow Diagram.

4 ISSUES AND SUGGESTIONS

Implementing BDA technology in water domain does imposed several issues and misconceptions in the initial stage. Below are the lists of misconceptions arose and our suggestions to carry out the BDA projects.

4.1 The thought of completing the V's

In the case of NAHRIM's BDA project, the thought of complying all the Vs (Volume, Velocity, Variety, Veracity, Visualisation and so forth) for our datasets was the first issue raised and mandatorily be fulfilled. NAHRIM's assumption was, BDA project will not be successfully developed if our data did not fall in every criteria of the Vs for BDA. But throughout this project, NAHRIM decided to focus on optimising and exploring our 10 billion hydro-climate simulated projected data in structured format. We were proven wrong by the results of the analysis where Volume, Velocity and Visualisation is more than enough to give us the outcomes that NAHRIM required. Understanding our own data is the key-successful factor of BDA implementation. Based on the data, organisation should know what are the expected result required and what are the processes involved including data to be used, type of data, technology to be used, the technique to store the data, the method to process the data and how to integrate them and so on to gain insights and depth to solve real problems.

4.2 The role of IT in BDA

Most of domain owner believes that BDA elements comprise of technical aspects only. The truth is, business owners should be involved at all events of BDA development to solve problems related to their business and field. In NAHRIM

case study, we use BDA as a tool to develop an application that can centralized the information and analysis on web application. NAHRIM took advantages of the current BDA technology through outsourcing mechanism by technology provider while NAHRIM focus on providing advice and content that consist of data, methodology and algorithm in the analysis phase of the project. Our BDA project will not be successful without good cooperation among researchers and engineers in providing the data and it is not subject to technological problems alone.

4.3 The dispersion of data

Unstructured data is one of the data type that revolved around us most rapidly and increasingly. This data type mostly are random and not modelled. Therefore, there is an assumption that only unstructured data is used for BDA. As a data provider, NAHRIM is no exception into thinking that combination of unstructured and structured data is a must to be used for BDA. The initial form of data we collect such as rainfall, runoff and streamflow, are structured data used for hydroclimate projection for the years 2010-2099. These structured data are then analysed through algorithm accelerated by technology provided by technology provider. Through this data processing, we are able to produce data output such as drought visualization by state, month and year, rainfall patterns, magnitude of storms and so on. In this case, NAHRIM do not intent to use unstructured data since NAHRIM would like to focus on optimising the current structured data. This experience can refute the notion that combination of structured and unstructured is a must to perform BDA.

4.4 The relevancy of BDA

Most BDA use cluttered data, and not all data is valuable for analysis. Because of this criterion, the process employed to analyse the data obtained are often time-consuming starting from the data collection, data processing and data visualisation. Hence, there is a perception that the end result of BDA is only the data visualisation on the dashboard. The right thought should be what are the accurate action that business owners and organisations need to take with the analysed data. Based on NAHRIM experience in applying BDA, we only provide data for analysis. The information that has been generated through this technology is hopefully can help the ministries, government departments and agencies such as Ministry of Natural Resources and Environment, Ministry of Energy, Green Technology and Water, Ministry of Agriculture and Agro-based Industry, Department of Public Works, Department of Irrigation and Drainage, state governments and private sector to make a strategic planning and immediate action in a holistic manner that lead to sustainable development and climate resilience such as water management issues, drought and flood.

5 CONCLUSION

As the aim of Big Data Analytics is to “turn data into insights” for better decision making (Dargam et al., 2015), NAHRIM has response to Malaysia government initiative to implement BDA technology in managing our datasets that related to water and its environment. We shared briefly NAHRIM involvement in Big Data projects and explain the process of implementing Big Data Analytics which we have completed by identifying our own BDA Matrix Process, Requirement Analysis, System Architecture, Data Flow Diagram, System Development and Deployment. We also have discussed some of the common issues that we have encountered during the execution of Big Data projects and finally, we conclude the paper by presenting several suggestions to carry out big data projects. Through this NAHRIM's BDA project, it shows and indicate with a correct data, people and technology, BDA concept can be implemented especially in Government sector despite there are challenges and confusions towards the understanding of BDA concept itself.

ACKNOWLEDGMENTS

This project was supported by MAMPU, Malaysia Digital Economy Coporation (MDEC) and MIMOS and we are thankful to our team members from NAHRIM as well who provided expertise that greatly assisted the implementation of this project.

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BIG DATA ANALYTICS TOOLS ON MALAYSIA'S CLIMATE CHANGE PROJECTED DATA TO REINFORCE WATER RISK MANAGEMENT

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ABSTRACT

Climate change and variability together with non-climatic drivers have had physical impacts on the hydrological cycle especially in the recent decades. Evolution in the hydroinformatics discipline, with incorporating rapidly expanding body of climate change information is indispensable particularly for a sustainable water resources and risk management. As such, the growth of big data analytics technology have enabled stakeholders to quickly processed and analysed huge volumes of climate change data and generate accurate insights, that subsequently would open up promising disaster resilience and risk reduction approaches. The National Hydraulic Research Institute of Malaysia (NAHRIM) has been applying big data analytics over 10 billion projected hydroclimate data for Peninsular Malaysia, downscaled and generated at 6km horizontal spatial resolution from 15 realizations based on the 4th Assessment Report (AR4) of the Intergovernmental Panel on Climate Change (IPCC AR4). Big data technology elevated processing, analysis and visualisation of hydroclimate data that assist in mining and identifying the potential future water related extreme events and their magnitudes such as extreme rainfalls, floods, storm centers and drought. The analytics system enabled NAHRIM to highlight predictive, scientific and data-driven evidence of climate change impacts on the hydrology of Peninsular Malaysia. Therefore, the analytics and functions of big data indubitably provide support in strategies and decision making process as well as strengthening related policy on water related disaster risk management, resilience and adaptation of climate change such as National Policy on Climate Change and National Water Resources Policy.

Keywords: Big data, climate change, water disaster risk management, disaster risk reduction, decision making tools

1 INTRODUCTION

In recent decades, the occurrences of natural disaster events which brought catastrophic impacts to the environment and socio-economics, are increasing when compared to during in the 20th century. Meteorological and hydrological events such as storms and floods are becoming more frequent. In year 2015, both category of disaster events reported about 41% and 42% of the total natural disasters worldwide respectively. The 2011 flood event in Thailand is recorded as one of event with the highest losses of USD 43 billion with 813 fatalities (Munich Re, 2016). As in Malaysia, continuous heavy monsoonal rainfall from 14th to 25th December 2014 over the east coast of Peninsular Malaysia had caused widespread floods especially in the state of Kelantan. This flood event is considered as the worst flood in the history of the state, which thirteen (13) deaths were reported and around 340,000 flood victims were evacuated with an estimated total loss about MYR1.7billion (DID, 2015).

Furthermore, both climatic and non-climatic drivers such as rapid population growth, increased urbanization, industrialization and pollution factored the increasing number of water excess as well as water stress events that threatened the sustainability of our water resources. Changes in the climate system is expected to have a wide range of impacts on ecosystems, infrastructure, health systems, the economy, and particularly on natural resources. Subsequently, water-related risks are possibly magnifying in the future, thus early identification and investigation of potential hydro-meteorological extreme events throughout future climate scenarios are essential for providing and disseminating scientific evidence to develop and strengthen water related disaster risk management, resilience and adaptation strategies.

The global climate change agenda has become a key issue in water management all over the world. According to Ishida and Kavvas (2017), climate change impacts on the water-related sector and management needs to be reflected in order to enhance its reliability, and thus, watershed-scale climate change assessment is essential particularly in flood control and water resources management. Climate change projections at very coarse scales are available from various Global Climate Models (GCMs), however they are unable to resolve significant sub grid scale features essential for climate change impact assessment to hydrologic regimes (Fowler et al., 2007; Kavvas et al., 2007; Ishida and Kavvas, 2017). In general, two (2) fundamental downscaling approaches for bridging the resolution gaps between GCMs and regional and local scale that are statistical and dynamical methods. The statistical method established fixed empirical relationships across spatial scales between the large-scale climate and local climate. However, the latter makes use of limited-area models with progressively higher spatial resolution than the GCM and it is able to produce finer resolution information that can resolve atmospheric processes on a smaller scale, but requires intensive computational (Fowler et al., 2007; IPCC, 2013; Ishida and Kavvas, 2017).

Hence, application of big data technology and predictive analytics can be important tools in mining and processing massive volumes of hydroclimate data

generated from intensive and complex computation particularly in the context of climate change modeling and its relation to climate resilience. As the impacts of climate change are accelerating, and the urgent need for effective solutions is required, therefore, emerging role of big data technology in understanding and mitigating climate change risk are considered innovative and effective solutions for climate change (Namrata, 2017). Data-Pop Alliance (2015) and Emmanouil and Nikolaos (2015) have explored the opportunities, challenges and required steps for leveraging this new ecosystem of big data to monitor and detect hazards, mitigate their effects, and assist in relief efforts, which is ultimately to build resilience and maintain hazard equilibrium. Furthermore, the utility and potential of big data for disaster management is growing as the number and access to datasets are expanding rapidly (Beth et al., 2015). Although it is still being regarded as an emerging technology (Frey et al., 2016), it has been recognized as a promising approach in order to harness data science and big data for climate action by means of identifying revolutionary new approaches to climate mitigation and adaptation (UN Global Pulse, 2017).

In view of high potential effectiveness and high impact of big data analytics (BDA) technology on socio-economy activities particularly to address the current challenges faced by government agencies, the Government of Malaysia has announced the Big Data Analytics (BDA) initiatives in November 2013. Subsequently, in mid-2015, four (4) government agencies has been selected, which includes NAHRIM to participate in a strategic BDA initiatives project entitled "The BDA-Digital Government Open Innovation Network (BDA-DGOIN) and Proof-of-Concept (POC)" which were co-organized by Malaysian Administrative Modernisation and Management Planning Unit (MAMPU), Malaysia Digital Economy Corporation (MDEC) and MIMOS Berhad.

Therefore, this paper emphasized NAHRIM's first attempt in utilizing the technology of big data to analyse our own downscaled hydroclimate data over Peninsular Malaysia. Further development works carried out after the POC project in order to provide scientific insights and support resources and water engineering, planning and risk management are also highlighted particularly in the context of future hydro-meteorological potential impacts and consequences to the safety level of water risk under the climate change conditions.

2 DATA USED AND METHODOLOGY

2.1 Data Used - Projected Hydroclimate Data for Peninsular Malaysia

A comprehensive study conducted to assess the impact of climate change on the hydrologic conditions of Peninsular Malaysia (NAHRIM, 2014). Fifteen (15) climate projections for the 21st century by three (3) different coupled land-atmosphere-ocean Global Climate Models (ECHAM5 of the Max Planck Institute of Meteorology of Germany, CCSM3 of the National Center for Atmospheric Research (NCAR) of the United States, and MRI-CGCM2.3.2 of the Meteorological Research Institute of Japan) under four (4) different greenhouse

gas emission scenarios (B1, A1B, A2, A1FI) based on IPCC AR4 were dynamically downscaled onto 3888 grids at 6x6km spatial resolution by means of a Regional Hydroclimate Model of Peninsular Malaysia (RegHCM-PM) (Amin et al., 2017). The model includes a mesoscale atmospheric component and a physically-based hydrology model component, which the atmospheric model component used is the MM5 (the Fifth Generation Mesoscale Model) from NCAR (National Centre for Atmospheric Research) – a non-hydrostatic model which can be downscaled even to 0.5km spatial resolution (Kavvas et al., 2007; Shaaban et al., 2010; Amin et al., 2017). By coupling MM5 with the physically-based hydrology model known as Watershed Environmental Hydrology Model (WEHY), a realistic estimation and interactions between the atmospheric and land surface hydrologic process can be modelled (Amin et al., 2017).

There are five (5) main projected hydroclimate parameters readily available at these grids i.e. precipitation, air surface temperature, runoff, evapotranspiration and soil water storage, which are produced at up to hourly temporal increment for 30 years of simulated historical period (1970-2000) and 90 years future period of 2010-2100 for each fifteen (15) climate change realizations. These data, together with projected streamflow data are also available at basin scale for thirteen (13) river basins in Peninsular Malaysia. The size of the total projected hydroclimate data are over 10 billion records, which is hugely challenging to be analysed using traditional approaches and methodologies, especially in order to predict the future impacts of these climatic changes to hydrologic conditions and water resources.

2.2 Big Data Analytics - NAHRIM Hydroclimate Data Analysis Accelerator (N-HyDAA)

The term 'big data' is generally described and characterised by the amounts (volume), velocity and variety of data, exhaustive in scope, fine-resolution, relational in nature as well as flexibility in terms of data size extensionality and scalability (Data-Pop Alliance, 2015; Emmanouil and Nikolaos, 2015; Kitchin, 2013). The projected hydroclimate data in NAHRIM is massive in volumes, but not in terms of velocity and variety, as the dataset is already pre-processed and produced in structured and flat file format. In order to utilize, visualize and analyse about 1450 simulation years of projected hydroclimate data, the BDA system in NAHRIM known as NAHRIM Hydroclimate Data Analysis Accelerator (N-HyDAA) was constructed essentially for tracing and identifying specific data pattern, magnitude and extends of extreme hydro-meteorological events throughout the century.

The NAHRIM's BDA system was established based on Mi-Galactica, a high performance query accelerator that further advances data processing speeds by using Central Processing Unit (CPU) or Graphic Processing Units (GPUs) for massive parallel processing of unpredictable, complex and long-running query workloads developed by MIMOS Berhad (MIMOS, 2016). The developed data processing accelerator system, N-HyDAA addresses the high volume data processing challenges by utilizing the following features:

- Columnar storage for parallel data processing
- Heterogeneous structure query engine using GPU technology
- Geo-spatial accelerated processing
- Data visualization system of multi-billion record datasets

For instance, the effectiveness of data processing accelerator was capable to perform quick analytics and visualization in only 14 seconds based on one scenario table for the whole 3888 grids, and about 3.5 minutes for all tables and scenarios. Comparatively, the identification of 127 million points for polygon of river basins and regions, and transformation into visualization in web server takes 4.3 seconds, which is 77 times than other post-GIS systems.

In general, there are currently eight (8) analytics features developed, which four (4) modules are from POC: drought, temperature, rainfall, storm center and streamflow, while three (3) new analytics features: climate change factor (CCF), water stress index (WSI) and WSI simulation are developed to assist engineers, development and utility planners, and main stakeholders in making timely decision and strategies in water-related risk management and development projects. The overall infrastructure of the developed system is generalized in Figure 1.

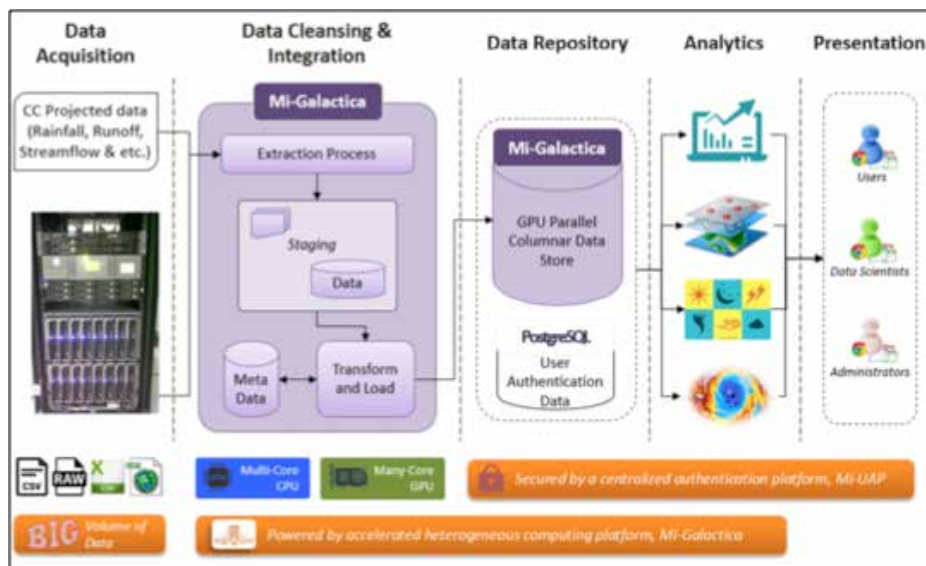


Figure 1. NAHRIM's BDA N-HyDAA data warehouse infrastructure.

3 ANALYTICS 1: POTENTIAL IMPACTS

Big data can help significantly in the prevention and preparation of water-related disaster and crisis management. Information derived from big

data analysis can help to anticipate crisis or reduce the risks that would arise (Emmanouil and Nikolaos, 2015). The findings from NAHRIM (2014) indicate that future rainstorms will change in temporal and spatial variability which are mirrored in the formation of runoff and streamflow leading to intensification of floods and droughts.

Predictive analytics in N-HyDAA functions enable data mining and extractions of possible future rainfall trends, patterns and magnitude at either yearly, monthly or even weekly basis, and based on each climate change scenarios concerned. For instance, Figure 2 shows a sample of visualized changes in the projected yearly gridded rainfall for year 2040, 2070 and 2100 extracted from the system. The figure shows that rainfall magnitude are projected to increase towards the end of the century. At the same time, daily-based temporal resolution for projected temperature and corresponding projected runoff to the rainfall distribution also can be investigated and visualized. The BDA system accelerated these analytics and visualizations, providing quick insights and identification of potential impacted areas, degree of severity and planning of strategic approaches in short or long-term mitigation and adaptation.

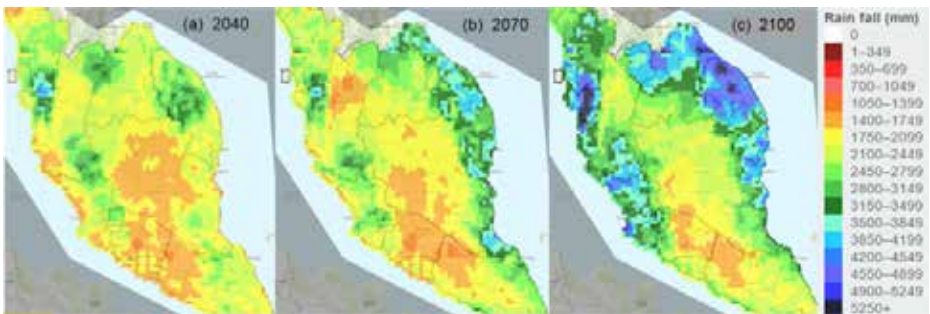


Figure 2. Projected changes in future yearly rainfall depth (in mm) and spatial distribution for year 2040, 2070 and 2100.

Further example, analysis conducted on future rainfall and river flow pattern in Kelantan River basin has detected possible rainfall and flood event in projected time horizon 2030-2040, with nearly similar pattern, magnitude and even storm centres with the disastrous flood that hit Kelantan and east coast states in 2014. The histogram in Figure 3 shows the projected basin-averaged daily rainfall (in mm) for the flood period in time horizon 2030-2040 (based on average of A1B scenario) that resembles the pattern of 2014 flood, that was identified and generated instantly by the N-HyDAA system. There are two episodes of highest daily basin rainfall identified – 168mm and 160mm during the event in the mentioned period. Concurrently, the system is also able to visualize the rainfall distribution and magnitude during the event as also shown in Figure 3.

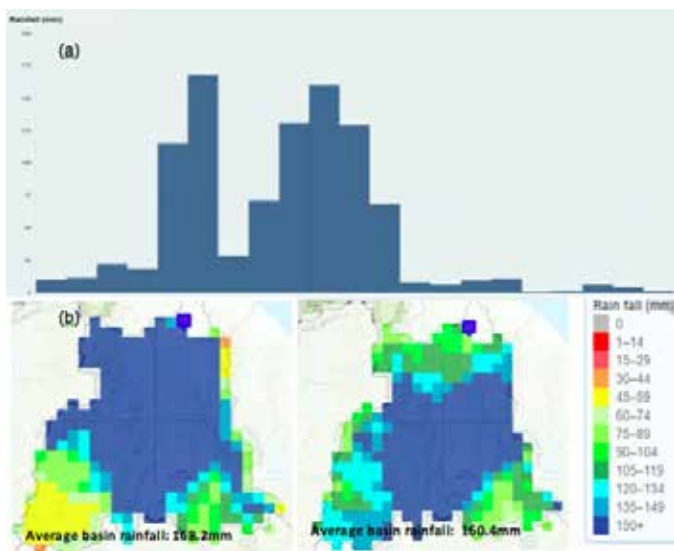


Figure 3. (a) Graph of basin averaged projected daily rainfall during the identified flood event in time horizon 2030-2040, and (b) the spatial distribution of the two highest rainfall depth during the future projected flood event in Kelantan River basin.

The system also enables us to query and predict the area and extents of extreme, prolonged dry periods that may poses threats to future water resources and supply. Previously through manual hydrological assessments, NAHRIM has discovered potential future drought years; based on drought intensity and recurrences. With BDA system, the areas of possible drought, reduction and changes in rainfall pattern and magnitude can be easily identified and visualized, and thus proper mitigation and adaptation strategies can be carried out to adhere the impacts. Figure 4 shows the projected three-monthly gridded rainfall in early century for time horizon 2020-2030, which has been identified as one of the extreme drought event that may affected the whole peninsular. Almost all states are projected to receive very low rainfall of below 700mm during the first six months of the period mentioned above.

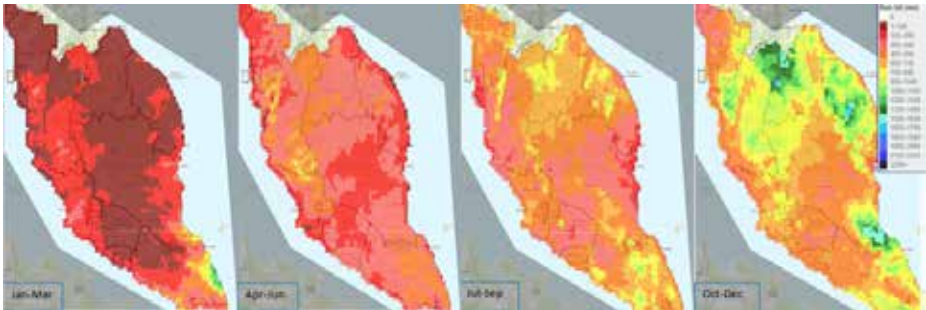


Figure 4. Changes in projected three-month rainfall (in mm) in time horizon 2020-2030.

Hence, the extraction and assessment of the future hydroclimate information influences longer-term evaluations of hydro-meteorological hazard and risk reductions in addition of resource management strategies through assumptions about possible precipitation and runoff conditions as these physical variables are translated into assumed variability in future water supplies, demands, and/or operational constraints.

4 ANALYTICS 2: EXPLORING THE SAFETY LEVEL OF WATER RISK

4.1 Climate Change Factor (CCF)

The BDA system is then further developed to analyse degree of vulnerability of water excess and stress due to changes in rain depth and intensity, and its consequence to river flows. NAHRIM (2013) has introduced a method to estimate degree of changes and impacts of future rainfall through derivation of a climate change loading factor or Climate Change Factor (CCF). CCF is generally defined as a ratio of projected future hydrological data such as rainfall to simulated historical data. By adopting the methodologies derived in NAHRIM (2013), the Generalized Extreme Value (GEV) and Extreme Value Type 1 (EV1) approaches are used to calculate the return periods of maximum daily rainfall events with return periods of 2, 5, 10, 20, 25, 50, 100 and 200-years. The same fundamental probability distributions are also applied in developing 1-day CCF for high flows, while estimation of low flow CCFs are based on GEV and Weibull distribution.

These equations /methodologies are embedded/incorporated into N-HyDAA analytics algorithm, which then are based by custom selection of climate change scenarios, by grid or region, and future 30-years time slices (2010-2040, 2040-2070 and 2070-2100). Figure 5 shows the estimated 1-day maximum rainfall under the average 14 realizations from three (3) emission scenarios (A1B, A2 and B1) for 50-year and 100-year Average Recurrence Interval (ARI) during middle of the century (time horizon 2040-2070). The maximum CCF values for both ARI years reach 1.90. The CCF value indicates that there might be an increase of

90% in rainfall depth compared to baseline/simulated historical years (1970-2000). However, it can be seen from the 100-year ARI map that the extents/areas with the maximum CCF values are increasing especially at the northern states, Selangor and Johor Bahru; compared to 50-year ARI's. However, at the same time, there are places that may have reduction in rainfall amount, as indicated by CCF values lower than 1 (Figure 5).

Whilst analysis based on GEV third quartile distribution yet shows a possibility of higher rainfall intensity and magnitude in future period 2040-2070, as depicted in Figure 6. Almost the whole Peninsular Malaysia is analysed to have CCF values of more than 1 in both 50 and 100-year ARI. The areas that are projected with extreme CCF values of 1.9 and above in the return period 100-year are distinctly wider, affecting most of Kedah and Perlis in north, Terengganu and Selangor, as compared to the same ARI in Figure 5. This estimation information is an approach of quantifying the scale of climatic change to surface water systems, which can act as an alarm and indication of future hydro-meteorological condition, and thus should be integrated into future water-related planning and risk management through strategic adaptive capacity.

