Analysis of steady river flow through a sluice gate with a case study of Ciliwung River

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Abstract. During wet seasons, tropical rain may cause high discharge on rivers. To prevent flooding in the watershed area, sluice gates can be installed to control the water surface level. However, operating a sluice gate is not a trivial task; we must be aware of the backwater effect. Backwater is an increase of the upstream water level resulting from an obstruction of the flow, here is caused by the installation of a sluice gate. In addition, characteristic of the upstream and downstream flow which depend on flow types (subcritical or supercritical) must also be examined. In this paper, we analyse the behaviour of steady river flow through a sluice gate. On a river flow, the presence of a sluice gate (with a fixed opening), will change the total energy of the flow. Then, using the specific energy curve we can determine water surface level behind the gate. Further, by implementing the standard step method, a steady water surface on the upstream part of the gate can be calculated. This surface forms a backwater flow that extends far enough behind the gate. Characteristic of this backwater flow depends crucially on the flow discharge and the opening of the sluice gate. For the case study, we took the Ciliwung River with the Manggarai sluice gate, located nearly in the middle of the river. With the aim of preventing flooding in the watershed area, we calculated several scenarios of sluice gate opening under various river discharge. Our results may help sluice gate operator in controlling the Manggarai sluice gate of the Ciliwung River.

1. Introduction
A sluice gate is a man-made hydrology structure which controls the discharge or water level of an open channel flow along a river. Sluice gates control the flow depth if they are placed at the middle of the channel. Meanwhile, they control the discharge if they are placed at the channel’s entrance or a reservoir. In Indonesia, there are many sluice gates installed on many rivers such as Bendung Pasar Baru gate on Cisadane river, Jagir gate on Jagir river, and Demangan gate on Bengawan Solo river, see figure 1.

Indonesia is a tropical country with dry and rainy seasons, which causes the rivers having its discharges fluctuate over the years. In a season of heavy rains, it will make operating sluice gates to be harder while avoiding floods. Thus flooding hazard becomes a threat. Moreover, the biggest problem of rivers in Indonesia is not all of the sluice gates have human/machine
operator. This operator is expected to know how to manage the sluice gates from the discharge information of other sluice gates in the upper stream.

![Sluice Gates](image)

*Figure 1. Several photos of sluice gates in Indonesia: a) Bendung Pasar Baru sluice gate [1], b) Jagir sluice gate [2], c) Demangan sluice gate [3].*

Therefore, the purpose of this paper is to analyse the river flow behaviour and the effects of sluice gates with a model based on Saint-Venant equations and obtain the solution with a numerical approach. In fact, for Saint-Venant equations, there have been many attempts to approach the solution with different numerical methods for unsteady flow such as Finite Element Method [4], Upwind Conservative Scheme [5, 6, 7, 8, 9, 10], Finite Difference Method with MacCormack Scheme [11], etc.

Open channels are channels where the flow has a surface exposed to the atmosphere. In order to model water flow on such channels, this paper used Saint-Venant equations as the governing equation. To solve the governing equations, the Trapezoidal method is used to integrate the equations, then standard step method produces the sketch of steady river flow surfaces as result. The effect of sluice gates to the surface will also be discussed.

This paper is organised as follows, in Section 2 the model of open channel flow is presented. In Section 3 the standard step method is elaborated for approximating water surface profile. Moreover, the applications of model into study case Manggarai sluice gate in Ciliwing river, Indonesia is given in Section 4. The last section, the conclusion of this paper is drawn in Section 5.

### 2. Open channel flow

In general, there are two types of open channel, prismatic channel and non-prismatic ones. Prismatic channels are channels which cross-section shape and bottom slope are constant, usually man-made such as canals, flumes, normalised rivers, etc. Otherwise, it is non-prismatic such
as natural river [12]. For simplicity, the channels used in the model of this paper is considered prismatic. There are several cross-section shapes which have been applied to made prismatic channels such as rectangular, trapezoidal, triangular and circular channel. Figure 2 presents an illustration of a trapezoidal channel cross-section.

