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River Flood Modelling Using SOBEK: A Case Study from Ciliwung Catchment, Indonesia

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Abstract: ***This case study presents flood management by diverting excess flood water to a neighbouring catchment which has a safe capacity to carry excess flood water. Floods in a river may be happen due to surface and sub-surface runoff during periods of extreme rainfall or cyclone. In this study it was shown that during extreme rainfall event Jakarta city which is located in Ciliwung catchment is vulnerable to flood. The Ciliwung River has been changed in the last 100 years due to unplanned city development and narrowing the flood plain of the river. Flood can be managed by diverting excess flood water to other catchment where it is feasible. In this case study, possible options to divert excess flood at Ciliwung River to a neighbouring catchment were assessed. SOBEK 1D Rural model is used for simulation river flood. From this simulation, different reliable outputs are obtained by analysing the sensitivity of the numerical and physical parameters. Every step of analysis some recommendation and opinions are made for further improvement of this project which included relocation of tunnel, using a weir, construction an open channel, build a seasonal dam etc. Further detail study is required to make a final decision, by assessing river morphology and considering socio-economic and ecological aspect of the catchment. Similar approach can be applied in other catchment to reduce flood vulnerability of that particular area.***

Keywords- **Ciliwung River, SOBEK, Flood Modelling, Jakarta**

I. Introduction

Ciliwung River (Figure 1) is one of the major river in Java Island flowing through Bogor, Depok and Jakarta (Doan et al., 2012). The total length of the river is about 136 km with catchment area of about 352 km^2 (Sun et al., 2014) which is divided into 9 sub-basins for rainfall-runoff modelling. Total population along the Ciliwung river basin is about 4.1 million (Doan et al., 2012). Catchment average annual rainfall is about 1700 mm. January is the wet season peak rainfall month with monthly average rainfall is about 400 mm and September is the least rainfall month with monthly average rainfall is about 30 mm (WMO, 2014). Ciliwung River causes lot of flooding problem in Jakarta city. This project describes a small model for the simulation of flood diversion from the Ciliwung catchment to a neighbouring catchment. The diversion route will be created via a tunnel. Diversion of flow is initiated as soon as the crest of the fixed entrance weir is overtopped. Figure 2 shows plan view of the study area.

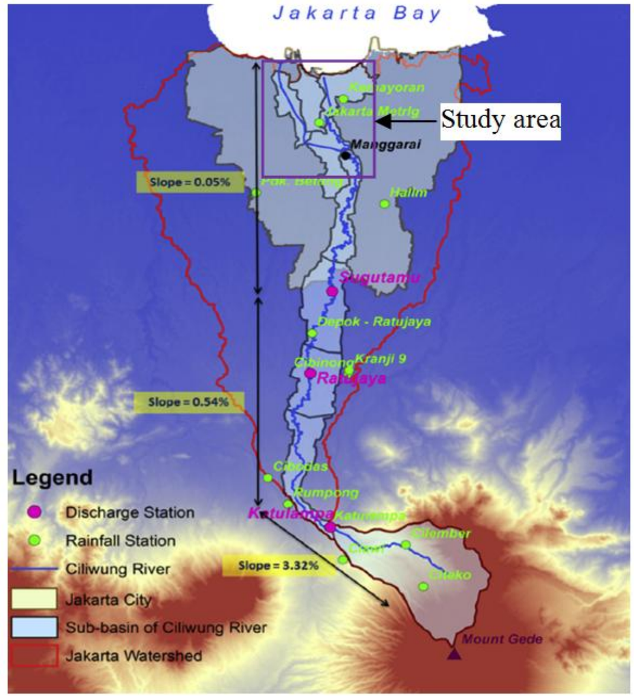

Figure 1: Ciliwung River and its catchment (after Sun *et al.*, 2014)

Model schematisation:

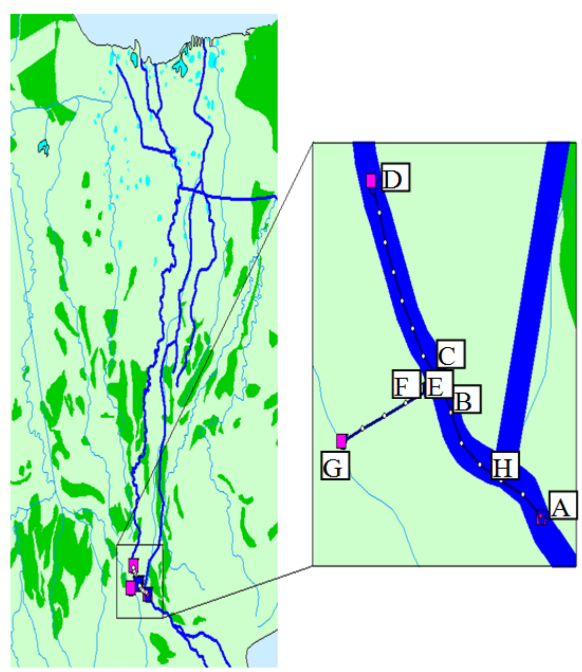

Figure 2: Plan view of the study area

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Table 1: Schematisation of the Initial data

Reach	Length (m)	Elev. $_{initial}$ (m)	Elev. $_{final}$ (m)	s (m/m)	Section	n ($s/m^{1/3}$)
A-B	1916.94	280.82	254.07	0.014	x-y	0.06
C-D	2043.36	254.07	225.5	0.014	x-y	0.06
C-E	149.98	254.07	254	0.0005	Trapezium	0.025
E-F	45.92	254	254	0	Trapezium to Square	0.015
F-G	1243.21	254	242	0.0097	Tunnel	0.012

Table 1 shows model schematisation data. The length of the weir at point E is 30 m and the elevation of its crest is 255.5 amsl.

II. Material and Methodology

The Tunnel was connected with the river using a horizontal transition from a trapezoidal to a rectangular intake structure and its parameters are presented in Table 2. The tunnel has a circular shape with 6.00 m diameter.

Table 2: River & Tunnel Connection Parameters

Parameter	Trapezium		Rectangle
Bottom Elevation (amsl)	254		254
Surface Elevation (amsl)	258		264
Manning n	0.025		0.015
Width (m)	100	120	20
Height (m)	10		9

Necessary cross section was defined in front of the intake structure to provide values of wetted perimeter (Stoesser et al., 2010), surface roughness (De Marchis and Napoli, 2012) and the river slope (Phillips and Slattery, 2007) into the model. Then the tunnel and the weir were added, the connection between the river and the tunnel is done by a channel with an initial trapezoidal section and a final rectangular section. These data is shown in table 1 and 2 above.

After running the simulation, the results are shown in following figure in below.

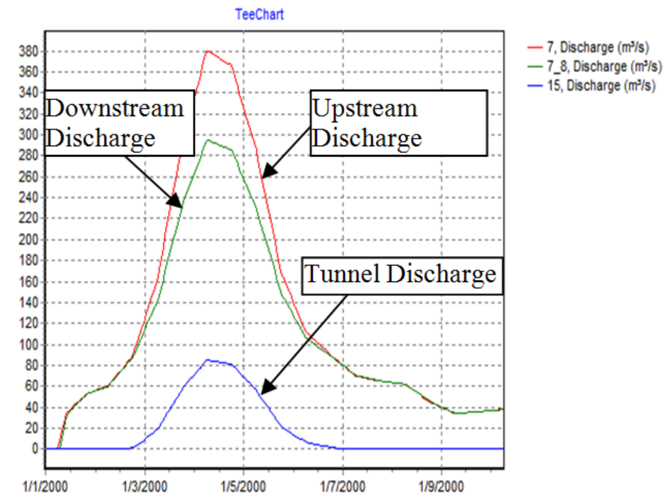

Figure 3: Simulation Results of the discharge in (m^3/s)

Figure 3 shows that the maximum discharge in the tunnel is about 80 m^3/s, (21%) of the peak discharge of the upstream hydrograph which is (380m^3/s). Figure 4 shows, the velocities in the river upstream is 4 m/s and downstream is about 3.25 m/s , while the velocity in the tunnel is higher than 9 m/s during the peak discharge, due to continuity (Abedijaberi and Khomami, 2011) which is expected higher velocity at narrower section.