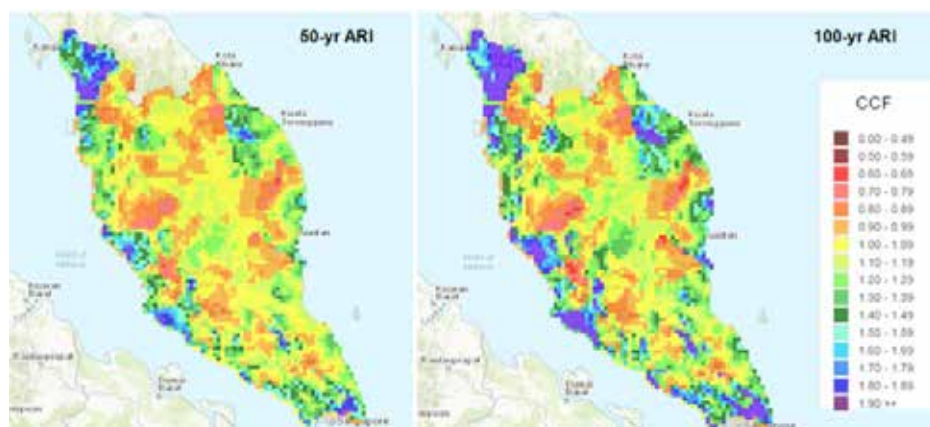


Figure 5. Comparison between 50-yr and 100-yr ARI of gridded rainfall CCF for time horizon 2040-2070

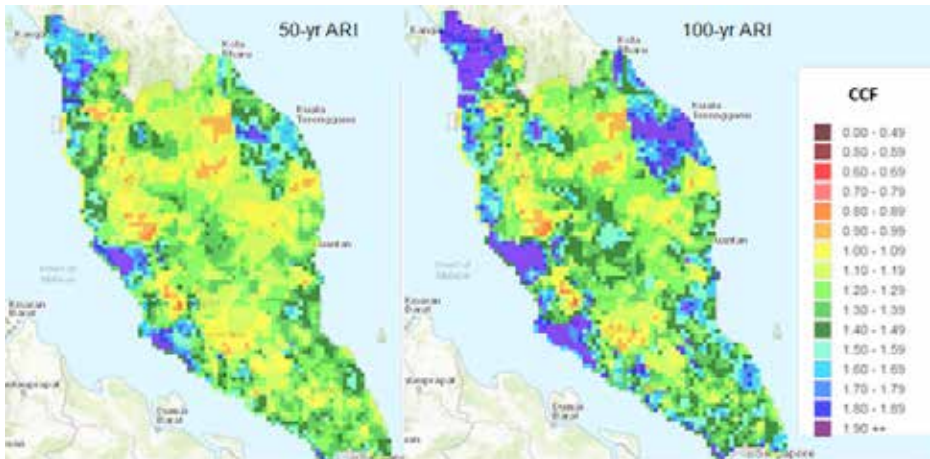


Figure 6. Comparison between 50-yr and 100-yr ARI rainfall CCF based on third quartile of GEV distribution for time horizon 2040-2070.

4.2 Water Yield and Water Stress Index (WSI)

Drought, changing patterns of precipitation and increased evaporation will effect water availability. Through big data analytic tools, federal and state government agencies, water authorities and stakeholders can have a quick but very informative input on future water yield compared to historical availability as an example shown in Figure 7. The figure shows the calculated water yield for historical and future period for 2030, for 80 districts of the entire Peninsular Malaysia, calculated based on simulated future runoff under four (4) different gas emission scenarios (B1, A1B, A2 and A1FI) by means of an average of fourteen (14) projections (A1B, A2 and B1), average A1B, average A2 and average B1. It can be seen from the figure that water yield for year 2030 under average of 14 scenarios, A1B and A2 are expected to increase compared to historical period (2 billion cubic meter (BCM) to 8 BCM) except for B1, which is expected to decrease by 3 BCM. In terms of temporal variations throughout the whole Peninsular Malaysia, the monthly trends of future water yield based on each scenario are similar to the simulated historical water yield, as depicted in Figure 8. Some monthly future show decreasing yields, though six months i.e. January, February, March, April, July and October might have increase water yield (from 1%-13%, 19%-26%, 27%-31%, 15%-27%, 4%-9% and up to 8% respectively) in the future. In December, the yields are projected to decrease about 3% to 17% (925 million cubic meter (MCM) to 6,196 MCM) in the future scenarios.

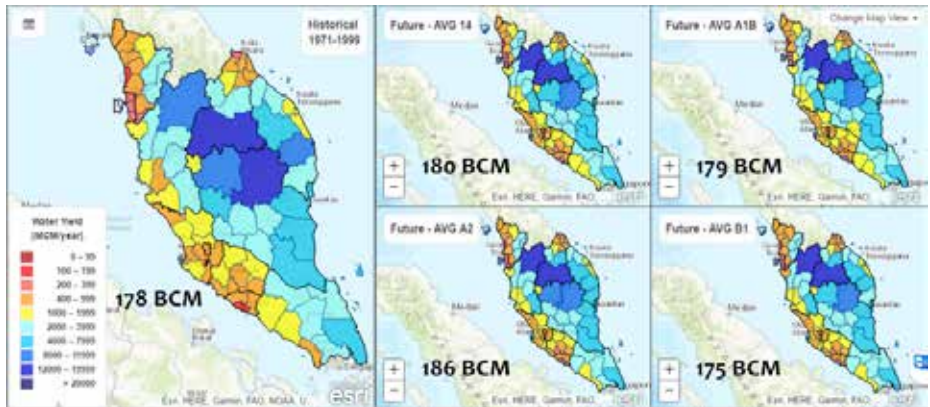


Figure 7. Comparison between simulated historical and future water yield (in billion cubic meter, BCM) in 2030 based on average scenarios of A1B, A2, B1 and fourteen scenarios.

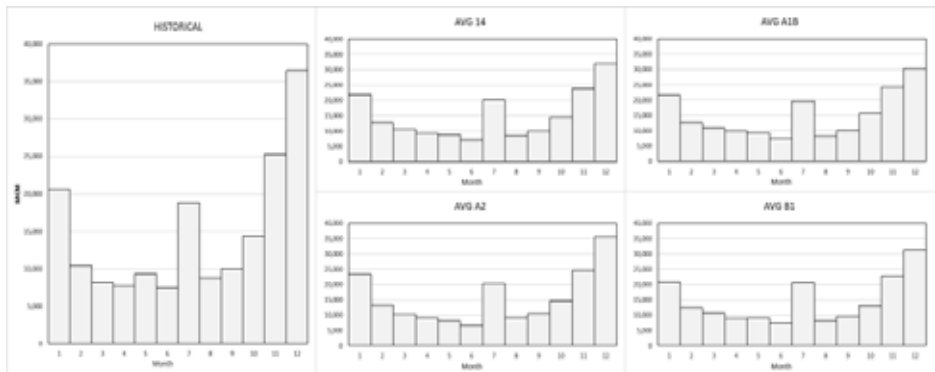


Figure 8. Comparison between monthly simulated historical and future water yield (in million cubic meter, MCM) in 2030 based on average scenarios of A1B, A2, B1 and fourteen scenarios.

Generally, all districts show an increase in future water yields, however a few districts in Pahang (circled in Figure 7) have relatively decreasing yield, which translates to possibly lesser water. The district of Lipis is projected to have 20% less of water under the scenario of an average of 14 projections compared to the simulated historical value (from 9,645 MCM to 7,698 MCM in the future), -20% based on average A1B, -16% (average A2) and -23% under the average B1 projections. In Jerantut, future water yields under the four (4) groups of average scenarios are also estimated to decrease about 23%-30% when compared to the simulated historical condition of 13,539 MCM.

The decreasing trend of future water yields in the two districts of Pahang can be correlated with the decreasing volume of projected monthly rainfall in 2030 compared to simulated historical rainfall, as shown in Figure 9a and 9b. In Lipis (Figure 9a) the total rainfall based on the scenarios are projected to decrease ranging from 16%-21%, which monthly rainfall for six (6) months are projected to be significantly lower than the historical values. The same rainfall pattern is projected for Jerantut (Figure 9b), where the future rainfall for each scenario decrease for about 2,880 MCM to 3,635 MCM from the simulated historical value of 17,600 MCM.

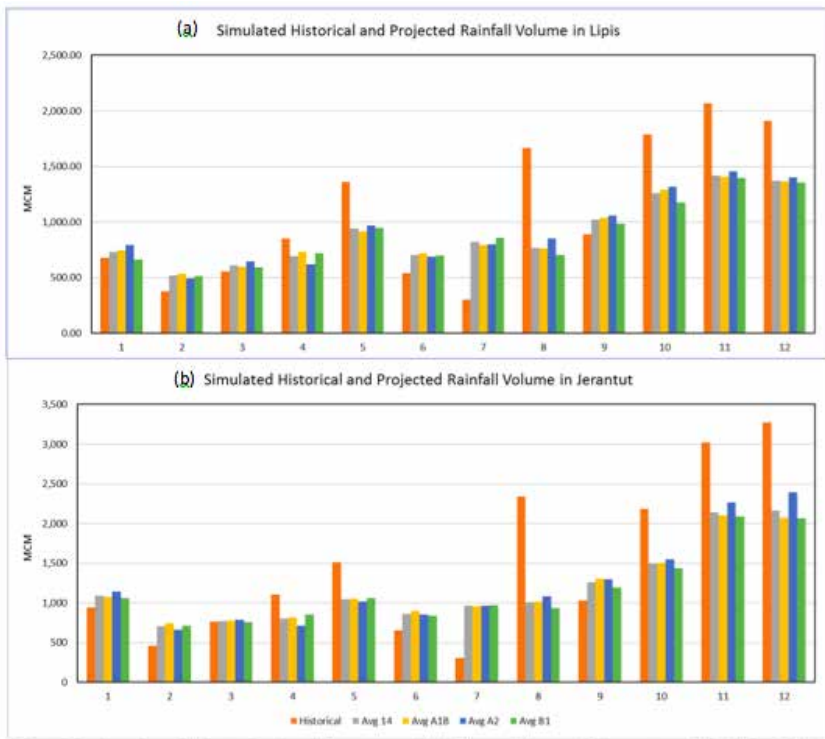


Figure 9. Monthly simulated historical and future rainfall (in million cubic meter, MCM) in 2030 based on average scenarios of A1B, A2, B1 and fourteen scenarios for district (a) Lipis, and (b) Jerantut

Furthermore, the Water Stress Index (WSI) approach developed by Pfister et. al. (2009) is used to construct water stress indices by means of projected water yield and water demand that is possibly impacted by climate change conditions. WSI is defined as an index calculated based on Stephen Pfister's model to represent the level of water stress in specific area by means of a ratio of total

water demand or consumption against water yield or availability (Brown et.al, 2011).

The estimation of WSI is determined by means of water withdrawal to water availability (yield) calculation as given in Eq. [1]. Water to water availability (WTA) is the ratio of total annual freshwater withdrawal (WU) made up of domestic, industrial and agricultural sector, to hydrological (water) availability (WA). The variation factor (VF) is derived to consider variation in precipitation. VF is calculated using Eq. [2] by means of the standard deviations of average monthly ($s^{\circ}\text{month}$) and annual ($s^{\circ}\text{year}$) rainfall, in order to calculate a modified WTA (WTA° as in Eq. 3), which is used to differentiate watersheds with strongly regulated flows (SRF) or non-strongly regulated flows (non SRF) (Pfister et. al., 2009). Finally, WSI is calculated based on a logistic function as in Eq. [4].

$$WTA = \frac{\sum WU}{WA} \quad [1]$$

$$VF = e^{\sqrt{\ln(s^{\circ}\text{month})^2 + \ln(s^{\circ}\text{year})^2}} \quad [2]$$

$$WTA^{\circ} = \begin{cases} \sqrt{VF} \times WTA & \text{for SRF} \\ VF \times WTA & \text{for non SRF} \end{cases} \quad [3]$$

$$WSI = \frac{1}{1 + e^{-6.4 \cdot WTA^{\circ} \left(\frac{1}{0.01} - 1 \right)}} \quad [4]$$

WSI analysis through BDA system in N-HyDAA is carried out for all 80 districts in Peninsular Malaysia. Consequently, the constructed district-based WSI are mapped for the respective districts and time horizon for Peninsular Malaysia as shown in Figure 10. As to determine the safety level of water yield-water demand, WSI is divided into five (5) stress categories as shown in Table 1.

Table 1. Water Stress Index (WSI) stress categories.

Stress Category	WSI
Low	< 0.1
Medium low	0.1 – 0.2
Moderate	0.2 – 0.5
High	0.5 – 0.8
Extremely high	> 0.8

As shown in Figure 10, it can be observed that most WSI in high and extremely high categories are located in the West Coast of Peninsular Malaysia by year 2030, especially in urbanized and highly populated areas such as in Penang, Klang Valley and Johor Bahru, as well in irrigation schemes areas such as Muda Irrigation Scheme (MADA) in Kedah, Kemubu Irrigation Scheme (KADA) in Kelantan and Barat Laut Selangor Integrated Agricultural Development Project

(IADA BLS), Under the average of 14 scenarios, for example, the high WSI in Sabah Bernam, Selangor would affect the water supply sustainability at IADA BLS, while the extremely high WSI in Johor Bahru is associated with high water demand due to highly populated and rapid development that would undoubtedly bring challenge in supplying sufficient treated water to consumers.

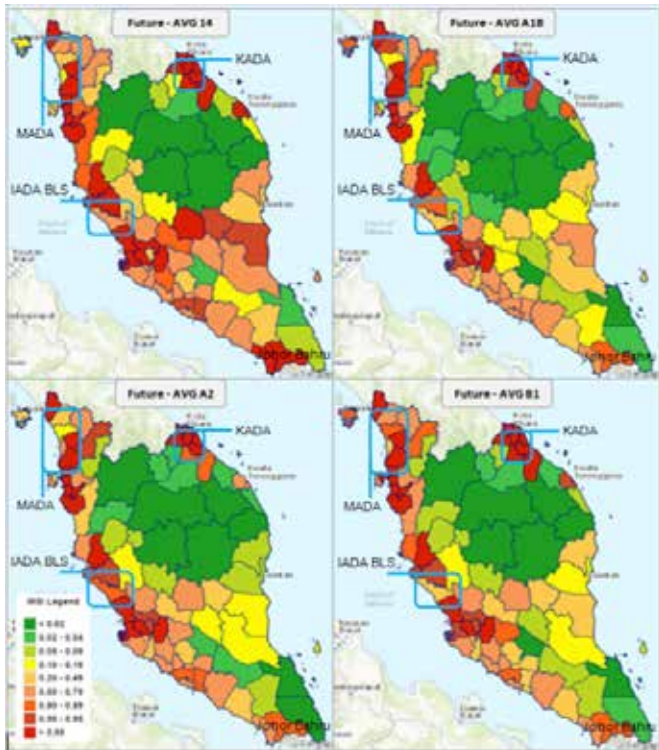


Figure 10. Water Stress Index (WSI) results for each scenario based on the districts for year 2030

5 CONCLUSION

Climate change studies by various agencies all around the world are in fact, applying the big data technology analytics in assessing and projecting climate variability in all its complexity and dynamic processes. Concurrently, the field of hydro-climate informatics, sciences and computational sustainability are rapidly growing and continuously changing. The uncertainties of future climate, its impacts to water resources and environment can be quickly processed, analysed and projected as key information in addressing and managing future water-related risks. Through data linkages, data mining and analysis, and predictive analytics, decision making processes are improved and thus can increase

situational awareness amongst high level decision makers, implementing agencies and local stakeholders in reinforcing our national policy on climate change, water management and disaster risk reduction for sustainable and resilient water resources such as National Policy on Climate Change and National Water Resources Policy. Possible changes, intensification and impacts to future water resources vulnerability and risk of extreme flood and drought events are identified and visualized through analytics on the hydroclimate projections and development of CCF and WSI. More occurrences of water related events with higher magnitudes are expected in the future, as indicated by CCF values more than 1.9 at wider areas, decreasing trend in water yield and alarming WSI categories of high and extremely high water stress in urbanized and highly populated areas such as in Klang Valley and Johor Bahru.

Application and enhancements of N-HyDAA in the future, such as incorporating crowd-sourcing inputs are hugely beneficial particularly in providing comprehensive monitoring and evaluation system regarding to climate and water related risk management. Besides, with the automated and systematic process, big data technology and analytics have reduced the current manual process by humans and improves quality and efficiency in mainstreaming climate change for a sustainable and resilient future.

ACKNOWLEDGMENTS

The authors would like to thank Malaysian Administrative Modernisation and Management Planning Unit (MAMPU) and Malaysia Digital Economy Corporation (MDEC) for the opportunity to involve in the BDA Proof of Concept Project, MIMOS Berhad for providing the accelerating computing platform, Mi-Galactica, and the Ministry of Natural Resources and Environment for funding the development of N-HyDAA.

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SEEPAGE SIMULATION ON PUTRAJAYA EARTH FILL DAM

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ABSTRACT

A seepage simulation is focused on clay core-rock fill dam all the time. In this study Putrajaya dam taken as a case study, the saturated stable seepage computation model was used to analyse Putrajaya dam seepage problem. Two dimensional finite element model of the clay core-rock fill dam was established by the Geo-Slope/SEEPW Software. The numerical simulation computation was carried on dam seepage situation under the different conditions. A proposed model is describing the flow pattern and seepage behaviour through the dam. The flow is assumed to be two-dimensional and under steady-state condition. The non-linear differential equation governing the flow is solved using an iterative finite element scheme. The finite element formulation is computer-implemented into a flexible computer program called SEEPW. The results show that the seepage amount through the dam was equal to $3.5620 \times 10^{-8} \text{ m}^3/\text{sec}$.

Keywords: Seepage, Hydraulic Conductivity, Finite Elements, Modelling, SEEP/W

1 INTRODUCTION

It was found (Hasan, 1999) that from 1000 of dams being built today, almost 10 of the dams are failed or the ratio of 1:100. From the records available, more than 200 dams that failed in the 20th century which is involved the dam's size of either more than 15 meters high or less than 15 meters high. Of the total failures, 30 incidents are recorded between years 1950 to 1959, and 25 incidents are recorded between the years 1960 to 1965. Since 1998 the number of unsafe Dams in United States has risen by 33% to more than 3500, while federally owned Dams are in good condition and there have been modest gains in repair, the number of Dams identified as unsafe is increasing at a faster rate than those being repaired. US\$10.1 Billion is needed over the next 12 years to address all critical non-federal Dams which pose a direct risk to human life should they fail (ASCE, 2005).

There are many type of dam failure but the two common failure mode are hydraulic failure (overtopping) and piping or seepage. According to ICOLD, The International Commission on Large Dams, the most common causes of failure of

earth fill and rock fill dams is seepage and piping as shown in Table 1.

Table 1. Causes for dam (over 15m height) failure (ICOLD)

Causes Failure	Percentage (%)
Piping and Seepage	38
Overtopping	35
Foundation	21
Others	6

This survey was carried out in 1973. Hydraulic failure accounts for over 40% of earth dam failure and may be due to one or more of the following:

- i. By overtopping: When free board of dam or capacity of spillway is insufficient, the flood water will pass over the dam and wash it downstream.
- ii. Erosion of downstream toe: The toe of the dam at the downstream side may be eroded due to heavy cross-current from spillway buckets, or tail water. When the toe of downstream is eroded, it will lead to failure of dam.
- iii. Erosion of upstream surface: During winds, the waves developed near the top water surface may cut into the soil of upstream dam face which may cause slip of the upstream surface leading to failure.
- iv. Erosion of downstream face by gully formation: During heavy rains, the flowing rain water over the downstream face can erode the surface, creating gullies, which could lead to failure.

Seepage always occurs in the dams. If the magnitude is within design limits, it may not harm the stability of the dam. However, if seepage is concentrated or uncontrolled beyond limits, it will lead to failure of the dam. Following are some of the various types of seepage failure:

- i. Piping through dam body. When seepage starts through poor soils in the body of the dam, small channels are formed which transport material downstream. As more materials are transported downstream, the channels grow bigger and bigger which could lead to wash out of dam as shown in Figure 1.

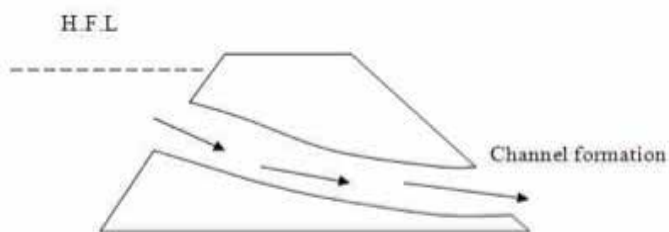


Figure 1. Failure of dam due to piping through dam body

- ii. Piping through foundation: When highly permeable cavities or fissures or strata of gravel or coarse sand are present in the dam foundation, it may lead to heavy seepage. The concentrated seepage at high rate will erode soil which will cause increase flow of water and soil as shown in Figure 2. As a result, the dam will settle or sink leading to failure.

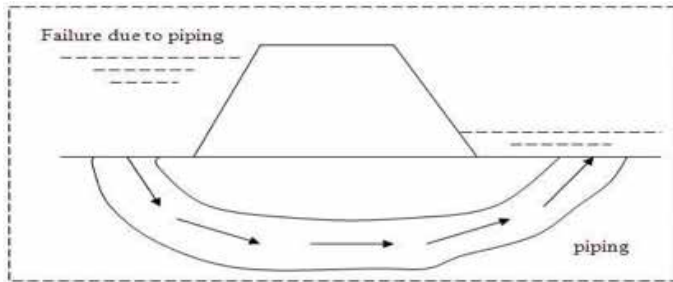


Figure 2. Failure of dam due to piping through foundation

- iii. Sloughing of downstream side of dam: The process of failure due to sloughing starts when the downstream toe of the dam becomes saturated and starts getting eroded, causing small slump or slide of the dam. The small slide leaves a relative steep face, which also becomes saturated due to seepage and also slumps again and forms more unstable surface. The process of saturation and slumping continues, leading to failure of dam.

2 PROBLEMS STATEMENT

Most of the past studies have involved seepage under the dam foundation (Ersayin, D., 2006). However, in embankment dams there is seepage in the dam body following a phreatic line. An earth fill dam's body prevents the flow of water from dam's back to downstream. However, with the most impermeable materials used in the dam's body, some amount of water seeps into dam's body and goes out from downstream of body slope. Seepage through or under an embankment may occur at a high enough rate to cause a boil, usually called a sand boil. Presence of sand boils can play a major role in embankment failure. Seepage of floodwater through or under an embankment is a normal process. However, when seepage occurs at a high rate, the seepage water can carry soil material with it.

3 STUDY AREA AND BASIC DATA

Putrajaya Dam is owned by Perbadanan Putrajaya was constructed in year 2001 with a 400-hectare man-made lake created. It is a zoned earth filled embankment located upstream of the confluence of Sg. Chuau and Sg. Bisa in the State of Selangor. The dam embankment was raised to EL 21.0m. It has a crest length of about 735m and maximum height of 18-30m with 145m length of embankment, and the seepage chamber location as shown in Figure 3.

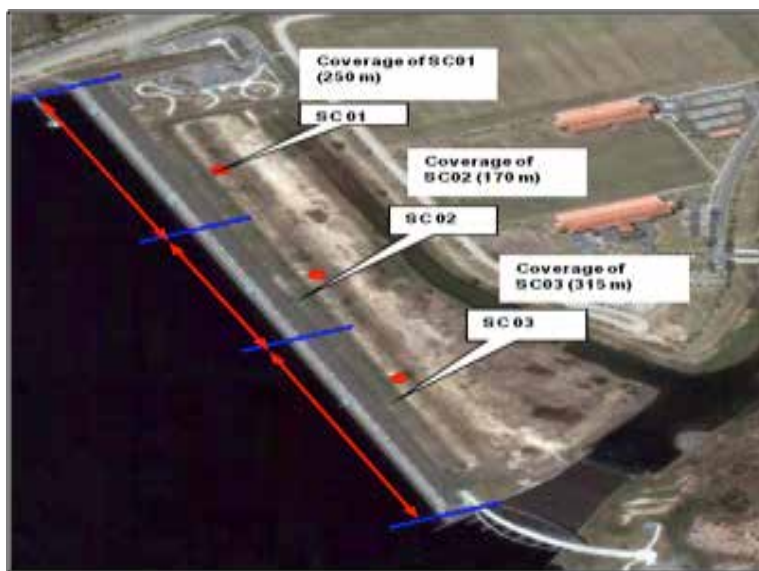


Figure 3. Seepage measurement chamber location – SC 02

The type of spillway is labyrinth with reservoir capacity of 24 million cubic meter and maximum flood discharge at spillway is 904m³/s. The main purpose of the dam is to provide recreational facilities to Putrajaya communities.