![Figure 2. An illustration of a trapezoidal open channel cross-section.](image)

Figure 2 shows several important parameters of an open channel flow. $A$ is the flow area, $B$ is the top width of the flow, $P$ is the wetted perimeter of the flow, $b$ is the bottom width of the channel, $H$ is the flow depth relative to the channel bottom, $m$ is the side slope of the channel which is defined as $\cot \phi$, $Z$ is the channel bottom elevation relative to the datum, and $h$ is the water surface elevation relative to the datum. There is another parameter which is hydraulic radius $R = A/P$ and hydraulic depth $D = A/B$. Rectangular channels can be considered trapezoidal with $m = 0$. Meanwhile, triangular channels can be considered trapezoidal with $b = 0$. However, circular channels need entire different equations which are explained in [13].

2.1. Saint-Venant equations

Saint-Venant equations are derived by Adhmar Jean Claude Barr de Saint-Venant to model shallow water flow conditions. The equations can also be derived from Navier-Stokes equations [14] which is used to model general fluid motion, with the assumptions for shallow water and open channel flow conditions [15, 16], which are:

(i) the flow is one dimensional,
(ii) the fluid is incompressible,
(iii) the vertical acceleration is negligible,
(iv) the pressure distribution is hydrostatic,
(v) the flow’s body forces are gravity and friction,
(vi) the channel bottom slope is small and constant, causing constant friction loss, and
(vii) the cross-section shape is consistent, invulnerable to erosion and deposition.

The formula of 1-D Saint-Venant equations for open channel flow in steady condition can be written as:

$$\frac{dE}{dx} = \frac{d}{dx} \left( h + \frac{Q^2}{2g \cdot A^2} \right) = -S_f,$$  \hspace{1cm} (1)
where \( E \) is the total head energy, \( g \) is the gravitational acceleration, \( Q \) is the discharge rate of the flow, and \( S_f \) is the friction slope caused by channel resistance.

### 2.2. Normal depth and critical depth

According to the governing equation (1), the total energy \( E \) is a function of flow depth \( H \) since \( Q \) is a given constant and \( A \) depends on \( H \). This means the specific energy \( E_s \), the energy relative to the bottom of the channel, is also a function of \( H \), as is shown by the following equation.

\[
E_s = H + \frac{Q^2}{2g \cdot A^2} \tag{2}
\]

For a given discharge \( Q \), the specific energy \( E_s \) as a function of \( H \) as given (2) is plotted in figure 3. The curve shows that for each value of \( E_s \), there are two possible depths which are called alternate depth. The depth of the extreme minimum point is called critical depth \((H_c)\). This means that for a given \( Q \), \( E_s \) will be minimum when the flow depth is equal to \( H_c \). Since \( H_c \) is an extreme minimum point, \( dE_s/dH \) at \( H_c \) should be equal to zero. This yields a formula presented in (3).

\[
E_s'(H_c) = 1 - \frac{Q^2 \cdot B}{g \cdot A^3} = 0
\]

\[
\frac{Q^2}{g} - \frac{A(H_c)^3}{B(H_c)} = 0
\]

By finding a reasonable root of (3), the value of \( H_c \) can be obtained.

Open channel flow can be classified into two cases in general, uniform flow and non-uniform flow. Uniform flow is an open channel flow where the flow velocity and depth do not change along a certain length of the channel. The illustration of uniform flow is shown in figure 4. This happens when the channel resistance balances out the weight force, hence the resultant force is zero and there is no acceleration. This means the flow velocity does not change anymore along the channel and becomes constant, so does the flow depth. Such depth is referred to as normal depth \((H_n)\). The absence of changes of depth and velocity transform (1) to:

\[
\frac{dZ}{dx} = -s = -S_f, \tag{4}
\]