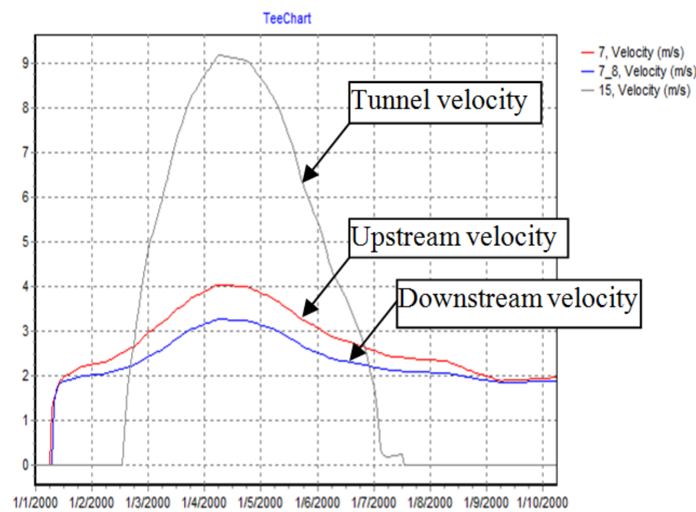

Figure 4: Simulation Results of the velocity in (m/s)

At point H in Figure 2, there is a tributary of the river that is not considered in the scheme. Since there is no information about it, it is not possible to judge if its contribution.

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Figure 5 shows water depth is higher at downstream in comparison to water depth at up-stream. As water levels are varying highly so it is clear the river is very steep slope.

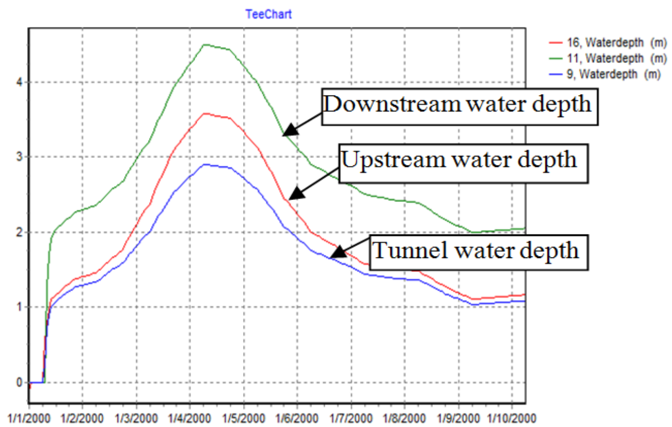

Figure 5: Water depth at DS, US and Tunnel

Boundary conditions:

The upstream boundary conditions(Prange and Gerdes, 2006; Prario et al., 2011) of the river is a hydrograph (Figure 6) at point A, downstream boundary condition is a rating curve (Figure 7) at point D and a fixed water level of 243 m at point G at the end of the tunnel.