4 METHODOLOGY

In this study, the analysis computation was used for dam's seepage by GEO-SLOPE/SEEPW finite element software. This software's function includes; process kinds of non-uniform nature soil layer distribution and complex dam situation; set assign head and current capacity, water-proof boundary, and so on many kinds of boundary conditions; compute saturation line automatically; output equipotential line, streamline, saturation line, kinds of computed result curve and seepage quantity, slope fall of seep export and etc.

The boundary conditions were hydraulic conductivity, pore water pressure, and reservoir levels. We can find pore water pressure at different point by multiplying the piezometric reading with unit weight of water which is 9.81 kN/m³. SEEP/W divided the entire flow domain into a finite element mesh. Each element in the mesh must be associated with a soil type .in some points it is needed to forecast the seepage fluxes across some sections. We can predict the critical section for the seepage rate and usually it is located at filter outlet of the dam. It has been noted several times earlier that in seepage analyses only the head (H) or flow (Q or q) can be specified as a boundary condition. There are, however, situations where neither 'H' nor 'Q' is known. A typical situation is the development of a downstream seepage face such as illustrated in Figure 4

Another common situation is the seepage face that develops on the upstream face after rapid drawdown of a reservoir.

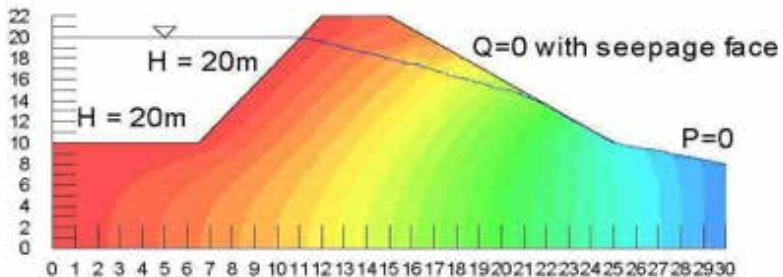


Figure 4. Seepage on down slope dam face (no toe under drain in this case)

A flow net is in essence is map of contours of equal potential crossed with flow lines. For the flow net to represent a correct solution to the Laplacian equation, the equipotential lines and flow lines must follow certain rules. The flow lines must for example cross the equipotential lines at right angles. Also, the area between two adjacent flow lines is called a flow channel and the flow in each channel has to carry the same amount of flow. A correctly constructed flow net is a graphical solution to Laplacian equation. SEEP/W does not create a true flow net because flow nets can be created for a few special situations. SEEP/W, however, does compute and display many elements of a flow net which are useful for interpreting results in the context of flow net principles (M. Subane, AR., 2010). For example, flow lines must be approximately perpendicular to equipotential lines. Features like this provide a reference point for judging the SEEP/W results. SEEP/W is formulated in terms of total hydraulic head. Contours of total head are the equivalent of equipotential lines. So equipotential lines can be drawn and displayed by creating a plot of total head contours. They are identical to equipotential lines in a flow net. In this example there are eight equipotential drops from 20 to 12, each one meter. In SEEP/W we can draw paths as illustrated in Figure 5 and flow net approximation as shown in Figure 6. These are lines that an imaginary droplet of water would follow from entrance to exit; they are not flow lines in the true context of a flow nets. In flow net terminology, the area between two flow paths is called a flow channel. In a flow net, the amount of the flow between each flow line must be the same; that is, the amount of flow is the same in each flow channel. It is possible for simple cases to compute flow lines so that they create exact true flow channels.

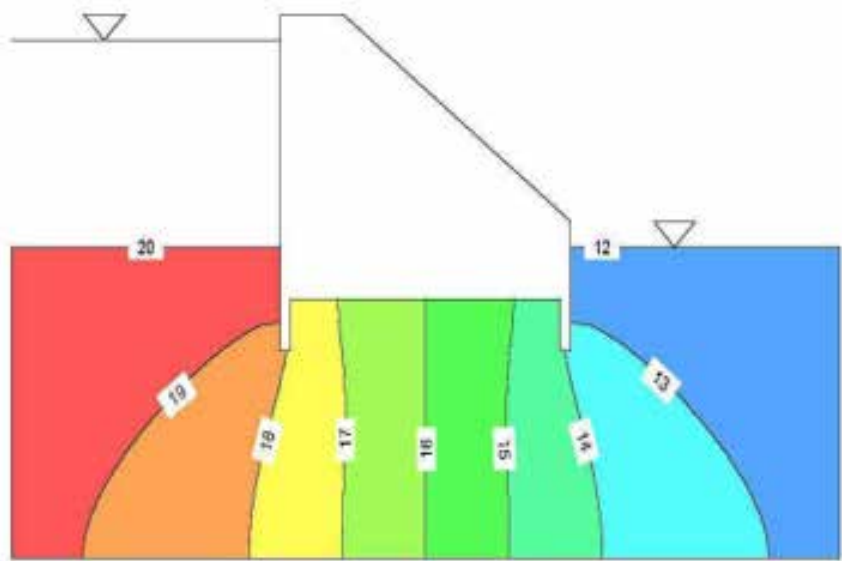


Figure 5. Plot of total head contours or equipotential lines.

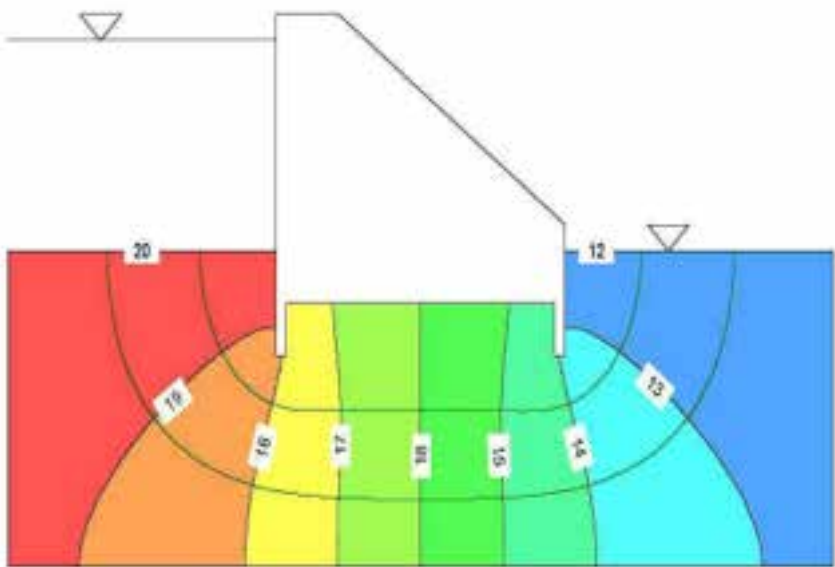


Figure 6. Flow net approximation

The permeability of embankment and foundation materials adopted in the design based on Angkasa Consulting Services S/B Consulting Engineers are as in Table 2. Only two main parameters were adopted in this simulation study. Three seepage measurement weirs are installed to facilitate the measurement of the seepage water through the dam embankment, only one of the chamber was used in this studied. The foundation and ground water at the abutments draining into the dam toe area (which is not significant in the case of Putrajaya main dam considering the abutment elevation is low.

Table 2. Permeability of embankment and foundation materials

Material	Coefficient of Permeability (m/sec)
Rock fill	1 x 10 ⁻²
Clay core	2 x 10 ⁻⁹
Shoulder Material	1 x 10 ⁻⁸
Sand fill	1 x 10 ⁻³
Organic soils and soft clay	1 x 10 ⁻⁸
Alluvial silty sand	1 x 10 ⁻⁷
Naturally occurring residual soils	1 x 10 ⁻⁸

Base on suggestion by Angkasa Consulting Services S/B, the maximum allowable seepage value at each seepage chamber estimated is 0.5 litre/sec. This seepage measurement chambers comprise of a 900 V-notch weir, stick gauge (depth indicator) and basin. The objective of the V-notch weirs is to measure the amount of dam seepage water. Base on Angkasa Consulting Services S/B, seepage quantity is estimated using the following equation:

$$Q = 1340 * H^{2.5} \quad [1]$$

where,

Q is the flow, litres/sec.

'H' is the head of water upstream over the bottom of the V-notch, meter. The measured seepage rate will be compared against the permissible seepage rate by International Committee of Large Dam (ICOLD), which is 0.5 l/s (as suggested by Angkasa Consulting Services S/B) for each seepage chamber set by the dam designer for dam safety evaluation. The Table shows the seepage V-notch weir Conversion Table 3 and the dam cross section is shown in Figure 7.

Table 3. The seepage V-notch weir Conversion Table

Water depth above weir (mm)	Flow (l/sec)	Water depth above weir (mm)	Flow (l/sec)	Water depth above weir (mm)	Flow (l/sec)
0.00	0.00	28.00	0.19	56.00	1.05
2.00	0.00	30.00	0.22	58.00	1.15
4.00	0.00	32.00	0.26	60.00	1.25
6.00	0.00	34.00	0.31	62.00	1.36
8.00	0.01	36.00	0.36	64.00	1.47
10.00	0.01	38.00	0.40	66.00	1.58
12.00	0.02	40.00	0.46	68.00	1.71
14.00	0.03	42.00	0.52	70.00	1.83
16.00	0.05	44.00	0.58	72.00	1.96
18.00	0.06	46.00	0.65	74.00	2.10
20.00	0.08	48.00	0.72	76.00	2.25
22.00	0.10	50.00	0.80		
24.00	0.13	52.00	0.88		
26.00	0.16	54.00	0.96		

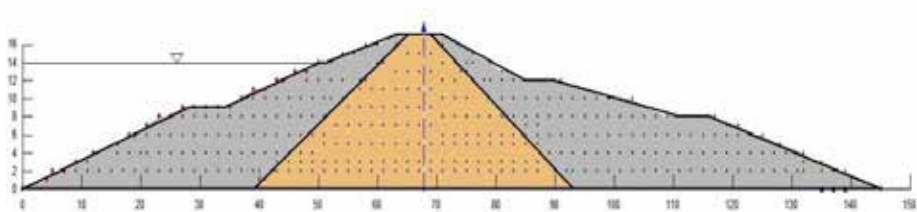


Figure 7. Dam cross-section

5 RESULTS AND DISCUSSION

The seepage simulation values obtained from seepage analysis using SEEP/W is $3.5620 \times 10^{-8} \text{ m}^3/\text{s.m}$ as shown in Figure 8 and Figure 9. Based on field data collection and using equation 1, the result for seepage collecting data for one-year period time (year 2010) is shown in Table 4.0.

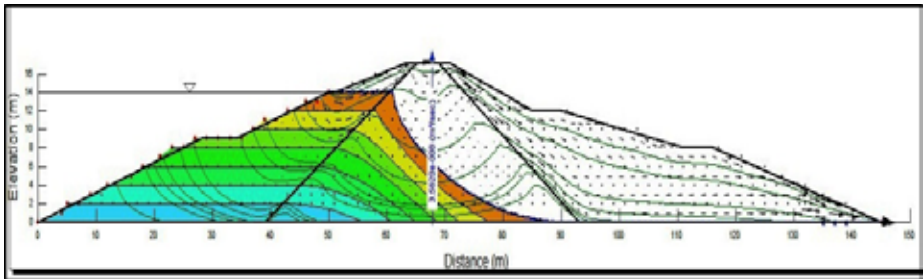


Figure 8. Contour and phreatic line ($q = 3.5620 \times 10^{-8} \text{ m}^3/\text{sec}$)

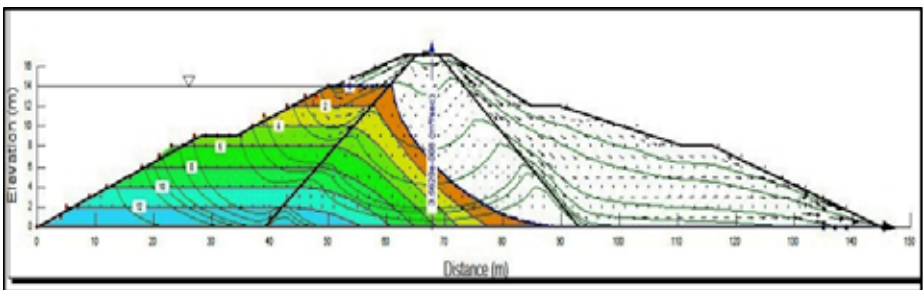


Figure 9. Phreatic line at 14.0m water head level ($q = 3.5620 \times 10^{-8} \text{ m}^3/\text{sec}$)

Table 4. Seepage rate base on varies value

Scenario	Hydraulic Conductivity			Seepage rate ($\text{m}^3/\text{s.m}$)
	Rock fills (m/s)	Clay core (m/s)	Reduce by	
1	5×10^{-3}	1×10^{-9}	2 times	1.73×10^{-8}
2	1×10^{-3}	2×10^{-10}	10 times	3.53×10^{-9}
3	1×10^{-4}	2×10^{-11}	100 times	3.56×10^{-10}

Based on the analysis carried out on the parameters and data of seepage flow on Putrajaya dam provided, it is found that the quantity is theoretically lower than the value set by the seepage of the International Committee of Large Dam (ICOLD) of $3.5620 \times 10^{-8} \text{ m}^3/\text{sec}$. Based on the analysis and simulation of seepage on Putrajaya dam found the seepage rate decreases with the reduction of hydraulic conductivity, k . Based on the analysis done by simulate three different values of hydraulic conductivity, seepage flow rate is found decreases with decreases of hydraulic conductivity (Mohamed, T.A., 2006). All simulation of these analysis as shown in Figure 10, 11, and 12. The lower rate of seepage is must better for stability of the dam (Ahmad A. A., 2009; Hasan, M.K., 1999). It's important for selecting material for filling the dam embankment with

low permeability material. It's ensuring the minimum seepage through the dam embankment.

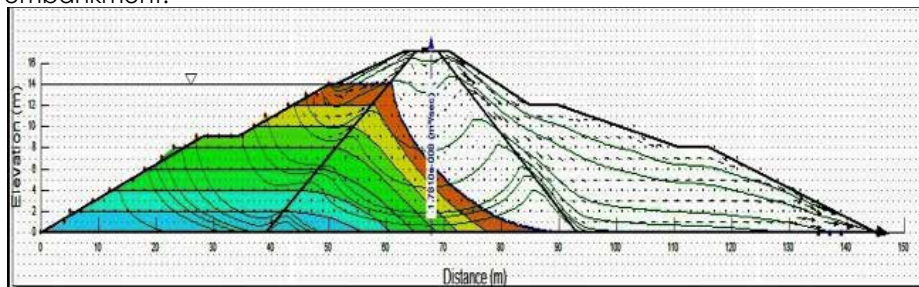


Figure 10. The seepage flow rate and phreatic line in reduce by 2 times.

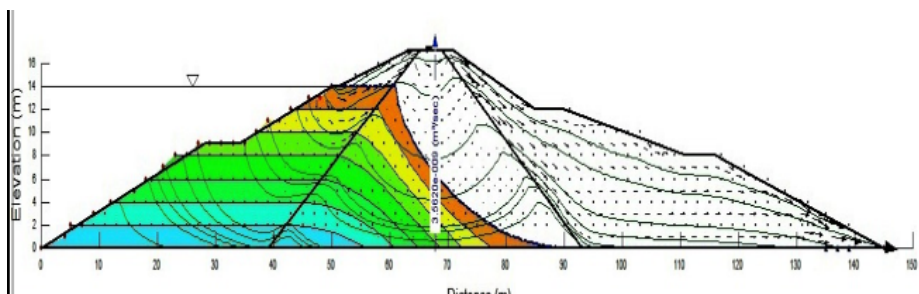


Figure 11. The seepage flow rate and phreatic line in reduce by 10 times.

6 CONCLUSIONS

In this study, the seepage problem through Putrajaya dam was carried out:

- i. A seepage flow rate value is not similar compared with the actual seepage monitoring value done by Perbadanan Putrajaya. The simulation value is less than combination of three seepage chamber values at Hydraulic Conductivity, k value of 1×10^{-2} m/sec for rock fill and 2×10^{-9} m/sec for clay core. The seepage simulation value is 3.5620×10^{-8} m³/sec.m, where the total seepage is 0.0262 l/s.
- ii. The simulation values of total seepage through the Putrajaya Dam were decreases when the value of Hydraulic Conductivity, k is reducing 2 times, 10 times and 100 times compare with allowable combined seepage limit of 1.5 l/sec, where the total seepage reduce to 0.0127 l/sec, 0.0026 l/sec and 0.0003 l/sec.
- iii. The seepage amount is reducing when the Hydraulic Conductivity, k is reduced. It shows that seepage dependent of k .
- iv. Finite element method can analyse seepage problem faster and more accurate than the other method such the flow-net method.

- v. The difference between observed value of seepage with predicted value of seepage is due to the limitation of two materials were considered compared to seven varies of materials zoning in the actual dam bodies.

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PHYSICAL HYDRAULIC MODELLING FOR THE DEVELOPMENT OF INNOVATIVE COASTAL PROTECTION STRUCTURE IN A 2-D WAVE FLUME

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ABSTRACT

NEXC Block is an innovative shore protection structure which is designed to protect the coast from severe erosion during monsoon season. While on a calmer period, the system encourages sediment accumulation thus expanding the beach naturally. NEXC Block is placed within Mean Higher High Water and Highest Astronomical Tide water level with sufficient toe protection for stability and efficiency, ensuring minimum adverse impact to existing coastal hydrodynamic. This paper aims to present the development process of NEXC Block in 2-D wave flume facility, where site measurement works are conducted for data validation and monitoring. Model dimension and hydrodynamic parameters are geometrically downscaled to 1:20. Two slope gradient values are used representing actual beach profiles at selected sites. Cohesive-form sediment size is 100 μ m. Three water levels; Mean Lower Low Water, Mean Sea Level and Mean Higher High Water with two significant wave height values H_s (irregular waves) are used for simulations. Beach profile is measured both before and after each simulation for erosion-accretion and stability analysis. Simulation on exposed beach without structure is also carried out as reference profile. The experiment shows that NEXC Block model structure installed with ample toe protection is a stable coastal protection structure. The structure is able to withstand strong wave impact during monsoon; no significant structural movement was detected. However scouring is prevalent at the toe of structure at unprotected structures. Some parts of NEXC Block and toe protection are exposed during monsoon season but less sediments are found eroded as compared to exposed unprotected beach; in which maintaining beach stability. Moreover during a longer calm period, the sediments are found to return back to the beach hence nourishing and expanding the beach naturally.

Keywords: Shore Protection, Hydraulic, Physical Model, Coastal Erosion, Natural Beach Accretion

1 INTRODUCTION

The main function of mostly available coastal protection structure is solely to protect shoreline from erosion. Some structures such as groin and offshore breakwater are more effective as those systems encourage sediment accumulation at leeside. However the installation of such systems are very costly and interfering with natural beach hydrodynamic processes. Wave breaking zone and the direction of longshore drift are directly affected hence creating adverse effects such as erosion to the other side of installation. Interference in this dynamic equilibrium leads to a change in sediment supply causing an increase or decrease in local sediment budget, resulted to accretion or erosion, respectively (Ghazali, 2006). In addition, the reflected wave energy back to offshore may cause beach profile changes due to resonant effects (US Army Engineers, 1984).

Seawall type of coastal protection can be considered as shoreline defense applications. Seawall is defined as an armoured revetment and an embankment with or without structural crest elements (Allsop, 1986). The protection wall is usually built parallel to the shoreline in order to prevent soil from sliding while offering protection from wave impact (Kamphuis, 2000). The construction material includes of concrete, rocks, steel, timber, rubber tyres, and sandbags (Pilkey, O. H.; Dixon, K. L., 1996). The shapes of coastal structures are determined by the use of the structure and generally can be categorised as vertical or nearly vertical, sloping, convex-curved, concave-curve, reentrant, or stepped. Shoreline defense structures, such as seawall and coastal blocks are situated beyond active hydrodynamic zone hence minimising adverse implication towards natural processes.

The aim of this study is to develop an innovative coastal block protection structure with minimum impact to existing coastal hydrodynamic processes. NEXC Block is designed and placed within Mean Higher High Water (MHHW) and Highest Astronomical Tide (HAT) water level to minimize the impact, and installed with sufficient toe protection for stability and efficiency. It protects the beach and acts as a wave breaker during monsoon season, while the system also encourage natural sediment accumulation on calmer period. NEXC Block, or NAHRIM Coastal Erosion Protection and Beach Expansion Block was developed by National Hydraulic Research Institute of Malaysia in 2015 – 2016. This applied research approach include conducting research and development using numerical models, experimental testing and pilot projects at selected sites.

Main objectives of 2D wave flume study include:

- a) To determine the ability of NEXC Block as a protection structure against coastal erosion,
- b) To assess the efficiency of NEXC Block as a coastal expansion mechanism in laboratory scale,
- c) To evaluate and verify the interaction between structures – water level – wave – sediment processes with NEXC Block site installations.

2 PROJECT SITES

Two locations are selected for the proposed NEXC Block pilot project development. The choices of sites are based on the differential exposures towards seasonal monsoon, morphological changes, hydrodynamic impacts and local socio-economic activities. Site A is located in Pantai Batu Lima, Port Dickson, Negeri Sembilan, within Regency Hotel compound which was severely eroded. The site is a sandy beach facing Malacca Strait, where wave impact is dominant during Southwest Monsoon (April – July). The bay is a popular attraction for tourism and water sport activities throughout the year.

Whereas Site B is situated at the east coast of Peninsular Malaysia, facing South China Sea. The shoreline are exposed to heavier wave impact during Northeast monsoon (November – February). The sandy beach of Pantai Rhu Muda, Marang, Terengganu is a traditional village community where fishing and small scale fishing related-industry are the main socio-economic activities. The straight coast off Marang is usually exposed and severely eroded during monsoon but eventually stable and naturally nourished during calmer period.



Figure 1. Site locations in Negeri Sembilan and Terengganu, Malaysia

Primary hydraulic data collection works have been conducted at both locations. Water level is measured with reference to the nearest tidal station at each site. Current and wave data are measured at 10m water depth using two ADCP units, approximately 3km from shoreline for a minimum 15 days. Bathymetry survey was done within 3km x 3km area, including sediment sea floor sampling and water quality monitoring. Beach profiling was done on monthly basis within few days after full moon event. Secondary historical wave and wind information are gathered from UKMO database. GIS survey was also conducted to verify existing physical, socio-economy and natural ecosystem at the selected sites. The data and information gathered are later analysed and used for numerical and experimental studies.

3 EXPERIMENTAL SETUP

Physical tests were conducted using a 50m - 2D wave flume facility located at Hydraulic and Instrumentation Lab, National Hydraulic Research Institute of Malaysia (NAHRIM). Model structure dimension and hydrodynamic parameters are geometrically downscaled to 1:20. Mildly sloping shore 1:15 and steep slope 1:3 are used representing actual beach profiles at selected sites. Cohesive-form sediment size is 100 μ m. Three water levels; Mean Lower Low Water (MLLW) 0.9m, Mean Sea Level (MSL) 1.0m and Mean Higher High Water (MHHW) 1.1m with two significant wave height values H_s (irregular waves) 0.1m and 0.2m are used for simulations. During the study, model structure is placed at a fix location along the flume, approximately at the centre of interest study area i.e. in active erosion / accretion zone. Beach profile is measured in the area of interest of 2.5m x 1.5m both before and after each simulation for erosion-accretion and stability analysis. Simulation run-time is 60 minutes referring to the duration of specific water level in each tide cycle. Simulation on empty unprotected beach without structure is also carried out as reference profile. Wave height and period is calibrated both manually and automatically using 3 units of wave probe located along the flume.

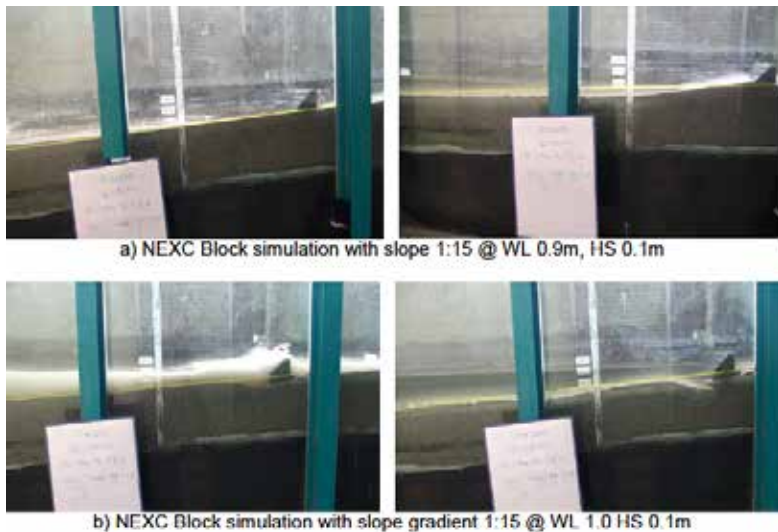


Figure 2. Experimental work in 2D wave flume facility

Interaction between waves and waves-structures due to variation of water levels and wave heights are studied and recorded. The scope of simulation works is simplified as below:

- a. Study on beach profile changes on accretion or erosion due to wave actions and sea levels on an exposed / unprotected beach (No Structure) as a control,

- b. Study on beach profile changes on accretion or erosion due to wave actions and sea levels with NEXC Block coastal protection structure model ,

In flume, the interest study area is marked as 0cm – 250cm with 10cm interval in Y direction (length of flume). Whilst in X direction the profile is marked from 0cm to 150cm (breadth of flume). Data sample for the first 30cm from both side walls in the flume is ignored and considered as unstable due to wave reflection effect. A datum or reference point is fixed at the top corner of flume wall. The distance between datum and beach surface is remarked as height in Z-axis at specific points. NEXC Block system is placed fixed along coordinate 0 – 150, 90 – 120 with single layer installation. Beach profile is measured manually each time before and after simulation to compare the differences, associated to erosion or accretion. In this paper, all erosion level is measured at the central of wave flume (x-axis) along the beach profile (y-axis) which is at cross-section 70, 30 – 160 (x, y). Figure 2 showed the experimental work conducted in wave flume with various hydrodynamic parameters on a stiff and mild slopes environment. Simulation equilibrium was set not more than 60 minutes.