where \( s \) is the slope of the channel.
It can be seen that (4) involves the channel resistance. There are several equations which can be used to model the channel resistance, with each equation uses different approaches and various variables. For the model in this paper, Manning equation \[17, 18\] is used. The formula of Manning equation is:
\[
S_f = \frac{n_m^2 \cdot Q^2}{R^{4/3} \cdot A^2},
\]
where \(n_m\) is Manning roughness coefficient. Substituting (5) into (4) will give us:
\[
\frac{R(H_n)^{4/3} \cdot A(H_n)^2 \cdot s}{n_m^2} - Q^2 = 0,
\]
where \(H_n\) is the root of (6).

The relation between \(H_n\) and \(H_c\) determines the slope type. The slope is called mild slope if \(H_n > H_c\), steep slope if \(H_n < H_c\), or critical slope if \(H_n = H_c\). Thus, it can be concluded that there are two or three zones separated by either both of \(H_n\)-line and \(H_c\)-line or \(H_c\)-line only. For an easier understanding, the surface profile of the flow in each zone on all types of channel slope has been analysed and sketched based on (1) by \[13, 12\].

### 3. Standard step method for sketching surface profile

As explained in the previous section, (1) is the governing equation of the model in this paper. However, (1) is a differential equation. Thus, integration of (1) is needed to obtain the solution.

Suppose the domain \(x\) is discretised into \(N\) partitions. Thus there will be \(N + 1\) points with the interval of \(\Delta x\) unit length. The integration of (1) can be written as:
\[
\int_{E_i}^{E_{i+1}} dE = - \int_{x_i}^{x_{i+1}} S_f \, dx,
\]
where \(E_{i+1}\) and \(E_i\) is the total head at point \(x_{i+1}\) and \(x_i\). The integral of \(S\) with respect to \(x\) from \(x_i\) to \(x_{i+1}\) can be approximated numerically with various methods \[19, 20\]. In this paper, the approximation is done with trapezoidal rule. The approximation formula of the trapezoidal rule can be written as follows:
\[
\int_a^b f(x) \, dx = \frac{\Delta x}{2} \left(f(a) + f(b)\right).
\]
Applying (8) into (7) and further elaborating the total head and the friction slope with Manning equation, then rearranging it, yields:
\[
\left(Z_{i+1} + H_{i+1} + \frac{Q^2}{2g \cdot A_{i+1}^2}\right) - \left(Z_i + H_i + \frac{Q^2}{2g \cdot A_i^2}\right) + \frac{\Delta x}{2} \left(\frac{n_m^2 \cdot Q^2}{R_i^{4/3} \cdot A_i^2} \right) + \frac{\Delta x}{2} \left(\frac{n_m^2 \cdot Q^2}{R_{i+1}^{4/3} \cdot A_{i+1}^2}\right) = 0.
\]
which its root is approximated by False Position method.

There are several ways to generate the sketch of water surface profile under steady state condition such as standard step method and Direct Step method if the known parameters are \( H \) at one point and \( Q \), or calculating the water depth at all points simultaneously using one from various methods to solve nonlinear equations system \[13, 12\]. The one that is implemented in this paper is the standard step method. The basic idea of this method is: to calculate \( H \) at all points one by one, starting from the point adjacent to point \( x_0 \) where \( H_0 \) is known, to either upstream or downstream of the flow until the predetermined distance is reached. The algorithm of the main procedure which implements the standard step method is presented by algorithm 1.