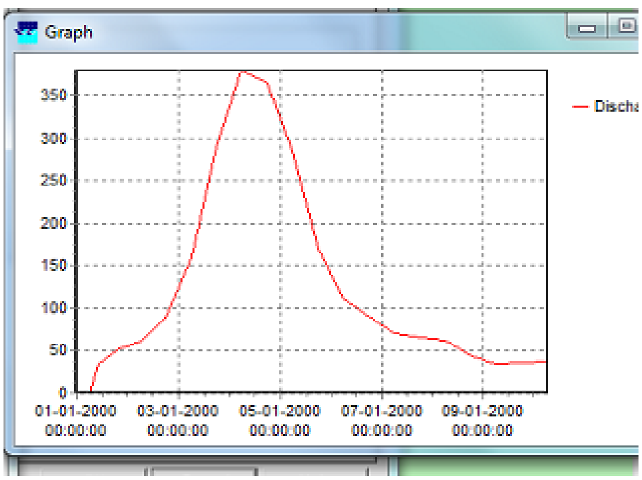

Figure 6: Upstream Boundary Condition

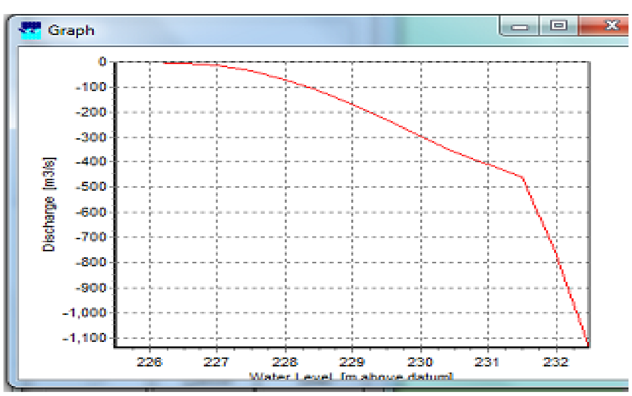

Figure 7: Downstream Boundary Condition

For solving the water flow equations (Darbani et al., 2011) (continuity equation and momentum equation), information about the water flow must be supplied at the model boundaries.

At each boundary node, one condition for the water flow must be specified. The following options are available:

1. Discharge (constant, tabulated function of time, tabulated function of the water level).
2. Water level (constant, tabulated function of time).

In the staggered grid used by SOBEK-Flow-module, the discharges are defined for the reach segments; a discharge boundary is imposed on the first reach segment next to the boundary. Therefore, the calculation point just before this first reach segment (first one in a reach) is undefined and will not be taken into a count during the calculation.

The water level boundary is defined in the first calculation point next to the boundary. This calculation point actually has the same coordinates as the boundary node.

The peak discharge is 380m³/s and the rating curve has a maximum capacity of 1100 m³/s. The fix level at the end of the tunnel doesn't seem to be a correct approach, it would only be so if the tunnel finished in the river and this is not the case.

III. Results and Tables

Sensitivity of the Numerical Parameters:

The adjusted of the numerical parameters is a very important stage during the modelling process. The selection of the adequate time step Δt and the space step Δx (Samadi et al., 2011) contribute to avoid the numerical instability and convergence problems.

The time step used in the model is 5 minutes; the hydrograph duration is about 10 days and the discharge data is given every 4 to 12 hours. The output time step is 1 hour.

In order to analyse the model sensitivity(Chávarri et al., 2013; Lespinas et al., 2014), the model was run for different time steps. Figure 8 shows the resulting peak discharge downstream the river and the tunnel, when the time step was increased.

SOBEK program would not allow the user to have a higher computational time step than the output time step. Therefore in order to analyse the sensitivity both time and space parameters should be change.