Stability of coastal block as a coastal protection structure

The proposed coastal block aims to function as a natural solution to beach erosion and renourishment. Based on simple conservative economics, the structure can be used on either semi-permanent to permanent basis, contingent on the scope of the renourishment project. Coastal blocks work in two ways; firstly by breaking down the wave energy, therefore lessening erosion impact to the shore. Secondly, it encourages beach expansion or natural nourishment by permitting sea water which contains sediment and sand to pass through the tapered hole. During ebb flow, sediments are trapped behind the structure and unable to return to the sea. Over the time especially during a calm period, sediments are accumulated hence expanding the beach naturally. Hence the design of coastal block must be sufficiently stable against overturning and sliding failures. The calculation of wave forces on coastal block is according to EM 1110-2-1100 (Part VI), Goda formula for irregular waves (Goda, 1974; Tanimoto et al., 1976).

Overturning failure is rotation of wall about its toe due to exceeding of moment caused due overturning forces to resisting forces. Eq. [1] show the factor of safety against overturning is given by:

$$F_0 = \frac{\sum M_R}{\sum M_o} > 1.5 \quad [1]$$

where,

$$\begin{aligned} \sum M_R &= \text{sum of resisting moment about toe} \\ \sum M_o &= \text{sum of overturning moment about toe} \end{aligned}$$

The factor of safety sliding may be expressed by Eq. [2] below, which generally a minimum factor of safety of 1.5 is required:

$$FS = \frac{\sum R_v \tan \delta + k c_2 B}{\sum R_H} \quad [2]$$

where,

$\sum R_v$ = sum of the vertical resisting forces

$\sum R_H$ = sum of the horizontal resisting forces

δ = angle of friction between the soil and base slab k = the range between 1/3 to 2/3

c = cohesion of soil R = resistance force B = bottom length of structure

Table 1. Overturning check on coastal block structure at point 0

Option	Θ	Moment at 0		$\frac{\sum M +}{\sum M -}$	Remark
		$\sum M +$ (kNm)	$\sum M -$ (kNm)		
1	300	58.2	18.88	3.1	Ok
2	450	18.1	7.58	2.4	Ok
3	600	8.74	7.64	1.14	No (redesign 3a)
3a	600	14.71	7.64	1.9	Ok

Table 2. Sliding check on coastal block structure at point 0

Angle (°)	$\sum R_v$ (kN)	$\sum R_H$ (kN)	B (m)	FS	Remark
30	68	18.45	1.85	3.7	Ok
45	40.2	15.36	1.2	2.8	Ok
60	31.75	12.34	1.05	2.	Ok

The result of analysis is given as above. Table 1 showed the moment acting on coastal block for stability analysis on overturning check. The factor of safety for overturning is more than 1.5. The force acting on coastal block for stability analysis on sliding check are summarised as shown in Table 2. The stability of coastal block is analysed using full scale data collected from site and secondary data where necessary.

The structure is exposed to both forces from the wave action and lateral earth action specifically active earth pressure. As the block moves away from the backfill, there is a decrease in the pressure on the wall. The decrease continues until a minimum value is reach after which there is no reduction in the pressure and the value will become constant. Coastal block must be designed to withhold wave actions in order to maintain its stability against overturning and

sliding failures. Vertical distribution of pressure due to wave force acting on sea block is measured based on wave velocity and state (peak or trough) and water level (Hanbin GU et al., 2003).

There are three available designs of NEXC Block with different face angles; namely 30°, 45° and 60°. Due to technical, engineering and financial justification, the 45° angle is selected and suited in the physical experiment and site installations. Actual dimension of NEXC Block is 1.2m x 1.2m x 1.2m and each unit weighs approximately 2,500kg, made of reinforced concrete grade 40 which is suitable with marine environment. A laboratory scale model weighs about 5kg each and was found sufficiently strong and stable in providing protection against a downscaled wave and water level impacts in 2D wave flume simulations.

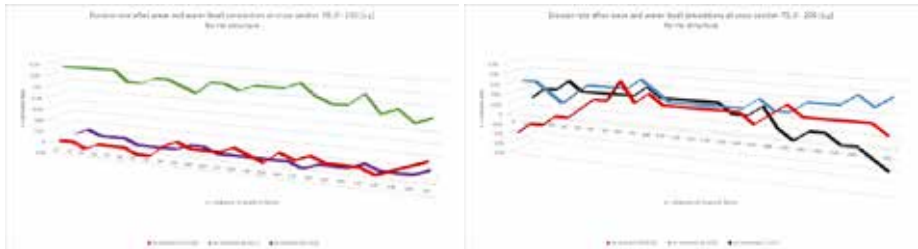
4 DISCUSSION ON LABORATORY DATA SAMPLING AND ANALYSIS

Figure 3 depicted the beach profiling on erosion-accretion analysis of selected simulations. No structure simulations as presented in Figure 3a shows that erosion occurred along the unprotected beach. Higher water level led to erosion farther landward. Higher wave magnitude caused greater erosion level or scarp height. Combination of high water and large wave magnitude caused catastrophic impact; total sand loss along the exposed shore. Rate of erosion is quicker and higher at a steeper slope and higher water levels conditions, until it reaches equilibrium. Data and information from this simulation is used as control and reference to the simulations with proposed coastal structure.

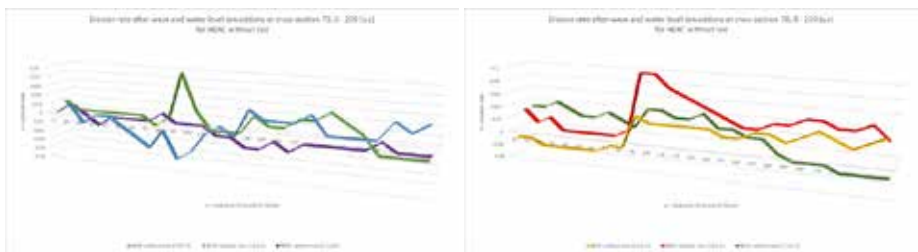
Figure 3b illustrated the erosion-accretion profile after simulations with NEXC Block are completed. It is found that with NEXC Block installation, erosion is controlled to occur only at the face of the structure, thus the backside of system is protected. Erosion was generally occurred at all water level scenarios MLLW, MSL and MHHW with H_s 0.2m. The protected beach at the backside of installation was found safe with the installation of NEXC Block. The system were also found stable against overturning and sliding failures.

It is also found that lower water level with lower wave magnitude encouraged sediment accumulation at the structure face. This was found in simulation with MLLW 0.9m and H_s 0.1m. In a longer calm period or outside monsoon season, the accumulation process is repeated until the sediments are returned to the beach and trapped behind the structure installation, resulted of a natural beach nourishment.

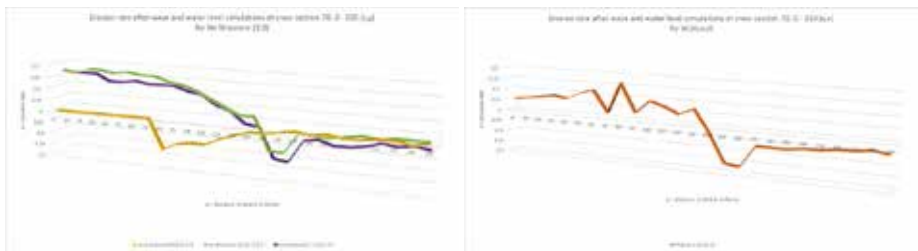
Even though erosion is controlled to occur only at the face of the structure, this has led to scouring problem at the toe of structure. Scour is found maximum when water level is at the structure level; within wave breaking zone. Wave impact on structure caused wave overtopping and sediment deposition near structure toe. Higher water level which submerging the structure minimized the impact of toe scouring. However due to higher water level reaching farther landward, it left the area unprotected hence erosion is found more landward. Nevertheless the structure managed to block and control the sediment from moving farther seaward from land.



a) Beach profiles at unprotected shores after simulations completed (control)



b) Beach profile changes with NEXC Block protection



c) Beach profiles with NEXC Block plus scour protection

Figure 3. Analysis of erosion-accretion after simulations completed

The simulations are also expanded by introducing scour protection installed at the toe structure to prevent structural failure. From Figure 3c, the analysis showed that sufficient toe protection had increased the stability of NEXC Block installation. Scour at toe structure can be minimized and naturally transferred at the bottom of the scour protector units. Scour protection units can be installed further seaward beyond breaking zone hence minimizing the impact of scour problem. However an increase in overtopping is expected with this type of installation due to the increase of height breaker.

Hollow slanting cone shape design at NEXC Block is the main feature that allows sea water to penetrate through structure during flood and ebb flows. Suspended sediments from offshore are carried by water and wave, then

penetrated the structure via hollow sections and trapped behind the installation as the water flows back out. Hollow sections on the structure must be designed less than 50% of surface area to ensure optimum sediment entrapment. The design was also able to distribute wave energy evenly during high tide, reducing overtopping and scouring impact.

5 ON SITE MEASUREMENT

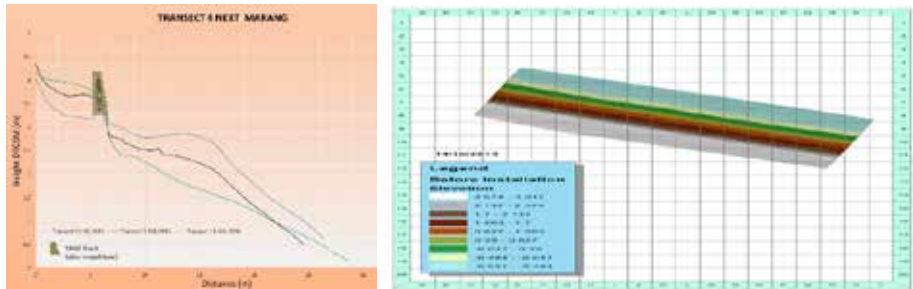
Data collection works and beach profile measurement at sites are shown in Figure 4a. The measurements are generally done on monthly basis to monitor continuous beach profile changes throughout the beach cycle period in one year calendar. Measurement is done automatically using Real Time Kinetic device, however in case of no signal or connectivity issues, auto level is used. In Port Dickson, NEXC Block system is installed within the Mean Sea Level to Mean High Higher Water marks. Changes in beach profile in front of the structure is greatly influenced by the high tide event every month. Scour protection as in Figure 4a is exposed during full moon and high tide events but the area is usually recovered during neap tide. The process is relatively typical for each tidal cycle.



Figure 4. Beach profile measurement at sites

However in Marang Beach, Terengganu the shore is only exposed to extreme event during northeast monsoon season. The installation of NEXC Block, located within Mean High Higher Water to Highest Astronomical Tide marks successfully control and limit the erosion at the face of the installation, whereas the backside area are protected. During northeast monsoon, the beach was starting to erode and scour impact can be seen as in Figure 4a. However NEXC Block has managed to block sediment from reaching further seaward. Sediments are trapped right at the back of the structure hence expanding the beach naturally. Coastal vegetation are also found to start recovering and regrow as in Figure 4b after monsoon season ended. Figure 5 showed the analysis carried out to investigate the changes of profile at sites due to NEXC Block installation. Based on the analysis it is proven that water level, wave magnitude and beach profile

are the main parameters in determining scour impact at the toe of structure, as per results in the study in 2D wave flume laboratory.



a) 2D Beach profiling analysis b) Plan view of beach profile changes in 3D

Figure 5. Analysis of beach profile changes.

6 CONCLUSIONS

In general, the rate of erosion is dependent of wave magnitude and water level. Higher wave breaks earlier and farther from the beach as compared to lower wave. However due to wave energy by higher wave, even after breaking, the wave propagates further inland with higher volume. Thus higher wave causes more erosion as compared to lower wave height, as same goes to water level parameter. As the water level increases, due to less bathymetry effect, the breaking point also moves to a point nearer to the beach thus leads to higher erosion rate. The experiment thus proves that the worst erosion occurs during the combination of high water level and high wave magnitude.

NEXC Block installation is found successfully controlled the erosion within the interest area. Sand loss due to erosion and scour is limited to only at the face of the structure, whereas the backside of installation is found stable. However during high water level and high wave event, scouring problem is prevalent. Hence scour protection units is proposed to be combined with NEXC Block installation. In a long term period, sediments are returned back to the beach hence naturally expanding the beach. Result of simulations in laboratory works in 2D wave flume has been validated using actual site measurement. Hence physical experiment approach is still considered as one of the best approaches in conducting study within coastal zones. For future work study, focus can be given to improve the efficiency of scour protection. It is also suggested that the material of NEXC Block is transformed into alternative environmental friendly materials.



a. NEXC Block installation in Marang, Terengganu



b. NEXC Block structure in Port Dickson, Negeri Sembilan

Figure 6. NEXC Block installation at sites

ACKNOWLEDGMENTS

All staff of Coastal and Oceanography Research Centre and Hydraulic and Instrumentation Laboratory, and National Hydraulic Research Institute of Malaysia is thanked for their assistance and contribution in conducting the experiment. Special acknowledgment also for the research team from Faculty of Civil Engineering, Universiti Teknologi Malaysia for their support. The Government of Malaysia is also thanked for the approval of research grant and related studies. NEXC Block has successfully accepted for registration approval in Industrial Design category MY16000690101 from MyIPO on January 2016.

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PREDICTING THE IMPACT OF CLIMATE CHANGE ON A WATER SUPPLY RESERVOIR

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ABSTRACT

Climate change was acknowledged to affect the hydrological processes and water resources sustainability. Predicting the impact of changing climate on reservoir water quantity and quality is necessary to ensure adequate supply of reasonable quality of raw water. An integrated hydrological and catchment management model was developed for Sg. Terip Reservoir using InfoWorks Integrated Catchment Model by means of hydrology Probability Distributed Moisture model to simulate the changes of reservoir inflow and capacity, and quality. Rainfall data at three nearby stations were used to calibrate the hydrological model over the period of 2010-2013. Future hydroclimate projection data at the lake catchment based on the downscaled of coarse resolution global climate model projection to 6km grid resolution by means of a regional hydroclimate model over Peninsular Malaysia were used to simulate the climate change impact on the water balance and water quality. The existing water balance of Sg. Terip reservoir is largely impacted by the river water abstraction at the nearby water supply intake and the water source from two water transfer schemes. Inflows from main tributaries were small. Existing water quality assessment showed good water quality throughout the measurement period. Long droughts will have large impact on the reservoir storage capacity. Impact assessment was carried out based on 3 slices period: 2010-2040, 2040-2070, and 2070-2100. Hydroclimate projection under the worst-case scenario, IPCC SRES A1FI of the global circulation model - CCSM3 showed that the lake catchment will receive lower rainfall amount in the mid of 21st century (2040-2070) and higher rainfall amount at the end of the 21st century (2070-2100). Simulation results indicated that lake water quality will change only slightly with alteration in these future rainfall events due to low pollution sources. Reservoir water quality may be influenced by the quality of water transferred into the water body.

Keywords: Drought, Lake Basin, Hydrology, Hydrodynamic Model, Water transfer

1 INTRODUCTION

Water supply reservoir has continuously been created for storage of water to support socio-economic developments. In Malaysia alone, more than 60 reservoirs have been constructed to date for water supply purposes since the first dam was built in 1896 (NAHRIM 2016). The size of the water supply reservoir ranges between 0.05 to 695 km² (NAHRIM 2016). The water availability and storage capacity of these reservoirs are very much shaped by the meteorological conditions and water abstraction. Changes in climate are expected to alter water availability by altering the hydrologic cycle, specifically precipitation and evapotranspiration, and other meteorological parameters such as temperature subsequently affects water balance and quality in the water bodies. A study in the Great Lakes region, for example, showed that a change in climate and land use affects the regional hydrology by increasing the percentage of precipitation resulting in increased surface runoff from 17.1% to 21.4% (Barlage et al., 2002). Similarly, projected climatic changes will lead to significant hydrologic response in the Californian Mono Lake Basin including more frequent droughts that ranges between 15% to 22%, a decrease up to 15% in occurrence of wet hydrologic years and a decrease by 15% of annual stream flows (Ficklin et al., 2013). Such changes in hydrological patterns will eventually affect water quality in the reservoirs including increase in nutrient run-off, alteration in thermal stratification and frequent algal bloom (Sahoo et al., 2013; Trolle et al., 2011). In Peninsular Malaysia, a downscale of the regional hydroclimate model predicted significant changes in rainfall pattern over the Peninsula (Amin et al., 2017; Shaaban et al., 2012). How these hydrologic changes affect water bodies in particular the water quality has not been investigated.

As water supply reservoirs were created based on past hydrology and water resources studies, the impact of climate change may not be incorporated in the original dam design. For example, a low rainfall pattern resulting from climate change may lead to water scarcity in the reservoir while an extreme rainfall event may disrupt the dam ability to store water and lead to excess water release or downstream flooding. Understanding how future climate change affects water quantity and quality in a reservoir is the goal of this study. Sg Terip reservoir in the Linggi Basin is one of the water supply reservoirs in Malaysia that are located within the water stress areas. The reservoir supplies water to 800,000 people in the district of Seremban and Nilai in Negeri Sembilan. The reservoir has recently been affected by frequent droughts induced by El Nino effects which affect its storage capacity. The lake water quality was reported to be in good condition while trophic state was within the mesotrophic to eutrophic in 2012 (Sharip et al., 2014).

The main objective of this study was to investigate the impact of future climate change, specifically rainfall pattern on the water balance and water quality of the water supply reservoir in tropical setting. The work is based on the development of an integrated catchment model comprising hydrological model and water quality model of a lake catchment.

2 STUDY AREA

This research was carried out in Sg Terip Reservoir, Negeri Sembilan ($102^{\circ} 00' 17''$ E, $2^{\circ} 45' 30''$ N). The reservoir was selected as research site due to the 2014 state water crisis where water level drops to the critical level. Sg Terip reservoir was created in 1987 for water supply purposes. It has a total surface area of 2.43 km² and catchment area of 25.9 km². The dam has a full supply level at 103 m above sea level corresponding to 47.7 MCM while the minimum operating level is 79.9 m above sea level contributing to an active storage of 44 MCM (SAINS 2010). The reservoir received water from three tributaries namely Sg Anak Buaya, Sg Terip and Sg Gantar. Two water transfer schemes provide source of water to Sg. Terip reservoir namely the Kelinchi-Talang Reservoirs (98 MCM) and Triang water supply scheme. The existing water balance of Sg. Terip reservoir is largely impacted by the river water abstraction by the nearby water supply intake (Sg. Terip Water Treatment Plant, 136 Mld) and the water source from the transfer schemes.

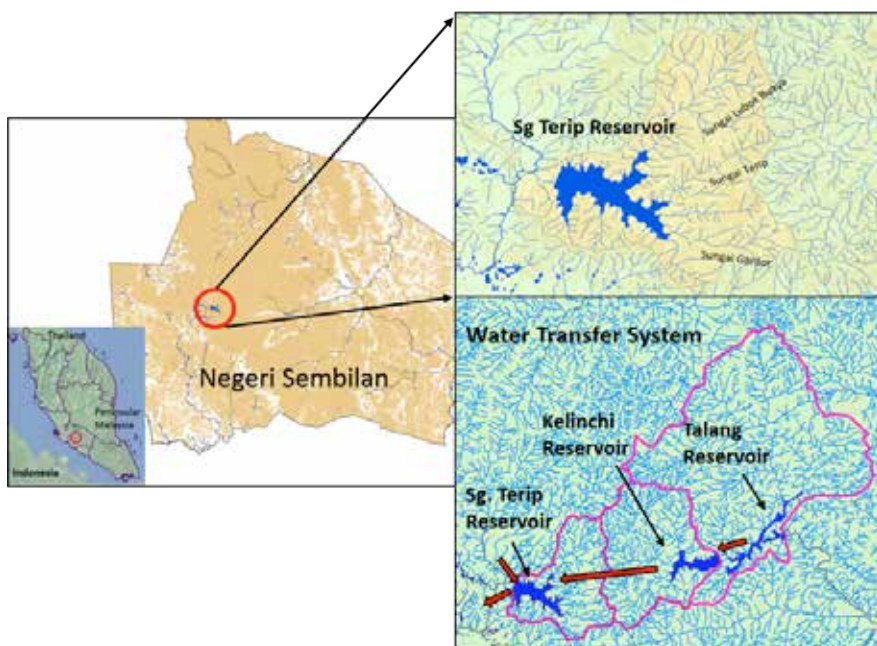


Figure 1. Sg Terip reservoir, its catchment and the water system. Inset the study location. Red arrow represents water transfer.

3 MATERIALS AND METHODS

3.1 Data collection

Water quality sampling was carried out in 2014 and 2016 involving measurement of physico-chemical parameters such as temperature, dissolved oxygen (DO), pH and turbidity using multi-parameter probe (YSI 6600) and water sampling. Sampling in 2014 was carried out once while sampling in 2016 was performed about 3-4 times. All sampling was done during the dry period. Water samples were analyzed for nitrate, ammonium suspended solids and phosphorus.

Bathymetric survey was carried out in September 2014 using a single-beam Reson-210 model of echo sounder unit. Depth soundings were acquired along 2.5 km² spaced at 100m intervals. In addition, a cross-lane was also established perpendicular to the main survey lines with symmetry with the survey lines to provide the cross-check comparison data needed to evaluate the consistency of the bathymetric data. Temporary tide level observation station was established near to the over-section Sg Terip Dam. All the measured tidal data corrected with the TBM value that fled from the Department of Survey and Mapping Malaysia's Temporary Bench Mark (TBM) to the tidal monitoring site. Rainfall and water level data were obtained from the Drainage and Irrigation Department of Malaysia and Syarikat Air Negeri Sembilan (SAINS). Rainfall data at three nearby stations (Kg Baru Pantai, Setor JPS and Ulu Bendul) were checked with rainfall data measured near the Sg Terip dam. Rainfall data were processed and analyzed for missing values and areal estimates. Water level data between 2003 and 2009 were assessed to understand trend and pattern of existing water balance.

3.2 Model development

The model platform utilized for this research was the InfoWorks two-dimensional Integrated Catchment Model (ICM) together with of the hydrology Probability Distributed Moisture (PDM) model, developed by HR Wallingford-CEH. ICM simulated the hydrodynamic flow and water quality pattern in the reservoir while the PDM model simulated the hydrology of the catchment taking into consideration the continuous rainfall and soil moisture (Moore 2007). The 2-D ICM model was based on the shallow water equations known as the depth-average version of the Navier-Stokes equations, Eqs. [1-3] assuming that the flow is predominantly horizontal while the variation of the velocity over the vertical coordinate is negligible (Innovyze 2015).

$$\frac{\partial h}{\partial t} + \frac{\partial(hu)}{\partial x} + \frac{\partial(hv)}{\partial y} = q_{1D} \quad [1]$$

$$\frac{\partial(hu)}{\partial t} + \frac{\partial}{\partial x} \left(hu^2 + \frac{gh^2}{2} \right) + \frac{\partial(huv)}{\partial y} = S_{0,x} - S_{f,x} + q_{1D}u_{1d} \quad [2]$$

$$\frac{\partial(hv)}{\partial t} + \frac{\partial}{\partial y} \left(hv^2 + \frac{gh^2}{2} \right) + \frac{\partial(huv)}{\partial x} = S_{0,y} - S_{f,y} + q_{1D}v_{1d} \quad [3]$$

Where,

h is the water depth; u and v are the velocities in the x and y directions;

$S_{0,x}$ and $S_{0,y}$ are the bed slopes in the x and y directions;

$S_{f,x}$ and $S_{f,y}$ are the friction slopes in the x and y directions;

q_{1D} is the source discharge per unit area; U_{1D} and V_{1D} are the velocity components of the source discharge q_{1D} in the x and y directions, respectively

Long duration continuous rainfall runoff model using PDM, which is available in InfoWorks was used for hydrological analysis. The hydrological analysis aimed at determining the rainfall-run off within the catchment which flows into the lake. The runoff from the sub-catchment will then be used as input to the 2D lake model. The catchment was divided into 29 sub-catchments to provide inflow from various tributaries (Figure 2). The water flowing directly to Sg Terip reservoir from Kelinchi through tunnel was included as inflow into the lake while the water abstracted to Sg Terip water treatment plant was included as outflow from the lake. Inflow from Triang Dam which was piped to Sg Terip reservoir since middle of 2016 was not included for model calibration. The survey data were imported into the ICM as ground model and interpolated using mesh technologies to capture the variation of the lake shape.