**Algorithm 1** Standard step method as the main procedure to sketch the water surface at all points of \( x \).

1: **procedure** MAIN  
2: Define the value of \( g, Q, m, b, s, n_m, L, N, Z_0, \) and \( H_0 \)  
3: \( \Delta x \leftarrow L/N \)  
4: **for** \( i \leftarrow 0 \) to \( N \) **do**  
5: \( x_i \leftarrow i \cdot \Delta x \)  
6: \( Z_i \leftarrow Z_0 - s \cdot x_i \).  
7: **end for**  
8: \( H_n \leftarrow \text{FALSE POSITION}(6), 0, b \)  
9: \( H_c \leftarrow \text{FALSE POSITION}(3), 0, b \)  
10: \( y_0 \leftarrow Z_0 + H_0 \)  
11: **for** \( i \leftarrow 0 \) to \( N - 1 \) **do**  
12: \( H_{i+1} \leftarrow \text{FALSE POSITION}(9), (1-r)H_i, (1+r)H_i \)  
13: \( y_{i+1} \leftarrow Z_{i+1} + H_{i+1} \)  
14: **end for**  
15: Plot \( Z, Z + H_n, Z + H_c, \) and \( y \) to generate the surface profile sketch  
16: **end procedure**

Before the calculation begins, the gravitational acceleration, the constants properties of the channel, the discretisation setting, and the channel bottom and water level at initial point need to be defined. Then the position and the channel bottom of each point are calculated. \( H_n \) and \( H_c \) are also calculated by finding the roots of (6) and (3) with False Position method \[19, 20\] or any other method for each respectively in order to make the flow type and the surface profile can be analysed from the generated sketch. Finally, the water depth of each point is calculated one by one by finding the root of (9) also with False Position method and plot the result to generate the sketch of the surface profile.

3.1. Implementation of Standard Step Method

For example, let us simulate a flow with \( Q = 10 \) \( m^3/s \) on a trapezoidal channel with properties: \( b = 2.5 \) \( m \), \( m = 1.5 \), \( n_m = 0.015 \) and \( s = 0.0005 \). The known water level at initial point is \( H_0 = 2 \) \( m \). The surface profile produced for 5 \( km \) at the upstream and downstream of the initial point are presented in figure 5.

4. Application in sluice gate operation (Case study: Manggarai sluice gate in Ciliwung River)

4.1. Backwater caused by a sluice gate

Under normal circumstance, open channel flow always flows at normal depth since there are no disturbances, regardless it flows on a steep slope channel or a mild one. However, when a
Figure 5. The produced surface profile at the: a) upstream and b) downstream of the initial point.

sluice gate is installed on the middle of the channel, and the opening of the gate is set below
$H_n$ for steep slope (or below the alternate depth of $H_n$ for mild slope), backwater arises at the
upstream part of the gate.

Figure 6. Plot of $H$ and $E_s(H)$
of a flow with $Q = 290$ $m^3/s$
on Ciliwung River near Manggarai sluice gate. The plot shows that there is another depth which has
the same amount of $E_s$ with $H_n$, and the depth below that requires more $E_s$.

Ciliwung River’s cross-section near Manggarai sluice gate has properties: $s = 0.0005,$ $n_m = 0.013$, $b = 14$ m, and $m = 2.8585$. Suppose a flow with $Q = 290$ $m^3/s$ flows on
it. These parameters are used to solve (3) to get $H_c$ and solve (6) to get $H_n$. This flow has
$H_n = 3.714$ m and $H_c = 2.876$ m. Since $H_c < H_n$, Ciliwung River is considered a mild slope.
The specific energy diagram of the flow presented in figure 6 shows that for depths lower than
the alternate depth of $H_n$, the required specific energy is higher. Presumably, the current is
flowing at normal depth. If a sluice gate was installed and set lower than the alternate depth
of $H_n$, the current flow will not have sufficient energy to pass the gate. In this situation, a
choke happens at the gate, where the water depth at the upstream side of the gate start to rise
slowly until the required energy to pass the gate is satisfied, which is when the alternate depth
of the opening height is achieved. In terms of algebra, both depths are the roots of (2); where
the gate opening is the root below $H_c$ and the backwater depth is the root above $H_c$. Thus,

1 Manning roughness coefficient for concrete.
2 The annual highest discharge of Ciliwung River in 2014.
the backwater depth can be approximated by using False Position method and specifying the interval higher than $H_c$.