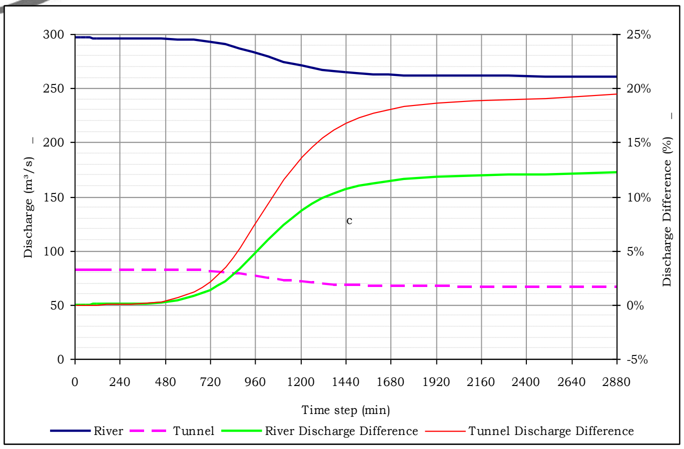

Figure 8: Sensitivity of the model with different time step

Figure 8 shows that for the time step lower than 480 minutes (8 hours), the discharge in the river is approximately the same, but as the time step is higher, the peak is lowered. This is explained because with the bigger time steps, the inflow hydrograph is simplified and the discharge peak is not considered, as shown in Figure 9.

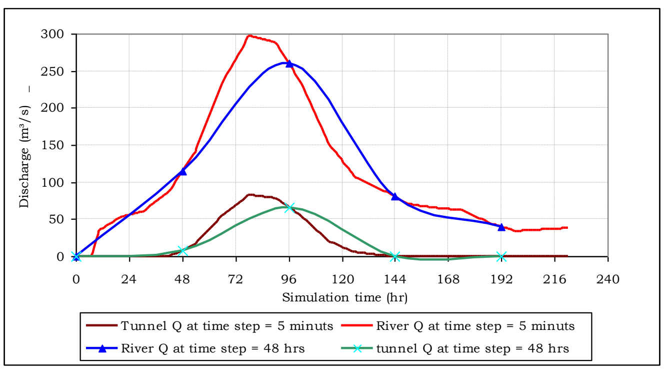

Figure 9: Resulting Hydrographs for different time steps

The inflow hydrograph time steps vary between 4 and 12 hours, therefore this parameter could be considered as the time step upper limit. This is not a rule, since it depends on the shape of the hydrograph because if the discharge rises rapidly (high peak), the time steps must be small in order to reach the peak, but if the hydrograph is flat, the time steps can be bigger.

In this case, even after increasing the time step from 5 minutes to 2 days, the difference in the peak discharge is less than 25%. This could be explained because the Delft-scheme(Werner et al., 2013) used in the SOBEK-Flow-module(Deltares Systems, 2014) to solve the water flow equations, uses a time step (Δt) estimation to temporarily reduce the simulation time step internally under certain flow conditions (large discharges in combination with little storage). The time step (Δt) that is selected by the user must be reduced in this case to guarantee a solution. The time step estimation uses the following formula to set the internal time step:

$$\Delta t = min\left(\frac{f.V_{1\,up}}{Q_1}, \frac{f.V_{2\,up}}{Q_2}, \dots\dots\dots, \frac{f.V_{n\,up}}{Q_n}\right) \dots\dots (1)$$

Where:

f : Factor, (equal to 0.5 for supercritical flow, equal to 2.0 for subcritical flow)

$V_{i,up}$: Volume of the node or calculation point upstream of reach segment i.

Q_i : Discharge in reach segment i.

Little storage (caused by small distance between calculation points in a reach) next to large discharges results in a small internal time step.

If the solution of the water flow equations with the used time step does not converge fast enough or if negative depths are computed, the time step is reduced by a factor 2 (Deltares Systems, 2014). Thus the results are not so sensitive to the time step.

Investigation of the sensitivity of the distance step

SOBEK-Flow-model consists of a network of reaches connected to each other at connection nodes. In each reach a number of calculation points can be defined.

The accuracy of the solution depends on the grid size; the smaller the grid sizes, the more accurate the solution is. Obviously small grid sizes result in a network with more elements and therefore in a longer simulation time. To avoid long simulation times, calculation points should never be chosen too close together. The smallest distance the Delft-scheme uses is 10 meters.

The distance step for this model is 500 m. Figure 10 shows the discharge in both the tunnel and the downstream section of the river for the changes in the distance step used in the program.