The water quality model process as described in Toriman et al. (2011) is simulated in 2-dimensional setting in ICM. Water quality determinants configured includes temperature, dissolved oxygen, pH, and nutrients such as nitrate, ammonium and total phosphorus.

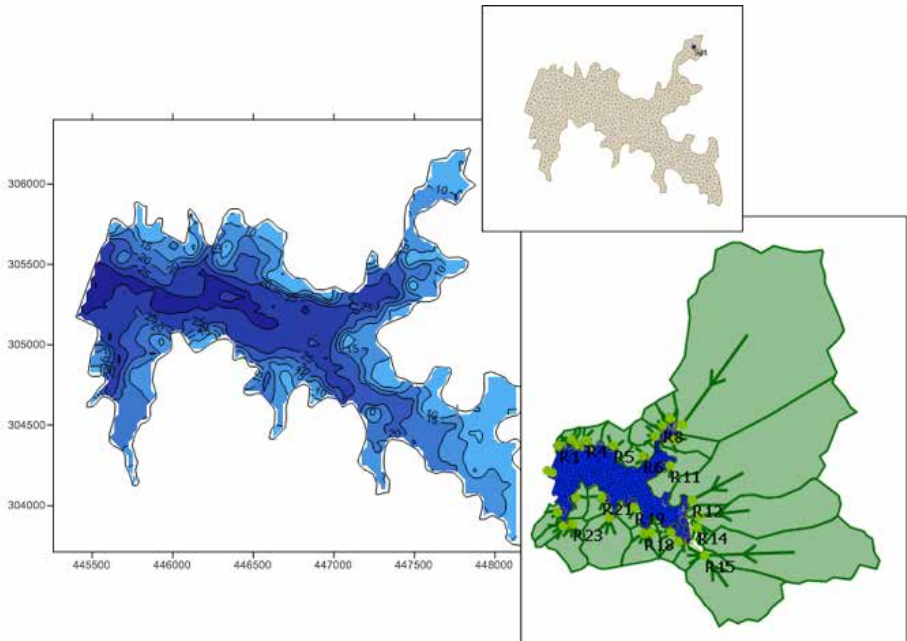


Figure 2. Bathymetry and two dimensional hydrodynamic and catchment models set up for Sg Terip reservoir

3.3 Model calibration and future climate projection

The model was calibrated based on rainfall data at three nearby stations and water level data for the period of 2010-2013. This calibration period was chosen due to availability of reasonably quality rainfall data. We used future hydroclimate projection data at the Sg. Terip catchment based on the downscaled of coarse resolution global climate model projection to 6km grid resolution to simulate the climate change impact on the water balance and water quality. The downscaled model was based on a regional hydroclimate model over Peninsular Malaysia developed by the National Hydraulic Research Institute of Malaysia (NAHRIM 2014a). Impact assessment was carried out using the hydroclimate projection under the worst-case scenario, IPCC SRES A1FI scenario of the global circulation model - CCSM3 and was based on 3 slices period: 2010-2040, 2040-2070, and 2070-2100.

4 RESULTS AND DISCUSSION

4.1 Measurement and calibration

Measurement over the period of July to October 2016 showed mean total inflows from main tributaries were small ranging between 0.8 – 2.8 m³/s. These low inflows were very much influenced by small catchment and long drought. Mean annual rainfall in 2016 was 1876.5 m³ compared to mean rainfall in 2015 (1911.1 m³). Figure 3 shows the variation of annual rainfall in all four stations. Average rainfall is about 2000 mm/year.

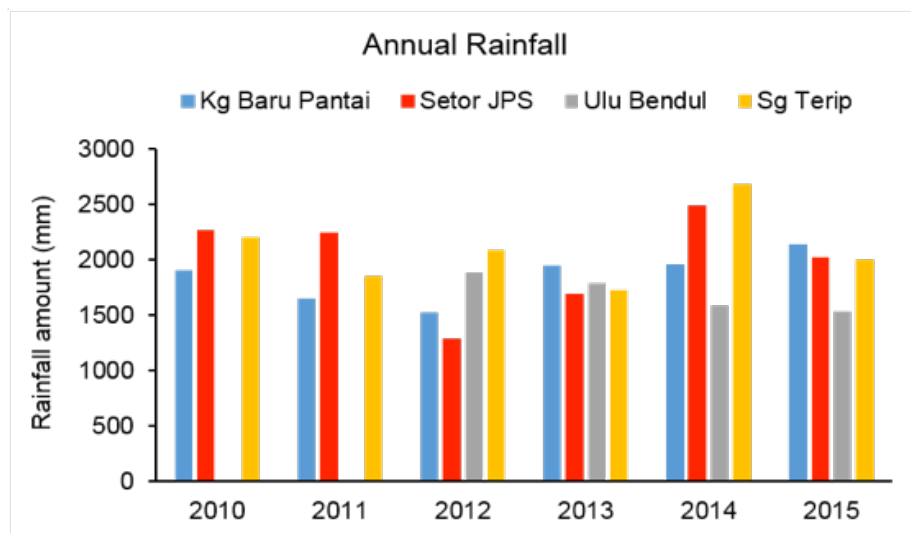


Figure 3. Annual rainfall at few stations near Sg Terip reservoir

The calibration process for the PDM uses similar approaches as describe in Azad et al. (2016) which include analysis of rainfall data, correlation between rainfall data and water level and rainfall factor. In this study, the calibrated parameter consists of rainfall factor, moisture storage distribution and time constant for surface flow. Model calibration showed reasonable match to the measured water level (Figure 4). However, it under-estimated the water level during the dry season likely due to differences in areal rainfall pattern, surface water evaporation and varying amount of water abstraction.

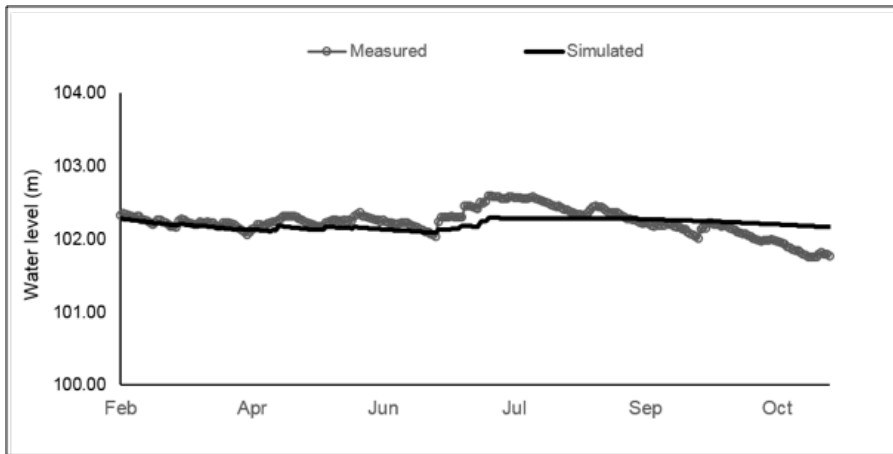


Figure 4. Measured and simulated water level in 2010

4.2 Existing conditions

4.2.1 Water quantity assessment

Based on the annual dam water level, the water level during 2016 was the lowest compared to the water level between 2010 to 2015 but showed similar trend of water level pattern for 2015 (Figure 5a). The water level in 2016 was about 2 m lower than the water level in 2015. Lower water level in 2016 could also be related to the small increase in dam level in 2015 which was likely to be related to longer dry season. Low water level in 2015-2016 were similar to the low water level in 2003-2006 (Figure 5b). Three-year water level of 2011-2014 showed similar pattern with three-year water level of 2007-2010 with no significant monthly variations.

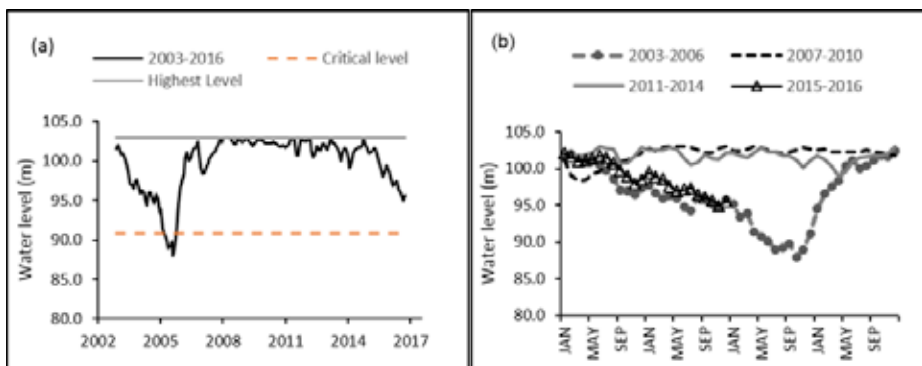


Figure 5. Pattern of reservoir water level over the period of 2003 – 2016 (a) long-term annual (b) three-year

4.2.2 Water quality assessment

Existing water quality assessment showed good water quality condition throughout the measurement period. Mean values for temperature, DO, pH and turbidity were 30.5 °C, 7.8 mg/L, 6.8 and 5.1 NTU respectively. Based on the Department of Environment Water Quality Index (WQI), the overall water quality in Sg Terip Reservoir was at Class II or clean with WQI ranging between 90 and 93. Water in the lake was observed to be clear and good transparency. Chlorophyll concentrations were generally low in this reservoir (<10 µg/L), with mean value of 3.72 µg/L and higher value >5 µg/L occurred at around 5 m depth. This is consistent with transparency of about 1.5m where photosynthetic active radiation may be penetrated to deeper water column to provide the light environment for plankton growth. However, low nutrient levels may limit growth of plankton communities. TP concentration fluctuated from 0.01 to 0.24 mg/L. Chlorophyll-a and TP values reported in this study are consistent with prior findings (Sharip et al., 2014). However, transparency level was much lower compared to the transparency value of 2.7 m reported in 2012 (Sharip et al., 2014). Vertical temperature profile in 2016 showed isothermic conditions indicating the non-existence of thermal stratification likely due to the artificial de-stratification system. DO remained above 4 mg/L in the surface layer until depth ~20m. Depth below 20 m has low DO nearing hypoxic conditions. Horizontal and vertical variation of water quality is shown in Figure 6.

The water quality of the incoming water is mostly clean. The stream temperature is colder < 29°C and transparent indicating undisturbed catchment. The estimation of sediment loading based on measurement of three main tributaries namely Gantor, Anak Buaya and Terip rivers in 2016 showed TSS load ranging between 0.13 to 2600 kg/d while TP and nitrate load ranged between 0.04 to 31.4 kg/d. Higher turbidity was recorded in water from Triang river that transferred into the reservoir (unpublished data SAINS). However, water quality in nearby areas measured between the period of September – November 2016, after the pipe is in operation, showed mild increase in nutrient and TSS.

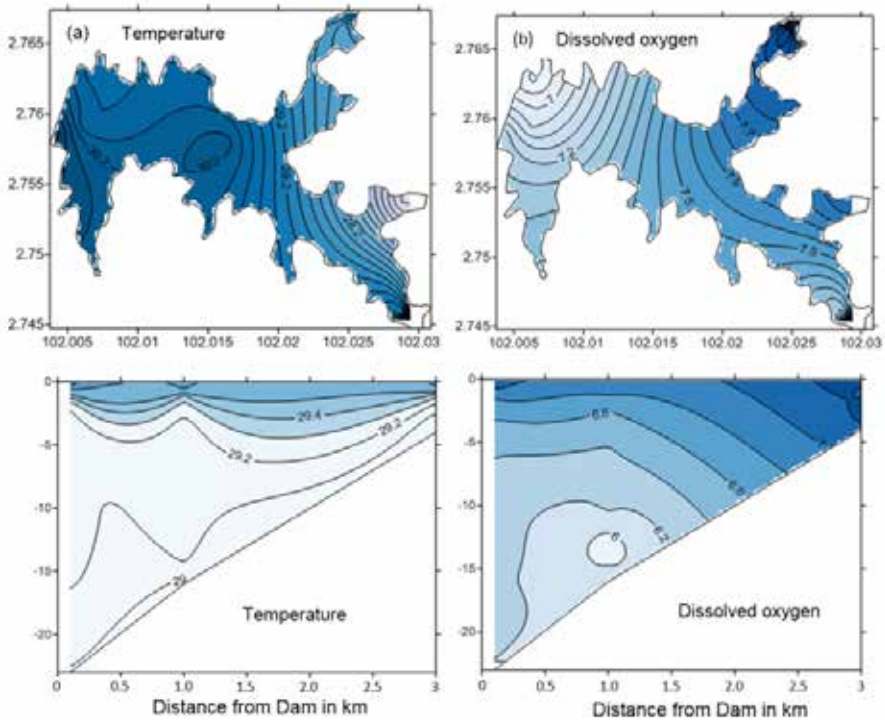


Figure 6. Horizontal and spatial variation of water quality

4.3 Conditions under future climate projection

4.3.1 Water quantity assessment

Rainfall pattern during future hydroclimate period using IPCC SRES A1FI scenario is shown in Figure 7. Based on hydroclimate projection under the worst-case scenario of the global circulation model - CCSM3, Sg Terip lake catchment will receive lower rainfall amount in the mid of 21st century (2040-2070) and higher rainfall amount at the end of 21st century (2070-2100). The mean and maximum rainfall amount during the mid-21st century was 2.8 mm and 225 mm per day, while the mean and maximum rainfall amount during the end of 21st century was estimated at 6.6 mm and 689 mm per day respectively.

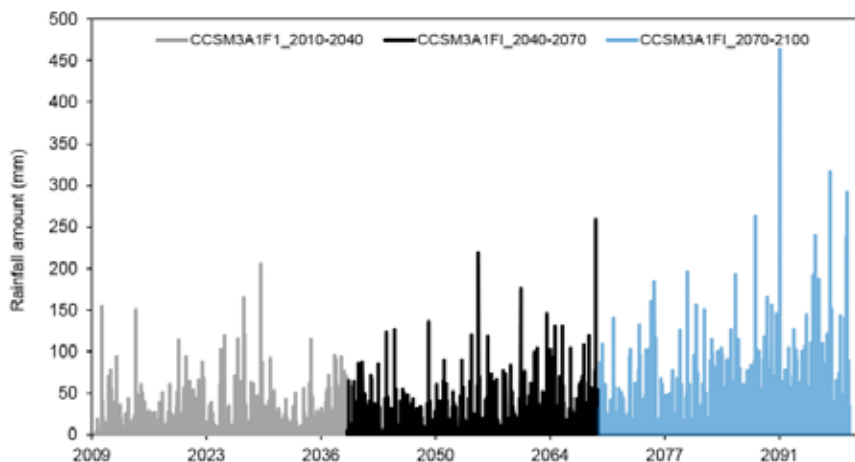


Figure 7. Daily rainfall pattern during future hydro-climate period

Under the present inflow, future climate projection during 2040-2070 will lead to a decrease in water level below the 100 m above sea level while future climate projection during 2070-2100 will lead to increase in water level exceeding 104.9 m. The simulated water level pattern during 2040-2050 and 2090-2100 is shown in Figure 8. The drop-in water level to 98 m above sea level which is equivalent to 80.7% of storage capacity. This value is still high compared to the lowest dam level recorded at 87.64m which was registered on 6 October 2005 (unpublished data, SAINS). Lower water level such as observed in 2005 and 2016 could be associated to lower rainfall within the catchment, higher evaporation rate in the lake and higher water abstraction. The varying changes in evaporation rate, which is associated to temperature, and changes in the amount of water transfer and intake need to be included to improve further the model.

Future climate change projection 2070-2100 suggest that frequent flood will occur especially during 2092-2100 resulting from simulated water level exceeding the spillway level of 103 m above sea level. The excess water can be tapped for future use and is recommended to be stored in bunded storage to avoid flood to downstream areas. This is consistent to an earlier study that predicts flooding as a major problem for the Sg Terip reservoir (NAHRIM 2014b). The earlier study which simulation was based on HEC-RESSIM model also reported that the reservoir will be free of drought, defined as the simulated water level exceeding the dead storage level (NAHRIM 2014b). This work assumes no changes in reservoir storage and catchment land use. The reservoir storage will change with time and land use due to deposition of sediments from eroded catchment areas. However, this effect was not considered in the analysis.

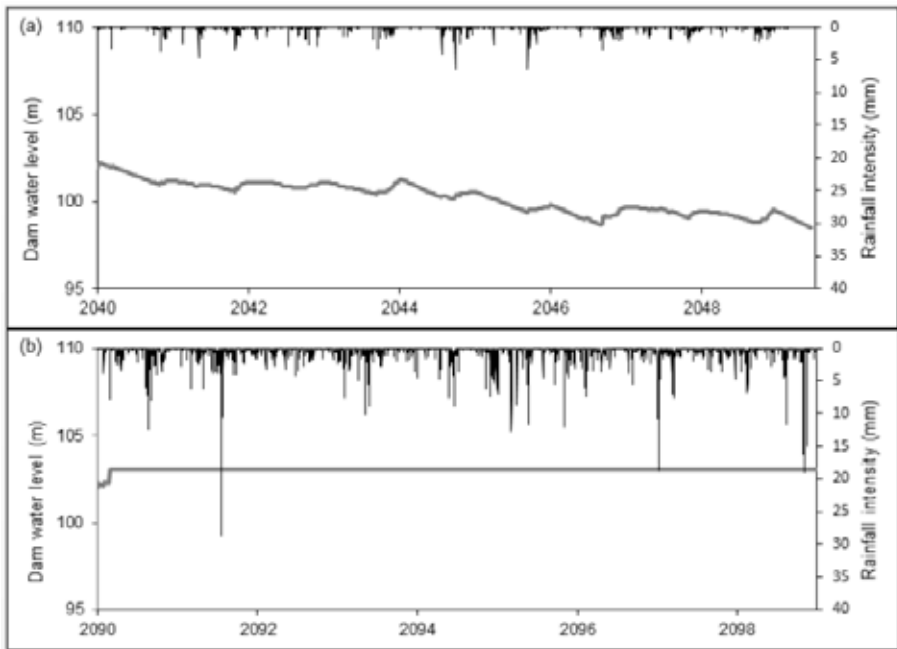


Figure 8. Rainfall pattern and dam water level during future hydro-climate (a) period 2040-2050 (b) period 2090-2100

4.3.2 Water quality assessment

Simulation results indicated that lake water quality will change only slightly with alteration in these future rainfall events due to low pollution sources. The water quality simulation shows small changes in water quality and pollutant transport due to very slow water movement and flow within the lake, and low pollution levels. Surface DO concentration remains above 7 mg/L under the future climate projection during 2040-2070 and 2070-2090. High DO is probably associated to the atmospheric flux and low oxygen demand due to moderate biological productivity. Higher rainfall amount under the climate projection during 2070-2100 may weaken potential intermittent stratification and improve the water quality through frequent flushing effects. Stratification dynamic, however, was not modelled in this study. Under the assumption of intact catchment, deterioration in water quality will remain small for this reservoir. Some studies reported that increased rainfall associated with climate change will result in increase in suspended and nutrient loadings (Riverson et al., 2013; Mooij et al., 2007). In this reservoir, water quality, however may be influenced by the quality of water that transfer into the waterbody.

5 CONCLUSIONS

Simulation results indicated that flood will occur resulting from excess in rainfall during the end of 21st century while drought will occur resulting from low rainfall amount during the mid of 21st century. Water quality simulation indicated slight change with alteration in these future rainfall events due to low pollution sources. Reservoir water quality may be impacted by the quality of water transferred into the waterbody. This study explores the prediction of the impact of extreme future hydroclimate projection on the condition of a water supply reservoir. However, the results of the study need to be verified further with flood rule curve and probable maximum precipitation/flood in order to enhance the model findings. Further testing of the model with other hydroclimate projection scenarios is also necessary and currently been undertaken to improve the model results and enhance the usefulness of the model for determining alternate management options.

ACKNOWLEDGMENTS

This project was funded by the Ministry of Natural Resources and Environment Malaysia through vote of Kajian Lanjutan Impak Perubahan Iklim Ke atas Sumber Air (vote No. P23170000190001). The first author would like to thank M. Azril Hilmi Shapiai and Mohd Hafiz Zulkifli for their technical support at field.

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RUNOFF SIMULATIONS OF MIXED LAND USE OF SKUDAI WATERSHED: SENSITIVE PARAMETERS

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ABSTRACT

For the first time, Hydrological Simulation Program-Fortran (HSPF) was used to study runoff simulation of mixed land use of Skudai watershed, the heaviest polluted river in the district of Johor, Malaysia. The aim of this study is to examine the sensitivity of HSPF parameters during runoff simulations for upstream of Skudai watershed. The model was calibrated using data from November 2013-February 2014 and validated with observed data from March-May 2012. Different land use types are shown to be sensitive to HSPF parameters especially to the index of the mean infiltration capacity of the soil followed by the fraction of groundwater inflow which will enter deep (inactive) groundwater, the upper zone nominal storage and the lower zone nominal storage. Statistical analysis comprising of coefficient of determination, Nash-Sutcliffe efficiency and the relative error were used as evaluation indicators between simulated and observed runoff. By examining the sensitivity analysis, it has been concluded INFILT is most sensitive to runoff followed by the DEEPPFR, UZSN and LZSN. Based on this study, HSPF is capable to simulate runoff using hourly time-step for mixed land use in Malaysia.

Keywords: HSPF, Skudai Watershed, Statistical Analysis, HSPF Parameters, Mixed Land Use.

INTRODUCTION

Nowadays, simulation models of watershed hydrology have become one of the principal aspects towards sustainable water resource development, planning, and management. These models are influenced by various factors such as spatial variability of soils, elevation, land used, weather and human activities. With rapid development in some watersheds, increasing pollution mainly from urban and agricultural lands has deteriorated the water quality in rivers. Hence, simulation models that combine the use of long-term continuous and storm event are needed in order to adequately manage watersheds and address water quantity and quality problems. Hydrological Simulation Program-Fortran (HSPF) is one of few models that combine the long-term continuous and storm event.

Hydrological Simulation Program-Fortran (HSPF) consists of sets of module that allow users to simulate complex and continuous storm events especially from non-point loadings such as agricultural and urban land activities. In order to achieve Best Management Practice (BMP), careful plan and accurate runoff models are needed to maintain agricultural productivity while minimizing adverse water quality effects. Besides, pollutants can be transported by runoff into the surface waters by infiltration (with deep percolation) into the groundwater. To enhance model predictions, suitable watershed models must be carefully selected. HSPF has been used widely due to HSPF capability to execute a continuous and short period simulation of less than a day prior to a flood event while generating excessive changes in river flow and water quality. Soil and Water Assessment Tool (SWAT) offers the same requirement but often used for rural areas that are dominated by agricultural applications. SWAT requires descriptive vegetative changes data and agricultural practices (Neitsch et al. 2002a)

HSPF has been widely used for watershed modelling especially in the hydrological simulation. Detailed simulation of 23 watersheds in the Patuxent River estuary found that annual water discharges from the watersheds increased relative to the proportion of developed land (Jordan et al., 2003). Zariello and Ries (2000) evaluated the effects of water withdrawals on flow in the Ipswich River Basins, Massachusetts. Singh (2004) and Diaz et al. (2011) utilized HSPF to simulate detailed hydrologic processes including pollutant fate and transport, as well as climate and land cover variability using water quality simulation models. Jing et al. (2009) used HSPF to improve the assessment of hydrologic activities in shallow ground water settings and Hayashi et al. (2004) simulates runoff and sediment load over a relatively short time interval for Upper Changjiang river basin in China. As an analytical tool, HSPF model subdivides watershed into smaller, more uniform pervious and impervious land segments based on land used types in the watershed. To develop User Control Input (UCI), Bicknell et al. (1996) has categorized HSPF model data requirement for users. To create UCI file, users must have three sets of data; spatially distributed data, environmental and meteorological monitoring data and point source data. HSPF model uses both daily and hourly meteorological data to generate outputs for continuous simulation. The present study aim to determine the most sensitive parameters associated with runoff simulation for mixed land use in Skudai watershed.

MATERIAL AND METHODS

The Skudai Watershed encompasses of 29,370 hectares (293.7 km²) of land area located at southern of Malaysia, in the state of Johor. The characteristics of the study site are summarized in Table 1. The land use in Skudai watershed are dominated by urban and agricultural. Figure 1 shows that Skudai River and its tributaries flow from Sedenak in Kluang District and drains into the Johore Straits in Johore Bahru District. Figure 2 shows variation of land use at the Skudai Watershed in percent. The Skudai River is subjected to seasonal fluctuation with maximum discharges usually occurred around November until March due to heavy rainfall during the North-East Monsoon.