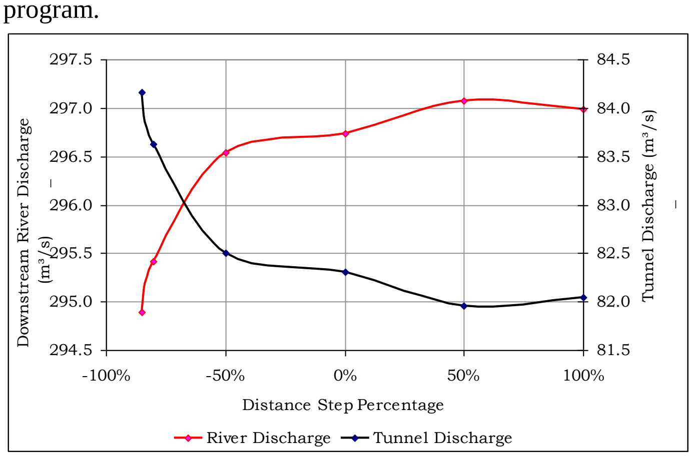

Figure 10: Sensitivity analysis of the distance step

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The difference is explained by the precision of the volume between calculation points. The distance between these calculation points is given by the distance step.

The discharge is sensitive to the variation in the distance step. In this case, since the cross sections are similar and the river is very straight, the difference between the discharge values is not so big, in relation to the total discharge. Still there's a noticeable difference as the distance step increases.

Sensitivity of hydraulic parameters

The sensitivity of the parameters is analysed by keeping all the values except one constant and comparing this value with the percentage of the total discharge or discharge diverted by the system. The downstream river boundary condition is rating curve and up-stream boundary condition is hydrograph. By increasing the value of the rating curve the sensitivity of the system was analysed. Figure 11 shows the dimension of intake structures.

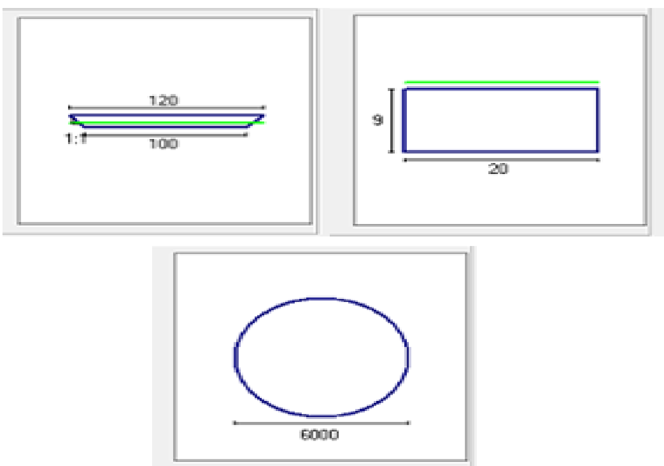

Figure 11: Dimension of intake structure

The existing diameter of the tunnel is 6000 mm or 6.0 m but the first intake structure (Trapezium) dimension (100m X 10m) is very large compare to tunnel diameter which found unwise, engineering point of view during the case study. Table 3 shows discharge variability with tunnel diameter.

Table 3 Discharge varying with tunnel diameter

Pipe diameter (mm)	Qriver (m^3/s)	Qtunnel (m^3/s)
3000	328.3	51
4000	301.3	77.8
5000	295.4	83.7
6000	295.4	83.7
7000	295.4	83.7

The sensitivity analysis of the tunnel then shows that the section diameter is not important when the diameter is reduced more than 20% since the flow upstream is controlled by the tunnel's boundary conditions.