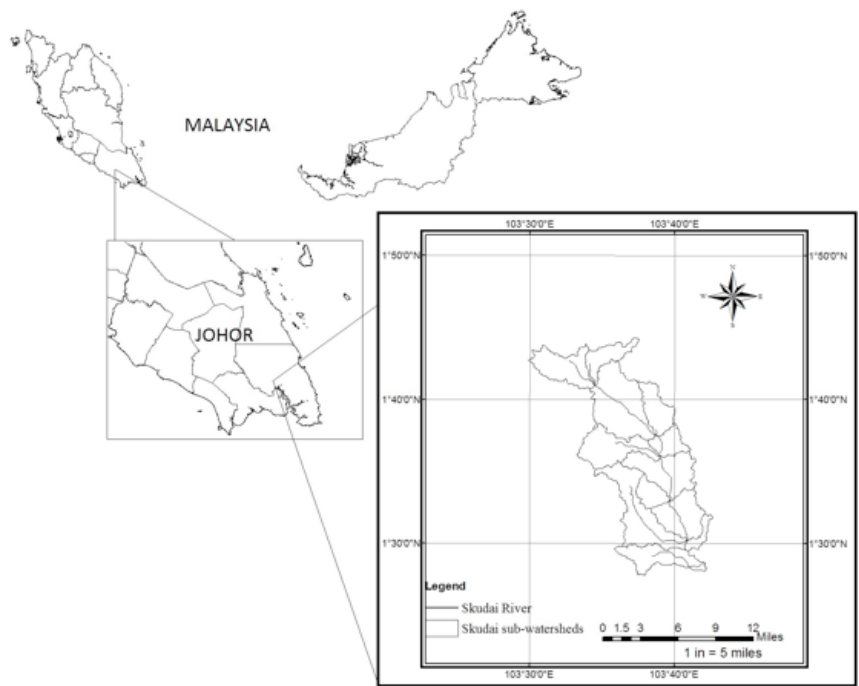


Figure 1: The Skudai river and its tributaries

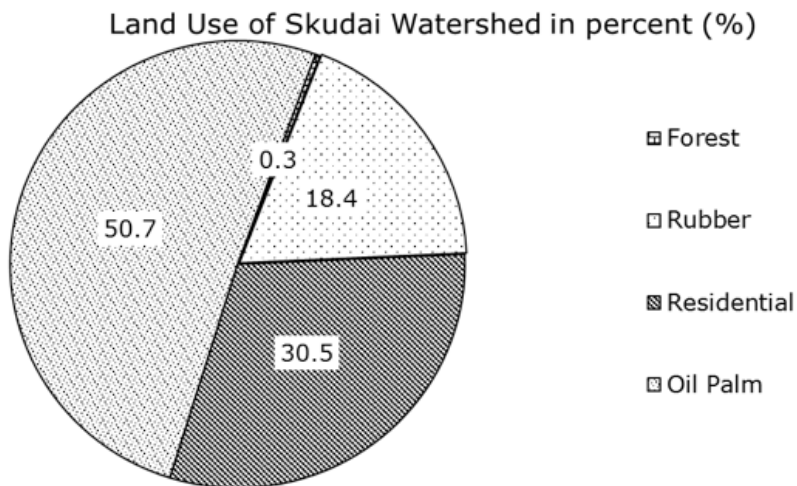


Figure 2: Land Use of Skudai Watershed
Table 1: Characteristics of the study site

Characteristics	Value
Annual Precipitation (mm)	2500
Channel Slope	1 : 204
Mean Temperature ($^{\circ}$ C)	27
Land Elevation (m)	0 to 500 m

Under current usage, Skudai Watershed is divided into pervious and impervious land segments. Pervious and impervious land segments consist of forestland (3.83%), agricultural land (39.76%), urban/residential land (52.46%) and barren land (3.96%).

THEORY AND CALCULATION

HSPF is a comprehensive water quality model that is able to simulate complex hydrological processes in comparison to simple method used in Export Coefficient Models. HSPF is designed with three different routines which are Pervious Land Segments (PERLND), Impervious Land Segments (IMPLND) and Reach Segment Operations (RCHRES). In HSPF, PERLND is the module that simulates the water quality and quantity processes which occur on a pervious land segment. The primary module sections in PERLND simulate snow accumulation and melt, water budget, sediment produced by land surface erosion and water quality constituents by various methods. Modules in IMPLND are similar to the PERLND module. However, the IMPLND sections are less complex, since they contain no infiltration function and consequently no subsurface flows. The operation module used to simulate Channel Reaches (referred to as RCHRES in the HSPF model) contains separate sections of code to simulate hydraulic behavior, pH, temperature and other water quality related processes. HSPF/BASINS used spatial variability of Skudai watershed to divide the basin into many hydrologically homogeneous land segments and simulating runoff for each segments independently. The HSPF needs precipitation, temperature and estimation of potential evapotranspiration input data to perform hydrologic simulation. Hourly precipitation and temperature input data were collected from Senai International Airport monitored by Malaysian Meteorological Department (MMD). Input data for potential evapotranspiration were estimated using maximum and minimum hourly temperature using Hamon PET equation (P. Hummel et al., 2001). These input data were disaggregated into hourly input data since the hydrological simulation was done using hourly time step.

The HSPF model for the Skudai Watershed were calibrated from 5/11/2013 to 5/2/2014 and then validated from 1/3/2012 to 15/5/2012. The watershed model was calibrated and validated at Department of Irrigation and Drainage Malaysia (DID) gauge station at Kampung Pertanian, Kulai Johor. The flow data was obtained from rating curve and from Automatic Global (WL16) Water Level Data Logger (Global Water, 2014) at Kampung Pertanian. The Rating Curve

for Skudai River was generated by using Automatic Sample Pump ISCO 6712 (Teledyne ISCO, 2012) that collected water level data every week. During high flows, the sample nozzle pump was submerged at the bottom of the river to determine an exact of total suspended solids had been collected. Subsequently, after sufficient data have been collected, linear correlation analysis between flow and total suspended solids were estimated.

The statistical analysis of hourly observed and simulated runoff were defined using linear regression (coefficient of determination, R^2), Nash-Sutcliffe efficiency (ENS) and the relative error (Dv). Nash-Sutcliffe efficiency is given as:

$$E_{NS} = 1 - \frac{\sum_{i=1}^N (O_i - S_i)^2}{\sum_{i=1}^N (O_i - O_{Avg})^2} \quad (\text{Equation 1})$$

where N is the number of observations during the simulated period, O_i and S_i are the observed and simulated values at each differences point i and O_{Avg} is the mean of the observed values.

ENS ranges from negative to infinity, with 1 denotes a perfect agreement with observed data. Nash-Sutcliffe efficiency has been used in many researches to determine model evaluation and considered as one of the best statistics for evaluation of continuous hydrograph simulation programs (ASCE, 1993)

The coefficient of determination, denoted R^2 and pronounced R squared, is a number that indicates how well data fit a statistical model. It ranges from zero to 1, with 1 is the perfect agreement between simulated and observed data. R^2 is given by:

$$R^2 = \left\{ \frac{\sum_{j=1}^n (O_j - \bar{O})(S_j - \bar{S})}{\left[\sum_{j=1}^n (O_j - \bar{O})^2 \right]^{0.5} \left[\sum_{j=1}^n (S_j - \bar{S})^2 \right]^{0.5}} \right\}^2 \quad (\text{Equation 2})$$

Where O_j is the observed flow at time step j, $\bar{O}$ is the average observed flow during the simulation period, S_j is the model-simulated flow at time step j and $\bar{S}$ is the average simulated flow at time step j.

The relative error (Dv) is calculated as:

$$D_v(\%) = \frac{X_m - X_s}{X_m} \times 100 \quad (\text{Equation 3})$$

Where X_m is the observed total runoff volume and X_s is the model-simulated total runoff. The smaller the number of Dv gives the better result for simulation and observation data. For a perfect model, Dv would be equal to zero.

RESULTS

The HSPF uses several parameters to select sensitivity analysis for hydrologic simulation which can be defined in EPA BASINS Technical Note 6 (USEPA, 2000). The time step of simulation used was 1 hour and was carried out within less than 3 months. Modelled parameters affect both water balance and flow by distributing capacity of water between surface runoff, interflow, baseflow and deep groundwater. Table 2 summarizes the final values of hydrologic pervious land parameters and Table 3 summarizes values of hydrologic impervious land parameters that were adjusted during calibration and validation process. These values fall within the range of values as stated at typical range below.

Table 2: Parameter values for hydrologic simulation for the Skudai river watershed (Pervious Land)

	Parameter	Calibrated Value	Typical Range
LZSN	The lower zone nominal storage	6.2	3.0-8.0
INFILT	The index to the mean infiltration capacity of the soil	0.24	0.01-0.25
LSUR	The length of the assumed overland flow plane	600	200-500
SLSUR	The slope of the overland flow plane	0.078-0.09	0.01-0.15
KVARY	The parameter which affects the behavior of groundwater recession flow	1	0.0-3.0
AGWRC	The basic groundwater recession rate if KVARY is zero and there is no inflow to groundwater	0.93-0.99	0.92-0.99
INFEXP	The exponent in the infiltration equation	2	2
INFILD	The ratio between the maximum and mean infiltration capacities over the pervious land segments	2	2
DEEPPFR	The fraction of groundwater inflow which will enter deep (inactive) groundwater	0.2	0.0-0.2
BASETP	The fraction of remaining potential E-T	0.05	0.0-0.05
AGWETP	The fraction of remaining potential E-T	0.05	0.0-0.05

CEPSC	The interception storage capacity	0.25	0.03-0.2
UZSN	The upper zone nominal storage	0.55	0.1-1.0
NSUR	The Manning's n for the overland flow plane	0.33	0.15-0.35
INTFW	The interflow inflow parameter	3	1.0-3.0
IRC	The interflow recession parameter	0.5	0.3-0.85
LZETP	The lower zone E-T parameter	0.5	0.5-0.7
CEPS	The initial interception storage	4	0.0-100.0
SURS	The initial surface (overland flow) storage	5	0.0-100.0
UZS	The initial upper zone storage	0.1	0.0-100.0
IFWS	The initial interflow storage	0.01	0.0-100.0
LZS	The initial lower zone storage	1	0.0-100.0
AGWS	The initial active groundwater storage	0.01	0.0-100.0
GWVS	The initial index to groundwater slope	0.01	0.0-100.0

Table 3: Parameter values for hydrologic simulation for the Skudai river watershed (Impervious Land)

Parameter		Calibrated Value	Possible Range
LSUR	The length of the assumed overland flow plane	250	50.0-250.0
SLSUR	The slope of the assumed overland flow plane	0.001	0.001-0.15
NSUR	The Manning's n for the overland flow plane	0.15	0.01-0.15
RETSC	The retention (interception) storage capacity of the surface	0.13	0.01-0.3

DISCUSSION

Sensitivity analysis

The sensitivity analysis was carried out by comparing monthly observed and simulated flow. The most sensitive factors governing simulated river flow for Skudai river were LZSN, UZSN, DEEPFR and INFILT. These indicate that baseflow component is very significant for the river flow. LZSN have a major effect in the routing of the flow in Skudai river and frequently adjusted when calibrating HSPF. In the present

study, after the initial calibration, LZSN was set at 0.16 m and held constant for all land uses. The upper zone nominal storage UZSN, a user specified parameter was set according to EPA BASINS Technical Note 6 (USEPA, 2000). Donigian et al. (1978) assumes UZSN as 0.06 of LZSN for steep slopes, limited vegetation, low depression storage; 0.08 of LZSN for moderate slopes, moderate vegetation, and moderate depression storage; 0.14 LZSN for heavy vegetal or forest cover, soils subject to cracking, high depression storage and very mild slopes and final value is set to be 0.55 inch. For Skudai Watershed, 0.08 of LZSN is used since it has moderate slope (0.5%) and moderate vegetation and depression storage. Since surface runoff and baseflow indicate direct contribution of water quality simulation, the fraction of groundwater inflow DEEPFR was set at 0.2 to ensure model stability. For the mean soil infiltration rate parameter, INFILT possible value was set to be 0.24 in/hr and may decrease to produce more upper zone and interflow storage water. Excessive value of INFILT can lead to a greater overland flow and interflow in Skudai river. To have a better picture of these four parameters sensitivity, Table 4 summarizes the detail analysis of these assessment.

Table 4: Parameters sensitivity analysis for Skudai River

Parameters	unit in (inch)	Runoff in (%)
INFILT	Increased 0.05	Increased 6.78
DEEPFR	Increased 0.1	Decreased 14.96
UZSN	Increased 0.3	Decreased 20.52
LZSN	Decreased 1.0	Increased 5.94

Based on the sensitivity analysis performed, it is concluded that runoff volume was highly sensitive to INFILT then to DEEPFR, UZSN and LZSN. High INFILT caused less water to flow out as runoff and low DEEPFR increased runoff, resulting reduced fraction of groundwater inflow.

Model Calibration

Calibration of the model was conducted between simulated and observed flow monitoring data from Kampung Pertanian station. During the calibration process, each of parameters were adjusted to match the observed and simulated flows. Initial value parameters were adjusted one by one within typical range and keeping others constant. The simulated output values were then compared with the initial values to check its sensitivity with total runoff volumes. A time-series plot of the observed and simulated hourly flows (cfs) in Figure 3 shows simulated monthly yields generally were within 10.09% of recorded values, although there were some unbalanced between first and third months. As shown in Table 5, simulated runoff volumes of November are consistently underestimated compared to observed runoff volumes may be due to low rainfall at Kampung Pertanian gauging station. The coefficient of determination (R^2) indicates that the model gives almost 74% of total variability in the observed data at an hourly

level which is considered as fair. At the daily level, the model correlates well with a coefficient of determination of 83%. NS values for calibration were 0.3 for hourly and 0.26 for the daily interval. In Kampung Pertanian study, HSPF underestimated hourly and daily flows by 77% for the relative error. It can be seen that the error in Table 6 may also be attributed to less detailed hydrological survey at this area (Tetrattech,2009).

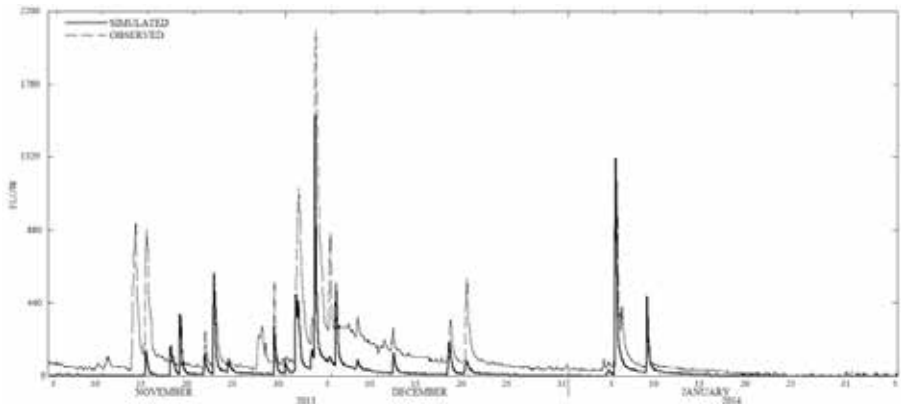


Figure 3: Observed and calibrated hourly streamflow at Kampung Pertanian, Kulai

Table 5: Observed and simulated hourly flow and percentages

Month	Observed (Acre-ft)	Simulated (Acre-ft)	Percentage (%)
November	10463.42	4863	-115.16
December	8093.69	10700	24.36
January	65.55	5149	98.73
Total	18622.66	20712	10.09

Table 6: Summary HSPF statistical analysis model results for calibration period

Statistical	Hourly	Daily
R2	0.74	0.83
ENS	0.3	0.26
Relative Error (%)	76.82	77.09

Model Validation

The calibrated HSPF model was validated from March 2012 to April 2012 and

the idea behind the selection of these time periods for validation was to test under lower rainfall months. The results of model validation for hourly runoff are presented in Figure 4. Although these periods were relatively wet as compared to the calibration period, the simulated runoff volume showed a moderate response when attained close to observed runoff pattern. However, the model underestimates runoff in the initial phase owing to drier soil and more infiltration but the model response changes during April when continuous rainfall was recorded. During April, validated runoff values matched well with observed and this could be attributed to unpredictable rainfall recorded in Kampung Pertanian gauging station. The observed and simulated runoff volumes (cfs) for validation is summarized in Table 7. The differences between observed and simulated runoff in both months may be due to the location of rainfall gauge. In general R2 and Ns values are fair for hourly intervals and good to very good for daily intervals as describe in Table 8. Statistical test shows R2 of 0.83 and 0.88 for hourly and daily respectively whereas NS values indicate 0.63 and 0.61 for hourly to daily respectively.

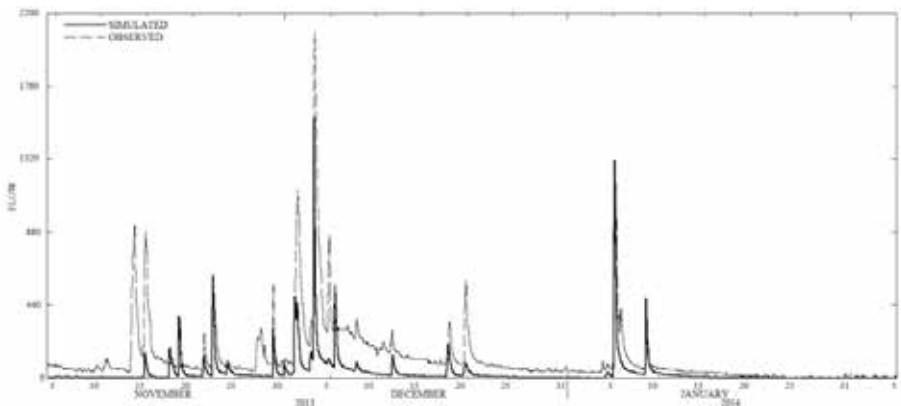


Figure 4: Observed and validated hourly streamflow at Kampung Pertanian, Kulai

Table 7: Observed and simulated hourly streamflow and percentages

Month	Observed (Acre-ft)	Simulated (Acre-ft)	Percentage (%)
March	2432.81	4870	50.04
April	5990	13510	55.66
Total	8422.81	18380	54.17

Table 8: Summary HSPF statistical analysis model results for validation period

Statistical	Hourly	Daily
R2	0.83	0.88
ENS	0.63	0.61
Relative Error (%)	28.03	29.98

CONCLUSION

The HSPF model predictions compare fairly close to observed measurements during wet and dry months and produce a working best-fit set of model parameters within acceptable ranges given from EPA technical note for runoff simulation. By examining the sensitivity analysis, it has been concluded DEEPR is most sensitive to runoff followed by the INFILT, UZSN and LZSN. Approximation sensitivity parameters from this study would be used in future applications of the model under similar types of watershed studies. The study demonstrated that the HSPF model is capable for evaluating hydrology processes in the tropical region of Malaysia on hourly time basis.

ACKNOWLEDGEMENT

The authors would like to acknowledge technicians and support groups from Faculty of Civil Engineering, Universiti Teknologi Malaysia for their support and co-operation and late Dr.NoorBaharim Hashim for his research grant.

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FLOOD MITIGATION MEASURES TOWARDS ACHIEVING ZERO FLOOD FOR SUNGAI KERAYONG CATCHMENT, SUNGAI KLANG RIVER BASIN

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ABSTRACT

Flash floods have caused a lot of distress to the people living in Klang River Basin; from long hours of traffic jam congestion, damages of cars, to the destruction of homes and business properties. Comprehensive flood mitigation measures have been implemented by the authority. However, the effectiveness of the flood mitigation measures are still being questioned since floods still occurs. Thus, this research aimed to analyse, evaluate, predict and provide recommendations for flood mitigation measures towards achieving zero flood for Sg Kerayong river basin, a subcatchment of Klang River Basin. This research covers the development of a hydrodynamic model which is capable to simulate the integrated hydrologic and hydrodynamic analysis, production of flood forecasting maps in real-time simulation which in later stage to serve as an important data for flood relief operations as well to propose environmental-friendly soft-structures with appropriate flood mitigation system. Flood maps for three different rainfall design (50-year, 100-year and Probable Maximum Precipitation (PMP) events) were produced using river modelling - TREX. This study found that Kerayong River Basin has undergone quite comprehensive flood mitigation measures which include ponds and river rehabilitation. The model simulations indicate that the mitigation measures are satisfactory to cater for the 50-year and 100-year design rainfall. However, about 7% of the catchment areas are predicted to be flooded if PMP storm events occur. If rainwater harvesting is implementing in these area, then peak discharge can be reduced by 39% and a reduction of 50% in peak discharge can be achieved if new material such as porous rock matrix filter are used for open spaces such as parking or pavement.

Keywords: Flood, River Modelling, Flood Mitigation, Rainwater Harvesting, Porous Pavement

1 INTRODUCTION

Flooding is the most frequent and disastrous weather phenomenon for Malaysia ever since long time ago. The main cause of severe flooding in Malaysia is the heavy monsoon or convective rainfall. Flash floods have caused a lot of distress to the people living in Klang River Basin; from long hours of traffic jam congestion, damages of cars, to the destruction of homes and business properties. Comprehensive flood mitigation measures have been implemented by the authority including enforcing storage ponds for construction sites, improving river channel sections and building the SMART tunnel. However, the effectiveness of the flood mitigation measures are still being questioned since floods still occurs. A thorough investigation with accurate and up-to-date analysis and hydrodynamic modelling will help to gauge how effective are the flood mitigation measures at Klang River Basin and to provide recommendation towards achieving zero flood. This research focused on implementation, types, maintenance and effectiveness of flood mitigation structures in Sg Kerayong river basin by developing a hydrodynamic model to simulate the integrated hydrologic and hydrodynamic analysis. Flood maps for three different rainfall design (50-year, 100-year and Probable Maximum Precipitation (PMP) events) were produced using river modelling –Two-Dimensional Runoff Erosion and Export (TREX).

2 STUDY AREA

Sg Kerayong river basin is located at the Federal Territory of Kuala Lumpur and is considered to be one of the important branches of the main Klang River. Figure 1 shows the location of Sungai Kerayong river basin and its land use details. Most of the areas have been developed as residential and industrial areas. The width and the length of the river are 20 m and 30 km respectively. The catchment area is approximately 55 km². The lowest and highest elevation is 33 m and 400 m respectively.

The study site experiences the tropical rainforest climate, which is warm and sunny throughout the year. It receives more rainfall especially during northeast monsoon season between October to March. The average annual rainfall ranges from 2,500 mm to 3,000 mm. The maximum recorded rainfall for 1-hour duration is 94.5 mm on June 1, 2010 Department of Irrigation and Drainage (DID), 2014 based on the recorded data from 2007 to 2010 (DID, 2014).

Several factors have been analyzed by choosing Sg Kerayong as case study area. For instance, it was recorded some of the worst flooding events at Sg Kerayong in the past few years and the impervious surface of the area had increased more than 95%.

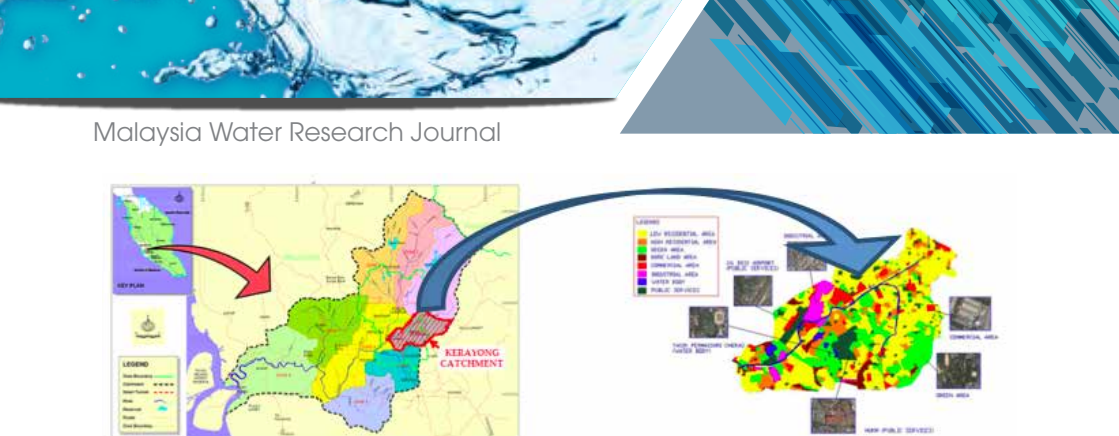


Figure 1. The location of Sg Kerayong Catchment Area and Landuse Details

3 METHODOLOGY OF STUDY

3.1 Data Collection and Analysis

The overall methodology of this research includes the desktop study, data collection from the relevant agencies and the development of hydrodynamic modelling and flood forecasting mapping. Figure 2 shows the overall methodology.

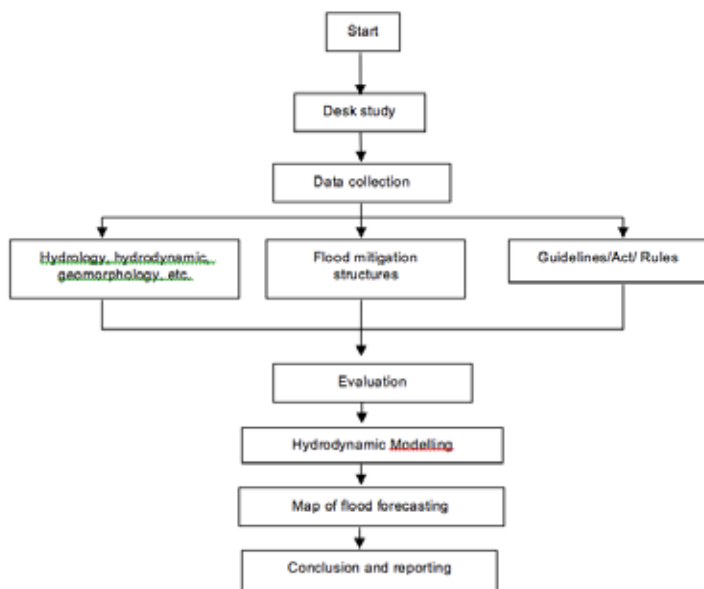


Figure 2. Overall research methodology

3.2 Model Set-Up

3.2.1 TREX Hydrodynamic Modelling

Two-dimensional spatially distributed TREX model is used to simulate the relationship between rainfall and runoff. There are four (4) main hydrologic processes included in the TREX model: (1) rainfall and interception, (2) infiltration and transmission loss, (3) storage, and (4) overland and channel flows.

There were eleven (11) layers prepared using ArcGIS software. These layers had been prepared from four (4) basic data, i.e. Digital Elevation Model (DEM), soil type, land use and river flow direction. Figure 3 shows TREX hydrodynamic modelling for Kerayong River basin. The DEM was obtained from the DID with a resolution of 50 m. Data for the channel flow and land use were obtained from DID and Department of Agriculture (DOA) respectively. The soil type is assumed to be limestone based on the geological map available from SMART website (DID, 2014).

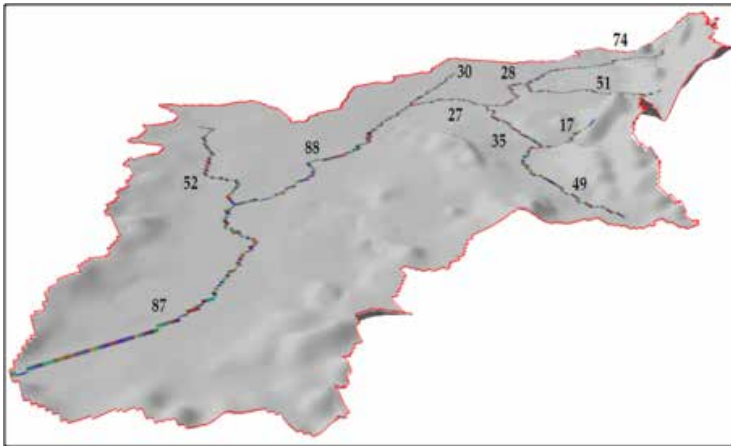


Figure 3. TREX hydrodynamic modelling

The input data had to be tested by applying the impervious condition. The hydraulic conductivity value of soil was set to zero. The rainfall intensity of 50 mm/hr was chosen. The rational method with runoff coefficient (C) equal to one (1) was used to compare the discrepancy between these two values (i.e. rational method and TREX model). The application of rational method is valid provided the hydrological simulations are: (1) the peak flow is reached when the entire watershed is contributing to the runoff, and (2) the rainfall intensity is assumed to be uniform across the watershed and over the duration of rainfall.

The Relative Percentage Difference (RPD) and Percent BIAS (PBIAS) methods are used for the statistical analysis to check on the model performance. The RPD is the simplest statistical method among others used to calculate the differences between observed and simulated peak discharge, total volume and time to

peak (Singh et al., 2005; Fernandez et al., 2005), while the PBIAS method is a statistical error analysis that measures the average tendency of the simulated results to underestimate or overestimate the observed data (Gupta et al., 1999). The summary of model performance evaluation for RPD and PBIAS methods are tabulated and shows in Table 1.

Table 1. General performance ratings to classify the performance of the model

Performance Rating	RPD and PBIAS
Very Good	RPD, PBIAS $\leq \pm 10\%$
Good	$\pm 10\% < \text{RPD, PBIAS} \leq \pm 15\%$
Fair / Satisfactory	$\pm 15\% < \text{RPD, PBIAS} \leq \pm 25\%$

3.3 Calibration and Validation

The calibration and validation procedure focused on the accuracy of simulated peak discharge, total volume and time to peak at the main outlet (i.e., at the point-end-downstream of the river/channel). After the calibration process, another storm event was used for the validation purpose.

Based on the study conducted by Abdullah (2013), Velleux et al. (2006) and Velleux et al. (2008), the typical hydrologic sub model parameters subjected to be calibrated are hydraulic conductivity (Kh) and flow resistance coefficient (Manning's n). They also added that other hydrologic sub model parameters, such as interception, hydraulic suction head and dead storage have less significant in affecting the discharge magnitude.

During the validation processes, the rainfall-runoff relationship was simulated using calibrated parameters (Kh and n) without any changes. Before the validation process is made, the grid size analysis was conducted (this process is parallelly conducted during calibration). The main purpose of this process is to determine the most suitable grid size that can be used for further analyses. Four (4) different grid sizes were used; 90 m, 150 m, 210 m and 270 m. Storm event on April 2, 2008 was used for the calibration and grid size analysis purposes. Total rainfall for this event is 113 mm which occur for 3-hour of rainfall duration. This is the highest recorded rainfall and discharge. The calibration and validation procedure focused on the accuracy of simulated peak discharge and time to peak at the main outlet (i.e., at the point-end-downstream of the channel).

Tables 2 and 3 show the summary of the calibration (and validation) and grid sizes analysis, respectively. From the calibration analysis, the goodness-of-fit-test (GoFT) indicated that all grid sizes can reproduce the storm event on April 2, 2008 with an error of less than 10% based on the RPD matric. The time to peaks were estimated with the highest lag time of 15 minutes. The lag value, for small catchment (definition of small catchment is less than 100 km² as described by Singh (1995) and used by Abdullah (2013)), is still acceptable.

Table 2. Performance of the TREX model during calibration and validation processes

CALIBRATION							
Grid Size (m)	Storm event	Q _{peak} (m ³ /s)			T _{peak} (Hour)		
		Obs.	Sim.	RPD (%)	Obs.	Sim.	Lag
90	02-Apr-08	184.4	175.9	4.6	19:00	19:06	0:06
150			201.3	-9.1		19:15	0:15
210			203.7	-10.4		18:48	0:12
270			164.8	10.6		18:51	0:09
VALIDATION							
Grid Size (m)	Storm event	Q _{peak} (m ³ /s)			T _{peak} (Hour)		
		Obs.	Sim.	RPD (%)	Obs.	Sim.	Lag
90	16-Apr-08	68.9	55.0	20.1	17:00	17:15	0:15
150			56.9	17.4		18:45	1:45
210			41.8	39.3		18:45	1:45
270			72.3	-5.0		17:09	0:09
90	27-Aug-08	97.2	61.9	36.3	19:00	20:12	1:12
150			73.2	24.7		21:00	2:00
210			95.8	1.4		18:00	1:00
270			87.9	9.5		18:06	0:54
90	14-Dec-08	36.1	42.1	-16.5	15:00	15:48	0:48
150			37.7	-4.4		16:15	1:15
210			21.6	40.2		17:06	2:06
270			55.8	-54.5		15:09	0:09
90	26-Feb-09	91.4	78.7	13.9	16:00	16:02	0:02
150			86.5	5.3		16:06	0:06
210			68.7	24.8		17:12	1:12
270			120.0	-31.3		16:00	0:00
90	24-Mar-09	54.1	74.4	-37.6	09:00	09:02	0:02
150			80.2	-48.3		10:02	1:02
210			71.6	-32.4		09:32	0:32
270			104.2	-92.7		09:00	0:00

Table 3. Grid size analysis for Kerayong River Basin for storm event on April 2, 2008

GRID SIZE (m)	Simulation Time		Q _{peak} (m ³ /s)			Time to Peak (Hrs.)			Volume (m ³)		
	Secs.	Mins.	Sim.	Obs.	RPD(%)	Sim.	Obs.	Lag time	Sim.	Obs.	PBIAS
90	3,708	60	176	184	-4.4	20:06	19:00	0:06	669,879	731,985	8.5
150	160	2.68	201		9.4	20:15		0:15	765,120		-4.5
210	222	3.7	204		10.7	19:48		0:12	752,017		-2.7
270	45	0.75	165		-10.4	19:51		0:09	1,048,352		-43.2

Table 3 shows the simulation time for smallest grid size is about an hour and decreasing for coarser grid sizes. The simulation time for the calibration is less than five minutes for grid size more than 150 m. Most of the grid sizes overestimated the volume of water except for smallest grid size, which is underestimated by 8%. However, the simulated volume using 270 m grid size is unacceptable which the difference between simulated and observed is more than satisfactory criteria, i.e. 25%. From this study, the conclusion can be made that the 90 m grid size is suitable for flood map and 150 m can be used for real time simulation. The model performance reached at least the satisfactory criteria in replicating these storm events (i.e. peak discharge and time to peak) when 90 m grid size is used. For the 150 m grid size, the model estimated time to peak with 1 hour lag time after the observed. Overestimated the peak discharge can be found for 270 m grid size. Similar conclusion was also been made by Abdullah and Julien (2014). They concluded that for a small watershed (less than 100 km²), the appropriate grid can be used to represent the water depth distribution is less than 90 m of grid. Figure 4 shows the distribution of water depth for storm event on April 2, 2008 at different grid sizes.

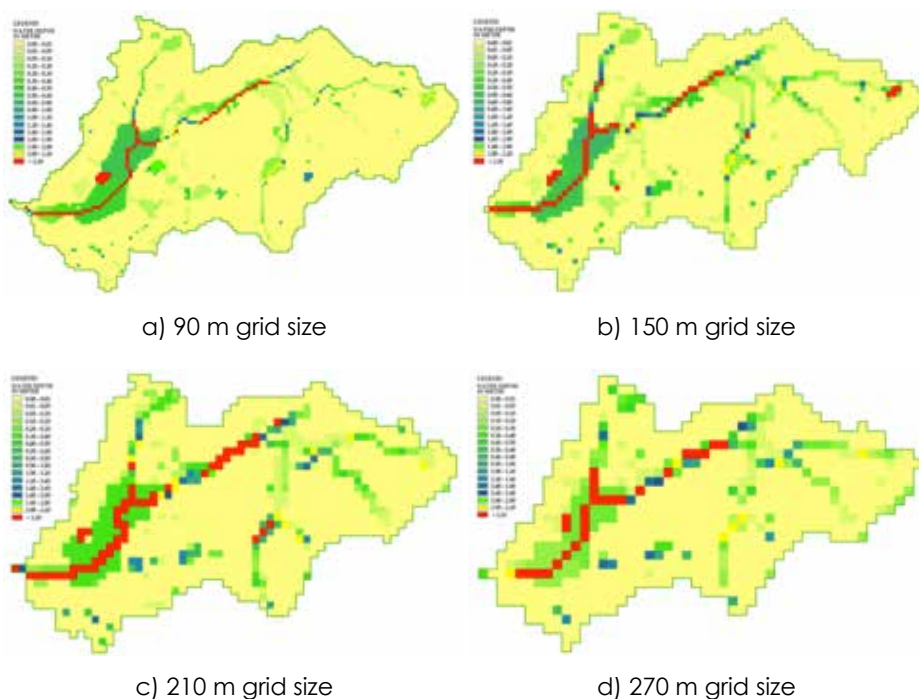


Figure 4. Distribution of water depth for storm event on April 2, 2008 at different grid sizes

4 RESULT AND DISCUSSION

4.1 Flood Map

The flood maps were produced based on the 50-year, 100-year and Probable Maximum Precipitation (PMP) data. The objective is to produce flood forecasting map using real-time simulation that can evaluate and assess the flood damage caused by flooding. These rainfall events occurred for two (2) hours duration. The rainfall intensities for 50- and 100-year are obtained from DID (2012) and PMP is obtained from NAHRIM (2008). Kirpich's formula is used to calculate the rainfall duration.

Table 4. Rainfall Intensity for 50-, 100-year and PMP

	Rainfall Intensity (mm/hr)	Maximum discharge (m ³ /s)
50-year	56	185
100-year	62	238
PMP	288	3,786

According to Abdullah (2013), the discharge magnitude of PMP event is between 6 and 15 times larger than discharge for 100-year Average Recurrence Interval (ARI). From this study, the estimated PMP peak discharge is 15 times larger than peak discharge for 100-year ARI. The maximum water depth is 0.3 meter which recorded for 50- and 100-year ARI. The water depth more than 1.0 meter can be found only in the channels.

The 50-year ARI storm event is similar to the calibration storm event (refer to Table 2). An additional of total rainfall by 10 mm does not change much on the water depth distribution. The existing of two ponds gives lot of advantage in ponding excess water from overtopping the channel. The flood prone area is increased when high rainfall intensity was used. The 288 mm/hr of rainfall intensity was simulated to estimate any possible flood area for the PMP storm event. Low-lying area which has an elevation less than 35 m were found prone to be flooded (Figure 5c). The grid size less than 150 m is recommended for flood mapping purpose so as more accurate representation of the actual scenario can be obtained.

For the purpose of real time simulation to obtain faster result, to be used for flood forecasting for example, grid size between 150 and 210 m is recommended. Most of the low-lying areas are covered by houses and business center. Using the design rainfall of 50- and 100-year return period as the input to the model, the flood maps produced were analysed and were found that none of the area was seriously inundated. It can be concluded that the existing flood mitigation system (i.e. Taman Seri Desa and Sri Johor Ponds) are reliable to reduce the magnitude of flood. The maximum rainfall intensity of these cases is 62 mm/hr. However, the overland water depth estimated had reached beyond 2.2 m when the rainfall intensity was increased by 5 times from 62 mm/hr to 288 mm/hr (PMP storm event). The flooded area was estimated to be 7% (approximately 4.0 km²)

which occurred at low land as shown in Figure 5 (c). The flood map can be used to assess flood damages knowing the values of the infrastructure and properties in the inundated areas.

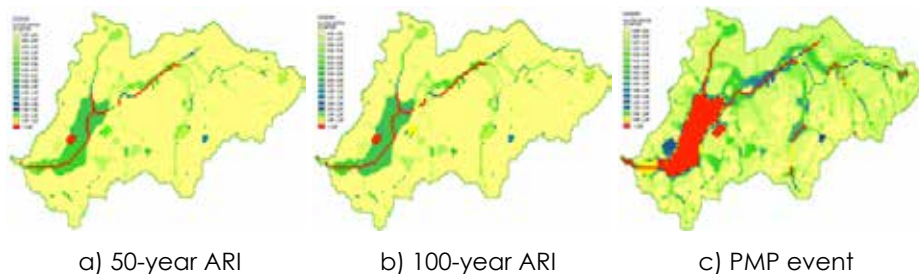


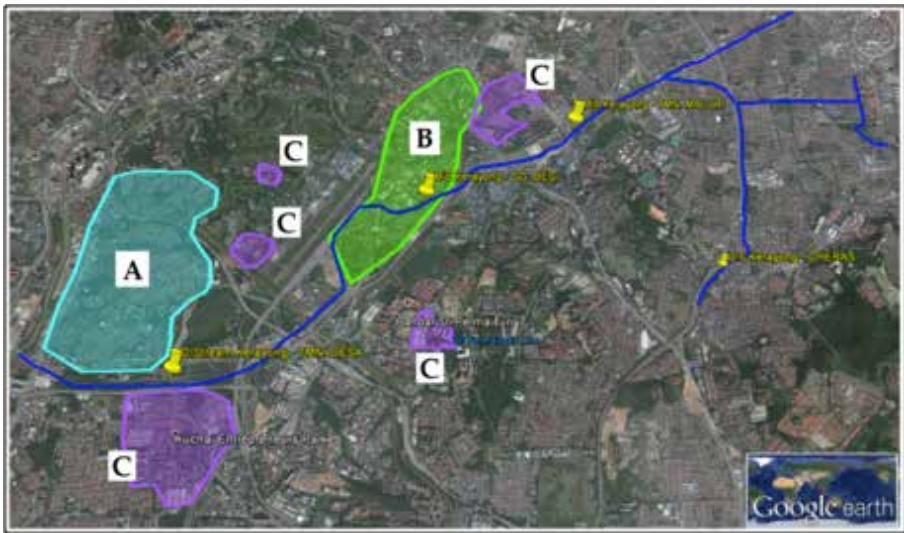
Figure 5. Water depth distribution for 50-, 100-year and PMP event

4.2 Environmental Friendly Soft Structures For Flood Mitigation System

Flood protection measures can be structural or non-structural (soft) such as source control, zoning, flood proofing, insurance and flood forecasting system (Menzel & Kundzewicz, 2003). Watershed management which focuses on the source control is a concept used in minimising the surface runoff, erosion and sediment transport. It includes vegetation cover management in which the concept of 'catching water where it falls' by enhancing storage of water on the land surface, is applied.

Another method used is green infrastructure such as installations of distributed stormwater controls such as porous pavement, bio-retention structures, green and blue roofs, infiltration systems and rain water harvesting system.

There are two different approaches considered in this study: 1) introducing more pervious and higher roughness surface materials and 2) installing rainwater harvesting system at residential and businesses areas. For the first approach, the 100-year storm event was used to study the effectiveness of this approach. A selected area which covers around 7% of Kerayong Catchment located at the flood prone area were replaced with more pervious surfaces with high roughness values. The simulated value of peak discharges was found to be reduced to 117 m³/s from the original value of 283 m³/s. The results show that the peak discharge can be decreased by 50% from the existing condition if there are more pervious surfaces with higher infiltration rate installed, especially at the upstream of the catchment. For the second approach, the alternative to reduce runoff is by storing the rainwater through rainwater harvesting concept. Figure 6 shows the potential residential and business area for installation of rainwater harvesting system.



Polygon	Area (km2)
A (Residential Area)	3.163
B (Residential and Industrial Areas)	2.158
C (Industrial Area)	$0.45+0.13+1.04+0.14+0.06=1.82$

Figure 6. Potential location for installation of rainwater harvesting

Table 5 shows the estimation of harvested water based on four scenarios for the location of rainwater harvesting. The simulation shows that 39% of peak discharge can be reduced if a rainwater harvesting systems is installed at various selected residential and businesses areas.

Table 5. Estimation of harvested water for rainwater harvesting

No	Scenario	Harvested area (km2)	Harvested water (m3/s)	Percent of reduction for peak discharge (%)
1	Installing rainwater harvesting at area A only	3.16	41	17
2	Installing rainwater harvesting at area B only	2.16	28	12

3	Installing rainwater harvesting at area C only	1.82	24	10
4	Installing rainwater harvesting at area A, B and C	7.14	92	39

5 CONCLUSION

It can be concluded that, as long as the flood mitigation measures at Sg Kerayong such as concrete channelization, rehabilitation with good floodplain, retention ponds are well maintained, there will be a reduction in flood risk. Many curative actions have been taken by related agencies in reducing the impact of flood in Kerayong River Basin. Hydrodynamic modelling using TREX is able to simulate the relationship between rainfall and runoff, and further to produce flood maps. Soft structure such as introducing pervious surfaces with higher roughness materials were installed in upstream of catchment, which could help in reducing peak discharging up to 50%. While, through installation of rainwater harvesting at all the residential and businesses areas, approximately 39% of peak discharge can be reduced. All these measures are suggested to be implemented especially in urban area and flood prone area like Kerayong River Basin which could help in achieving towards zero floods.

ACKNOWLEDGEMENTS

NAHRIM are glad that this project towards zero flood for Kerayong River Basin was successfully completed within a year. The successful completion of this project is made possible from the commitment and cooperation by various parties. NAHRIM also are thankful to those who have involved and commented for the improvement of the overall projects output.

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RIVERBANK FILTRATION (RBF) INDEX AND EFFECTIVENESS OF RBF AT WEST MALAYSIA

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ABSTRACT

Drinking water supply in Malaysia is based on surface water abstraction (98%) and groundwater (2%). As Malaysia is currently facing problems with climate change and the pollution of surface water by industrial, agricultural and municipal inflows, riverbank filtration (RBF) would offer in situ water treatment process and a low cost alternative for pre-treatment of raw water for potable use. Many important factors affect the site selection for RBF works such as groundwater and surface water quantity, current water quality situation, hydraulic interaction degree and exchange relationship between groundwater and surface water. In this research, a multi-criteria index system was developed for assessment of RBF site suitability. The RBF index system was based on a detailed analysis of physical geography and geological and hydrogeological conditions (which were considered to be the main influential factors of RBF), as well as on the development and utilization of water resources and water demand. Value index criteria based on specialist marking methods were used to determine weighting coefficients and weighted scores. The evaluation method was selected to use at RBF sites in Langat River, Linggi River and Muda River, Malaysia and integrated will put into the spatial analysis features of GIS to determine the distribution of suitable areas for RBF works. However, this research still has some limitations that must be mentioned here and will be studied further.

Keywords: RBF index, Bank filtration share, Riverbank filtration, Water quality of bank filtration, Enhancement of river and groundwater quality.

1 INTRODUCTION

El Nino phenomenon will affect the water resource in Malaysia, with its stable water quantity and quality and easy access of conjunctive use surfacewater and groundwater management, riverbank filtration (RBF) is suitable to adopt in this countries and an important of groundwater exploitation. The design and concept of RBF was common in Europe since 1870 (Schubert, 2002). In Switzerland, France, Finland, Hungary, Slovakia, Germany and Holland, the proportion of

total drinking water supplies by RBF water supply has reached 80%, 50%, 48%, 45%, 50%, 16% and 5% respectively (Tufenkji et al. 2002, Hiscock and Grischek, 2002). The conjunctive use of surfacewater and groundwater resources can be using through RBF, which stimulate and increase the recharge of river water to the aquifer (Shamsuddin et al, 2014). RBF is the infiltration of river water into a well through the riverbed and underlying aquifer material such as sand. This is a natural filtration process in which physic-chemical and biological processes play a role in improving the quality of percolating water. After a certain zone of mixing and reducing, the infiltrated water is at its cleanest, almost all river contaminants are removed. Wells are installed in this zone to pump the water to be used for drinking. The purity of this water and its suitability for drinking is outstanding, even in examples where there is an event that introduces a shock load of contaminants in to the river .Due to the geologic media's ability to remove the contaminants and travel time of water abstracted for natural filtration, the impact of such an event is minimal and requires. In Malaysia, the National Hydraulic Research Institute of Malaysia (NAHRIM) was evaluated the feasibility of developing the RBF system along the major rivers to mitigate water shortage problem optional water supply for domestic and industrial consumption. One of the main tasks in the RBF development project is the feasibility of site to develop the RBF system. The success of RBF depends on favorable hydrogeological conditions near the river. Riverbank filtration relies on the ability of a well to induce recharge from a nearby surfacewater source, with the degree of hydraulic connection between surfacewater and groundwater is often the factor that limits wellfield capacity. However, site specific and requires extensive site investigations and pilot studies to assess its feasibility based on local conditions. In Malaysia guidelines or index system of RBF was not available for evaluate the suitability of RBF and to the transfer of this sustainable and multiple-contaminant removal technology. Though very appropriate for both developed and developing countries, RBF has not been utilized (fully) in developing countries due to lack of knowledge and tools/methods for design of such systems. Factors affecting of performance of RBF System such as site specific, conditions source (river, lake), geology and soils, geohydrology (alluvial, unconfined aquifer, aquifer depth (depth to water table), permeability (hydraulic conductivity), travel distance, well placement and spacing between wells, travel (residence), time well placement and operation (pumping Rate) and permeability (conductivity) is more important factor to develop the index system. Design components for RBF system were including number of wells (production capacity per well), spacing between the wells, distance of the wells from the riverbank, share (%) of river water and native groundwater. Water quality obtained from the RBF system, and post-treatment requirements will consider in the index system of RBF. In this research, the downstream river basin of the Langat River, Linggi River and Muda River was set as the study area and the potential RBF index suitable area for RBF was evaluated. Based on the analysis of natural geography and hydrogeological conditions, an index system was established to evaluate the suitability of RBF along the main of the river in the study area. The aim of this research is to find potential suitable areas for future water development plans and further detailed

investigation of RBF systems. The specific goal of RBF is mainly to provide sufficient water resources with good water quality. A multi-criteria RBF evaluation index system was established based on water quantity, water quality, the development and utilization conditions of groundwater resources and the interaction intensity between surface water and groundwater. The index weight and detailed score criteria were all based on specialist marking methods, which is more effective for a specific site but may not be applicable to other areas.