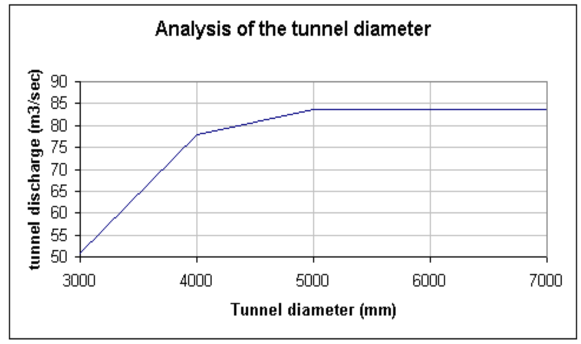

Figure 12: Analysis of the tunnel diameter

Figure 12 shows with existing intake structure the maximum amount of water diverted through the tunnel diameter 5m could be $83m^3/sec$ of flow. Moreover, we find that the diameter of the weir is not sensitive above 5 meter which implies the reduction of design costs too.

The system is also analysed for different water levels downstream the tunnel. The calculated values are shown in Figure 13 below

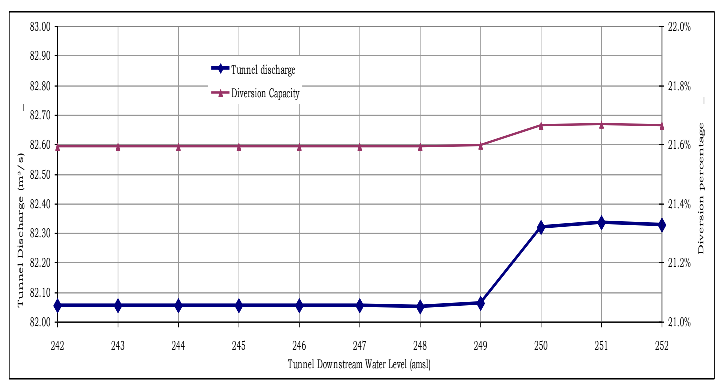

Figure 13: Tunnel Boundary Sensitivity Analysis

Figure 13 shows that as the water level downstream tunnel raises, there is no change in the tunnel capacity until the elevation 249.0 amsl, where the slope begins to increase and then goes back to an almost horizontal slope. Still, the diversion capacity is almost constant, since the variations are not bigger than 0.1%.

In order to let the downstream boundary condition control, the same analysis is performed without the weir (Figure 14), in this case as the water level downstream begins to control, the tunnel capacity is reduced as the water level increases.

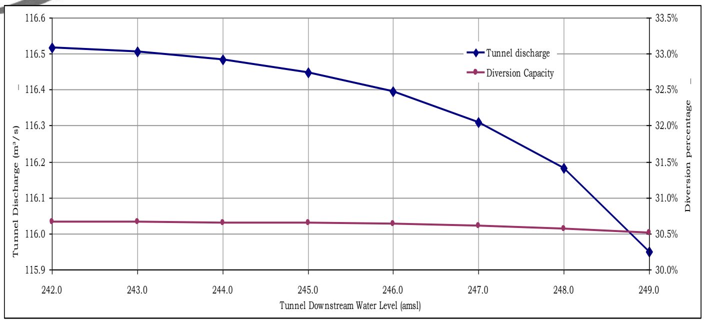

Figure 14: Sensitivity Analysis for the tunnel boundary without weir

Design:

Construction Costs

The impact on the design of the different parameters must be analysed depending on the system conditions. It is important to know where the control section is, in order to know which parameters are more efficient to change and to improve the system behaviour.

The main construction costs are usually in the tunnel length, therefore there could be two options to reduce the costs:

1- To move the tunnel to another location
2- To control the weir

Moving the tunnel to another location

From spatial analysis it is found at the further downstream the Ciliwung River bended towards west therefore the length of tunnel would be smaller at that section. Figure 15 shows the proposed location for the construction of the tunnel as well as diversion of excess flood water from the Ciliwung River to the neighbouring catchment. However, further study is required incorporating morphology of the river at that point and other socio-economic condition of that area.

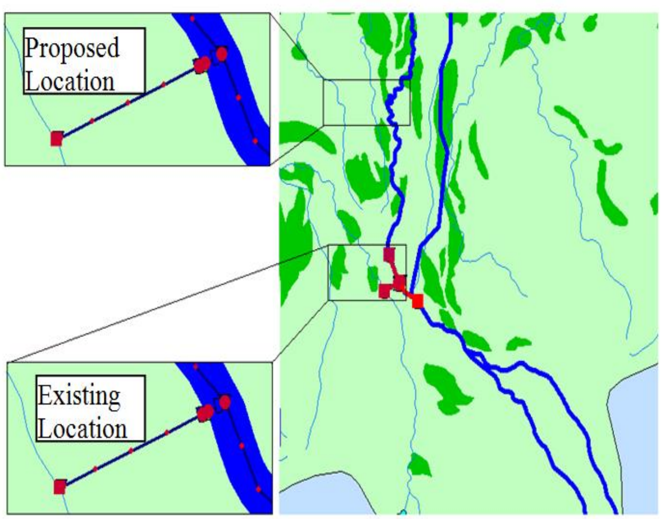

Figure 15: Proposed new location of the tunnel

Controlling the weir:

To divert flood for the discharge greater than 200m³/s in the main river, the water level for this discharge is needed. In order to do that, a rating curve in the intersection between the river and the diversion must be constructed. This is done using the properties of the reach and the node (staggered grid), the rating curve is shown in Figure 16.

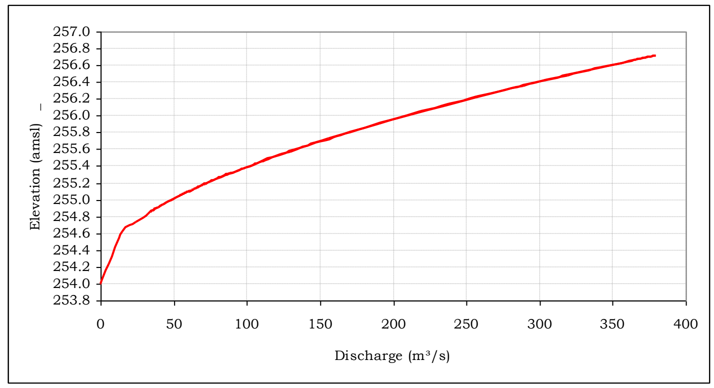

Figure 16: Rating curve at point C

Then, for a 200 m³/s discharge, the water level is 255.9 amsl. This elevation is the weir elevation, since the bottom of the channel is horizontal.

The objective of the weir in the entrance of the tunnel is to divert the water only during the floods, because it is possible to control from which discharge the water will be diverted. The disadvantage of that the diversion capacity will be reduced. But since the main objective of the tunnel is to divert floods and prevent the town from flooding, it is necessary to use a weir so as to maintain the water level in the river at the desired level.

Alternative for the design of the diversion work

1. Construct an open channel instead of the tunnel
2. There is also always the option to change the diameter of the tunnel, but it depends on the amount of water that must be diverted. And this off course an additional cost.
3. Some other alternatives for the design of the diversion works could be infiltration galleries, screened pipe intakes and built seasonal dams.

IV. Conclusion

Floods in a river may be happen due to surface and sub-surface runoff during periods of extreme rainfall or cyclone. In this study it was shown that excess flood comes to the main channel of Ciliwung catchment. The main objective of the present study was to divert the excessive water to the neighbouring catchment through a tunnel by effectively using a weir. For this purpose SOBEK 1D Rural model is

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used for simulation. From this simulation, different reliable outputs are obtained by analysing the sensitivity of the numerical and physical parameters. Every step of analysis some recommendation and opinions are made for further improvement of this project which included relocation of tunnel, using a weir, constructed an open channel, build a seasonal dam etc. Further study is required, by considering morphology of the river, social, economic and ecological aspect of the catchment.

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