2 STUDY SITE

RBF in West Malaysia aims to achieve the strategic goals of improving drinking water production including an improved availability and increase sustainability of drinking water supply at high quality, at low cost and with a limited generation of waste or contaminant product. This RBF pilot study site offers important advantages as the researchers can evaluate the RBF index of RBF technique for River in Malaysia. For this downstream river basin of the Langat River, Linggi River and Muda River, and was calculated the feasibility RBF index based on the parameters affecting yield in RBF systems. Three study sites for this study were chosen based on the following criteria:

2.1 Bank Infiltration systems

2.1.1 Langat River

The Langat riverbank corridor area at the Jenderam Hilir located approximately some 5 km south of Putrajaya was chosen and has been identified as potential site to do the feasibility study of using RBF system. It extends between latitude 20 53' 28.56" N - 20 53' 39.75" N and longitude 1010 42' 03.78" E – 1010 44' 14.58" E, covering an area of 10 km² radius (Figure 1.1). The aquifer system in the study area consists of alluvial deposits of sand, silt and gravel which form shallow aquifer. The unsaturated zone sits on the aquifer consists of clay. The thickness of this clay layer is about 1-3 m. The aquifer is mostly of fine to coarse grained sand with mixture of gravel. The thickness of aquifer layer ranging from 5-20 m. It can be locally heterogeneous due to the presence of beds of fine to coarse-grained sand. Based on drilling information, gravelly sand or sandy gravel aquifer layer are overlain by layer of clay, while some areas are overlain by a layer of low permeability fine sand or silt which make the aquifer unconfined and semiconfined depending on locations. Bedrock in the study area is located at depths of 20 m. Due to the presence of sandy gravel which has high porosity and transmissivity, and connection to the river, during high river flow, water from the river recharged the aquifer. Meanwhile during the river low flow, groundwater is discharged into the river. Rechargeable between river and groundwater through riverbed layer can be slow or fast depending to the thickness of hyporheic zone layer.

The average mean monthly flow at the study area is about 32.15m³/s. The 80% dependable flow for Langat River is 19 m³/s. The average low flow is 4.76m³/s for year 1978 – 2013. The riverbed permeability is 0.4m /day. The result of the river

survey has shown the cross sections single thread channel with bank full width ranging from 22.75 m to 50.20 m that representing medium size river. Two new test wells i.e. TW1 and TW2 with outer diameter of 250 mm, at a depth of 14.50 and 15.42 m respectively were constructed to determine the hydraulic parameters of the wells and aquifer. The pumping test at TW2 was started at a rate of 12.90 m³/hour with static water level of 3.24 m below top of casing (TOC). The flow rate was constant throughout the test at 64.50 m³/hour. The total drawdown at the end of the one day pumping test was 3.16 m. Recovery test in TW2 after step-drawdown test shows that the water level recovered back to 3.53 m after 7 hour pump stop with efficiency of 90.82%. The static water level was 3.50 m and the final water level was 7.03 m below ground level. The T and K values for TW2 were 42.16 m²/hour and 2.72 m/hour. According to this aquifer test, the range of travel time from river water to recharge the aquifer is averaged at 3-4 days. The travel time was estimated by dividing K in m/day with the aquifer thickness. The test well TW2 has successfully shown that well-constructed along the river banks is capable of yielding significant volume of water with very much better quality. Recovery in TW2 shows that the water recovered from final water level (7.03 m bgl) to 4.2 m bgl within 1 hour (80%), than slowly recovered after that. For TW1, a step-drawdown test (three steps), a 72 hours constant discharge test. The pumping test at TW1 was started at a rate of 6.53 m³/hour with static water level of 6.47 m below top of casing (TOC). The flow rate was constant throughout the test at 10.10 m³/hour. The final water level was 7.44 m. The total drawdown at the end of the one day pumping test was 0.97 m. The recovery test in TW1 after step-drawdown test shows that the water level recovered back to 6.66 m after 1 hour pump stop with efficiency of 80.41%. The constant discharge test was carried out with a pumping rate of 10.10 m³/hr. The static water level was 6.09 m and the final water level was 7.45 m below ground level. The T and K values for TW1 were 9.24 m²/hr and 0.63 m/hour. The drawdown of TW1 was very low, this shows that the well is capable of producing much higher amount of water but due to limitation of effective aquifer layer and pressure from thickness clay layer on top of aquifer, can cause slow recharge from the river. According to this aquifer test, the range of travel time from river water to recharge the aquifer is averaged at 1-4 days. The travel time was estimated by dividing K in m/day with the aquifer thickness.

2.1.2 Linggi River

The location of the RBF study is located at Water Treatment Plant Linggi River, Negeri Sembilan. It is located at the riverbanks of the Linggi River. Linggi River Water Treatment Plant supplies 60% and 100% of the water requirements for Seremban and Port Dickson, respectively (Figure 1). A total of 35 exploration wells of 50mm diameter were constructed as a preliminary assessment on the potential of the ground water in the study area. Based on information obtained from exploration wells, the next six test wells have been constructed in which five wells constructed in layers of alluvium and one well in hard rock. Test wells in the alluvium has a depth ranging from 6m to 13m with a diameter of 254mm and

the test well in hard rock with a depth of 36m with a diameter of 152mm. The discharge from test well in hard rock was estimated less than 5m³/hour during the drilling. However the test wells in the alluvial water discharges ranging from 28m³/hr to 65m³/hour. Generally, the drilling of the 100mm diameter monitoring borehole encounters yellowish brown soft clay with some decaying wood fragments from the surface to a depth of 2.0m, followed by dark grey medium grained sand up to a depth of 3.50m. This is followed by dark grey fine grained sand with slightly clay up to 5.50m, followed by dark grey medium to coarse sand and some dark grey silty to fine grained sand up to 7.20m, followed by schist bedrock. A total of 35 units of 50mm diameter shallow alluvium piezometers and two (2) units of 150mm diameter shallow alluvium test wells were constructed along Linggi River riverbank. The Step Drawdown Test that was carried out on the exploration tubewell SL-TW 1 consists of 5 discharging steps, each step for duration of 90 minutes. The 4 discharging steps that were carried out were 5.5m³/hour, 6.5m³/hour, 7.5m³/hour and 8.5m³/hour. The initial static water level was at 1.30m meter below ground level, and the final water level was at 2.83m below ground level, with a total drawdown of 0.52m. The Constant Discharge test was carried out at a rate of 7.96m³/hour for the exploration tubewell SL-TW1. This is the maximum capacity of the submersible pump that was used for the pumping test. The initial static water during the Constant Discharge Test was 1.1m below ground level and the final water level after pumping non-stop for 72 hours was 2.37m, with a total drawdown of 1.27m. The transmissivity value is 1.37m²/hour. The procedure for Recovery Test is for the water level in the pumped tubewell to achieve 80% recovery from its initial static water level. The Recovery Test was carried out for a total of 4.5 hours. The initial residual drawdown was 2.37m, and the residual drawdown after 4.5 hours was 0.3m, with a recovery of 76.38%. The transmissivity value for the Recovery Test is 18.21m²/hour. Constant Discharge Test was performed on the test wells SL-TW2 to determine aquifer characteristics. During the pumping tests, drawdowns of the groundwater level were also measured at the nearby monitoring wells (SL-MW15, SL-MW14, SL-MW13, SL-MW12, SL-MW11 and JL13). After the constant pumping at a rate of 17.1 m³/hour for 72 hours, the final drawdown is 7.24m with a total drawdown of 5.65m. The total drawdown at the monitoring wells ranged from 0.69m to 1.05m. Based on this analysis, the hydraulic conductivity of the test wells SL-TW2 is 17.1m/day, while for monitoring wells ranged between 54.1 to 63.1m/day. The average of the hydraulic conductivity is 53.1m/day. Transmissivity for the test well is 102m²/day, while for monitoring wells ranged between 325 to 379 m²/day. The average value of the transmissivity is 319m²/day. The Recovery Test resumed immediately after the Constant Discharge Test was completed. The Recovery Test was carried out for a total of 120 minutes after the pump was shut down. For a period of 120 minutes after the pump is shut down, the remaining drawdown of the test well SL-TW2 is 0.2m, while for monitoring wells ranged from 0.15 to 0.64m. Based on this analysis, it was found that the hydraulic conductivity of the test well SL-TW2 is 16.7m/day, while for monitoring wells ranged between 38.8 to 51.1m/day. The average hydraulic conductivity value is 43.0m/day. While the transmissivity value for the test well is 100m²/day, and for monitoring wells ranged between 233 to

306m²/day. The average transmissivity value is 258m²/day. Based on Kozeny-Carman equation, the hydraulic conductivity is between 3.6 to 174.6m/day with an average of 48.5m/day. Hydraulic conductivity values based on the Hazen are between 6.79 to 202.7m/day with an average of 61.4m/day. The values obtained are compatible with the results of grain size analysis carried out which showed that medium to coarse grained sand is the dominant size in the study area. The transmissivity, T values obtained from the Constant Test and Recovery Test ranged between 100 to 379m²/day, with an average of 257m²/day. Storage coefficient, S is obtained from discharge test equipment for pumping the test wells. The results of the analysis conducted found that storage coefficients ranged between 4.17×10^{-4} and 2.6×10^{-1} with an average of 3.93×10^{-2} .

2.1.3 Muda River

The study area is located in the northwest state of Kedah and Pulau Pinang within the Muda River Basin, Peninsula Malaysia, and extends between RSO longitudes 100029'0" to 100033'30" Easting and latitudes 5032'30" to 5035'30" Northing and covers an area of 35 km² (Figure 1). The nearest town to the study area is Sungai Petani (Kedah) and Kepala Batas (Pulau Pinang). The Muda River has been developed as one of the most important water resources for agriculture and water supply for Kedah and Pulau Pinang. The three major tributaries of Muda River system are Ketil River, Sedim River and Chepir River. This location near to the residential and crops area. The Muda River is the longest river in the state of Kedah with its upstream flow coming from the northern mountainous area of the state. The river which has length of 180 km, flows towards the southern area of the state and has a catchment area 4210 km². Downstream, the river charges its course towards the west coast after passing the confluence of the mainstream and its largest tributary which is the Ketil River. The annual rainfall in the study area is about 2000-3000 mm. Both Kedah and Pulau Pinang have the rights to use the water from the Muda River. The Quaternary stratigraphy of the study area is divided into Beruas Formation and Gula Formation. The uppermost layer is the Beruas Formation consisting of Holocene terrestrial sediments of brownish color. Underlying the Beruas Formation is the Gula Formation. It is comprised of clay, silt, and sand with shells. The presence of the Gula Formation acts as an impermeable layer, thus confining other formations beneath it. This site was chosen for RBF study due to the high water demand in the area and groundwater is seen as one of the source with very high potential to be developed as supplementary source to meet the high public water supply demand. Fifteen monitoring wells and two test wells were constructed at the study site and pumping tests have been carried on these two test wells. The pumping tests indicated that TW1 were able to produce 43.92m³/h and TW2 is about 51.609m³/h during the duration of 72 hours pumping tests with drawdown 3.22 m and 1.88 m respectively. Water quality analyses were carried out from the Muda River and groundwater. From the study site showed decreased in turbidity, nitrate, aluminium and sulphate in groundwater. The study on the effectiveness of BI method in Muda River Basin, Pulau Pinang and Kedah shown that the water quality from BI is improved rather than river water quality

and the quantity of water abstraction is high because depth of aquifer is about 30 m from ground level. The study was carried out to evaluate the hydraulic properties of the aquifer and riverbed and to determine the effectiveness of the BI at Muda River. Grain size analysis was conducted to identify the proportion of clay, silt, sand and gravel for both test well (TW). Based on the results of grain size analysis, the aquifer composed of fine to coarse grained sand with a mixture of gravel and higher moisture content down to the base of the aquifer system. The hydraulic conductivity of riverbed sediment based on grain size analysis is 80 m/day and the width of Muda River is 90 m to 130 m.

3 METHODOLOGY

3.1. Distance of RBF

The distance of RBF from river at study area was firstly considering following two criteria:

- i. The length of riverbank affected by a RBF system increases with the number of the applied wells and is expected to significantly influence yield. The more wells are situated along the riverbank the longer the reach of the river the aquifer gains recharge from runoff flow
- ii. Hydraulic connection extent between river water and groundwater. The feasibility of RBF is getting worse and may even be impossible with increasing distance from river. In order to get more complete background information on potential RBF sites, a large study area was recommended. According to the runoff quantity, the study range of RBF was divided along the river (Table 1). Where there is a surface water divide in the study range, this natural feature is used as the boundary of the study area.

Table 1.Type of surface runoff and distance of RBF to river

Type	1	2	3	4	5
Surface runoff (m3/day)	>600	600-250	250-100	100-40	<40
Distance from river (m)	200	180	150	130	100

3.2 Evaluation Index System

The aim of this research is to find potential suitable areas for future water development plans and further detailed investigation of RBF systems. The specific goal of RBF is mainly to provide sufficient water resources with good water quality. A multi-criteria RBF evaluation index system was established based on water quantity, water quality, the development and utilization conditions of groundwater resources and the interaction intensity between surface water and

groundwater. The index weight and detailed scoring criterion were all based on specialist marking methods, which is more effective for a specific site but may not be applicable to other areas (Table 2).



Figure 1. The location of the study area (A) Linggi River, (B) Langat River and (C) Muda River and the test wells and monitoring wells at the study area

Table 2. Evaluation index system and index weight

Category of evaluation Index		Evaluation Index (X)	Index Weight (W)	
Water quantity	groundwater	Hydraulic conductivity (K)	0.10	0.30
		Aquifer thickness (M)	0.10	
	surfacewater	Runoff in cross section (Q)	0.10	
Water quality	groundwater	Status of groundwater (G)	0.15	0.30
	surfacewater	Status of surface water quality (S)	0.15	

Interaction intensity between surfacewater and groundwater	Groundwater hydraulic gradient (I)	0.10	
	Possible influence zone width of surfacewater under the condition of groundwater exploitation (L)	0.10	0.30
	Permeability of riverbed layer (R)	0.10	
The exploitation condition of groundwater resource	Groundwater Level (D)	0.10	0.10

The main reasons for index weight are explained as follows. (1) The sum of evaluation index weight is equal to 1 by value assignment. The index weight of water quantity, water quality and interaction intensity between surface water and groundwater each accounts for 0.3 of total weight individually, and this allotment reflected the aim and decisive factors of RBF suitability evaluation of the study site. The index weight of development and utilization conditions of groundwater resources only account for 0.1 of total weight, because the factor will influence groundwater development cost and help to confirm the priority, but it is not a decisive factor for RBF suitability evaluation. (2) For the index of water quantity, the groundwater index accounts for 0.20 and surface water index accounts for 0.1, because the water source of RBF waterworks comes from riverside groundwater and surface water via bank filtration. Hydraulic conductivity and aquifer thickness influence the infiltration rate and capacity of surface water to aquifer. (3) For water quality, the index weight of groundwater and surface water are equal to 0.15. The groundwater quality only indicates the current status and treatment effect of RBF. In order to avoid any possible groundwater contamination from surface water, the treatment effect of RBF should not be addressed too much and the surface water quality should not be neglected. (4) The interaction intensity between surface water and groundwater controls the water exchange efficiency. Only those places within the influence zone of surface water, the aquifer prone to receiving sufficient water quantity through RBF, the permeability of riverbed layer controls the real water exchange rate and the groundwater hydraulic gradient partly reflects the real condition of aquifer property. (5) The index weight of groundwater depth accounts for 0.1, because it will only influence the priority order of potential RBF areas.

3.2.1. Water Quantity

3.2.1.1 Hydraulic Conductivity (K)

Aquifer hydraulic conductivity (K) reflects lithology and permeability of an aquifer. The index value of hydraulic conductivity is shown in Table 3.

Table 3. Index value of Hydraulic conductivity (K)

K (m/d)	>100	100-50	50-20	20-5	5-1	1-0.1	<0.1
Index value	100	90	80	70	60	30	0

3.2.1.2 Aquifer Thickness (M)

A suitable RBF site requires a certain scale of aquifer for adequate water production. The aquifer thickness index value is shown in Table 4.

Table 4. Aquifer thickness (M) index value

M (m)	>50	30-50	10-30	5-10	3-5	1-3	<1
Index Value	100	90	80	70	60	30	0

3.2.2 Surface Runoff (Q)

The larger the annual average surface runoff, the greater the recharge potential of surface water to groundwater, and the greater the potential suitability of the area for RBF. The surface runoff index values shown in Table 5.

Table 5. Surface Runoff index value

Q (m ³ /day)	250-100	100-40	40-10	10-5	5-1	1-0.1	<0.1
Index value	100	90	80	70	60	30	0

3.2.3. Water Quality for Groundwater and surfacewater

Water quality status of surface water or groundwater directly reflects whether is suitable for drinking, and indicates the cost of water treatment that will produce drinking water. In order to guarantee the water quality of potential waterworks, the water quality improvement effect of RBF was thought of as an additional assurance. Because of limit element of parameter in water quality index for surfacewater and no water quality index for groundwater was developed, Environmental quality standards for surface water (GB3838-2002) (General Bureau of China National Environmental Protection, 2002) and Quality standard for groundwater (GB/T 14848-93) (Quality Standard for groundwater, 1993), water of quality grades I, II and III can be used as drinking water directly.

Water of quality grade IV must be treated before it is supplied for drinking. Because higher index values of the quality grade indicate lower quality water, the assigned index values decrease as water quality grades change from I to IV. Water of quality grade V is a negative factor for the suitability of RBF, and its index value is assigned a negative number. This can make the negative water quality factor play a decisive role in the process of suitability evaluation of RBF. The groundwater quality (G) and surface water quality (S) suitability scoring criterion is shown in Table 6.

Table 6. Groundwater quality (G) and surface water quality (S) index value.

G/S	I	II	III	IV	V and Worse than V
Index Value	100	95	90	60	-275

3.2.4. Interaction Intensity between Surface Water and Groundwater

A large proportion of RBF waterworks primarily capture surface water; the interaction intensity between surface water and groundwater plays an essential role. Hydraulic Gradient (I) under the Current Condition The hydraulic gradient directly reflects recharge-discharge relationship between surface water and groundwater, and it also indicates recharge-discharge conditions of the interaction zone. A positive hydraulic gradient was defined to represent scenarios in which river water recharges groundwater; similarly, a negative hydraulic gradient indicates that river water is recharged by groundwater. A positive value of hydraulic gradient is beneficial to RBF in most situations; however, a very large value for the hydraulic gradient might mean that riverside groundwater is being exploited intensively or that the interaction between surface water and groundwater is poor, because the very low permeability of aquifer or riverbed will obviously enlarge hydraulic gradient and decrease water exchange intensity. A negative hydraulic gradient indicates that the surface water cannot be induced into the ambient aquifer at present. Nevertheless, a negative gradient might be reversed and to be beneficial for RBF under groundwater exploitation condition. If the present negative hydraulic gradient is excessively small, it may indicate that the permeability of aquifer or riverbed is very poor, and the large groundwater recharge potential from river water should not be expected under any exploitation conditions. The hydraulic gradient suitability scoring criterion is shown in Table 7.

Table 7. Hydraulic gradient (I) suitability evaluation criteria

I	>10	10-5	0-5	0 to -5	-10 to-5	<-10
Index value	40	80	100	90	80	60

3.2.5 Influence Zone Width of Surface Water under the Condition of Groundwater Exploitation (L)

The influence zone by surface water indicates the range of the hydraulic connection between surface water and groundwater. The greater the hydraulic connection range between surface water and groundwater, the superior the RBF site. The possible influence zone (L) was assumed to be equal to the ratio of the aquifer thickness (M) to the hydraulic gradient (I). The suitability index value for the possible influence zone by surface water is shown in Table 8.

Table 8. Index value for the possible influence zone width of surface water under the condition of groundwater exploitation (L).

L	$< \frac{M}{ I _{max}}$	$\frac{M}{ I _{max}}$ to $\frac{M}{ I _{average}}$	$\frac{M}{ I _{average}}$ to $\frac{M}{ I _{min}}$	$> \frac{M}{ I _{min}}$
Influence intensity index value	Strong 100	Medium 80	Weak 60	None 30

3.2.6 Permeability of Riverbed Layer (P)

Riverbed permeability indicates the exchange capacity between surface water and groundwater. Riverbed permeability is calculated using the hydrogeological cross section of the riverbed obtained from a field investigation as standard. Then, the lithology under the riverbed is analyzed to obtain its permeability, either by measurement or practical experience. The riverbed permeability suitability index value is shown in Table 9.

Table 9. Permeability of riverbed layer (R) suitability scoring criterion.

P (m/d)	>5	1-5	0.5-1	0.1-0.5	0.05-0.1	0.01-0.05	<0.01
Index value	100	90	80	70	60	30	0

3.2.7 Groundwater Level

When considering the development and utilization of groundwater resources, groundwater level is a primary concern because it indicates the cost of building RBF wells. If the groundwater level is too far below the ground surface, it may be in a highlands area or in a cone of a groundwater depression. In those places, the extraction of groundwater requires more energy. The groundwater level index value is shown in Table 10. Because groundwater level is strongly influenced by regional characteristics, this standard should be adapted to different hydrogeology conditions.

Table 10. Groundwater depth (D) index value.

D (m)	<5	5-10	10-15	15-20	20-25	25-30	>30
Index value	100	90	80	70	60	30	15

3.2.7. Suitability Index

According to the evaluation index system created above, the complex suitability index for a potential RBF site can be calculated by weighted summation using Equation (1).

$$A = XK \times WK + XM \times WM + XQ \times WQ + XG \times WG + XS \times WS + XI \times WI + XL \times WL + XR \times WR + XD \times WD \quad [1]$$

In Equation (1), A is the suitability index of a potential RBF sites, X is the index value of individual indices as defined in Tables 3–10 and W is the weight of each corresponding index as defined in Table 2. The RBF suitability was then classified into five grades according to the suitability index value (Table 11).

Table 11. RBF suitability class

Suitability Index Value	Class	Suitability Evaluation
90-100	I	Excellent suitable areas
80-89	II	Good suitable areas
70-79	III	Moderate suitable areas
60-69	IV	Poor suitable areas
<60	V	Unsuitable areas

In areas for which RBF suitability grades are between I and III, water quality satisfies human drinking water requirements and does not need treatment, and water quantity is adequate. Furthermore, the exchange between river water and groundwater is intensive, the water quantity and water quality of RBF is guaranteed, and there are good development and utilization conditions of groundwater resources. In short, areas in grades I–III are suitable for RBF. In areas for which the RBF suitability grade is IV, either water quality does not satisfy drinking water requirements directly or the water recharge is insufficient. After water treatment and limited exploration, such areas may be suitable for limited-scale RBF waterworks. Areas in grade V are unsuitable for RBF works because of unacceptable water quality, high water treatment costs or insufficient water recharge to sustain extraction at the required rate and volume.

4 RESULTS AND DISCUSSION

The various index values were integrated using Equation (1) to determine the overall suitability of locations at the study area for RBF works. Nine typical points were selected to show the characteristics of each class of suitability (Table 12). Based on the multi-criteria analysis, the suitability of many locations in the study area was classified as grade I and II and should be suitable for locating RBF works.

Table 12. All single index values and scores for three sites

Index	Muda River		Linggi River		Langat River	
	Value	Index Value	Value	Index Value	Value	Index Value
K, hydraulic conductivity	80 m/d	90	53.1	80	63.64 m/d	90
D, Aquifer thickness	17.9 m	80	7.5	70	17.1	80
R, surface runoff	432x104m3/day	100	518 x 104 m3/day	100	164 x 104m3/day	100
G, Groundwater quality	III	90	II	95	II	95
S, Surfacewater quality	II	95	III	90	III	90
I, Hydraulic gradient	-0.2	90	-0.1	90	- 0.035	90
W, Influence zone	strong	100	strong	100	strong	100
P, Permeability of riverbed layer	1.9 m/d	90	0.3 m/d	70	1.2 m/d	90
D, groundwater level	2.2	100	2	100	1.6	100
Total Index value		92.80		88.33		92.80
Class		I		II		I

5 CONCLUSIONS

Many important factors affect the site selection for RBF works such as groundwater and surface water quantity, current water quality situation, hydraulic interaction degree and exchange relationship between groundwater and surface water. In this research, a multi-criteria index system was developed

for assessment of RBF site suitability. The RBF index system was based on a detailed analysis of physical geography and geological and hydrogeological conditions (which were considered to be the main influential factors of RBF), as well as on the development and utilization of water resources and water demand. Value index criteria based on specialist marking methods were used to determine weighting coefficients and weighted scores. The evaluation method was integrated will put into the spatial analysis features of GIS to determine the distribution of suitable areas for RBF works. However, this research still has some limitations that must be mentioned here and will be studied further.

ACKNOWLEDGMENTS

The research reported herein is funded by National Hydraulic Research institute of Malaysia.

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