Sketching the surface profile around the sluice gate is possible by using the approximated backwater depth as the initial depth $H_0$ for the flow at the upstream side of the gate, and the gate opening $H_g$ as the initial depth for the downstream side. As an example, let us consider a case which the sluice gate was set with two different openings, 1.5 m and 1 m. The surface profile on the upstream part of the gate was calculated by the standard step method, and the results are presented in figure 7. The sketches shows that the surface profile of the flow is categorised as M1. Note that these results are strongly affected by the river discharge. In the next subsection, this aspect will be studied in great detail.

Figure 7. The surface profile on the upstream part of the gate, which are computed under the discharge level $Q = 290 \, \text{m}^3/\text{s}$ with sluice gate opening a) 1.5 m and b) 1 m.

4.2. Regulating the operation of sluice gates

From figure 7, it can be seen that lowering the gate by only 0.5 m produces a large backwater rise 8.962 m. In addition, the backwater reaches across over 25 km of the river meanwhile it only reaches around 10 km in the higher gate opening. If the maximum depth of the channel can contain lower than the produced backwater, the water will spill over the channel’s side, which may inundate to the surrounding area. Therefore, a minimum limit of the opening gate must be set to avoid the problem.

After Ciliwung normalisation, the maximum depth of Ciliwung River is 7.044 m around Manggarai sluice gate. In other words, the maximum $H_b$ allowed is 7.044 m. Since $H_b$ is the alternate depth of $H_g$, then minimum $H_g$ can be calculated by doing the reverse processes which is done in Sub-section 4.1, where the alternate depth of the maximum $H_b$ is calculated. The minimum $H_g$ for several given discharges of Manggarai sluice gate are presented in table 1.

For a constant $H_b$, a higher $Q$ flow produces higher flow velocity since the flow area is constant, thus more energy is required as shown in table 1. This also applies for a constant gate opening that more amount of energy is required to pass the gate for a higher $Q$ flow than a lower one. In other words, for the same gate opening, higher $Q$ flow produces higher $H_b$. Thus, as the result of setting the maximum allowed $H_b$, the height of minimum allowed $H_g$ increases as $Q$ increases as in table 1.

5. Conclusion

Steady state solution of Saint-Venant equations to model Ciliwung River flow’s surface profile under certain discharge has been calculated using the standard step method. The surface profile
Table 1. The minimum gate openings allowed for several given discharge rates at Manggarai sluice gate, Ciliwung River.

| $Q$ (m$^3$/s) | Min $H_g$ (m) | $E_b$ (m) |
|---------------|---------------|-----------|
| 10            | 0.060309      | 7.04409  |
| 20            | 0.119698      | 7.04435  |
| 30            | 0.178226      | 7.04479  |
| 40            | 0.235947      | 7.04541  |
| 50            | 0.292907      | 7.04621  |
| 60            | 0.349152      | 7.04718  |
| 70            | 0.404723      | 7.04832  |
| 80            | 0.459656      | 7.04965  |
| 90            | 0.513986      | 7.05115  |
| 100           | 0.567744      | 7.05282  |
| 200           | 1.07945       | 7.0793   |
| 300           | 1.55669       | 7.12342  |
| 400           | 2.01128       | 7.1852   |
| 500           | 2.45056       | 7.26462  |

of the flow has also been studied. The results show that an increase of $Q$ produces higher $H_n$, in other words, the rise of the water surface. The flow with $Q = 55$ m$^3$/s has $H_n = 1.532$ m meanwhile the flow with $Q = 290$ m$^3$/s has $H_n = 3.714$ m. Moreover, the placement of a sluice gate on a channel produces a backwater if the opening is low. Lowering the gate causes the backwater to rise. Therefore, there are minimum gate openings $H_g$ for given $Q$, and maximum backwater depth $H_b$. The model can be used to calculate the minimum $H_g$. The minimum $H_g$ for Manggarai sluice gate on Ciliwung River with max $H_b = 7.044$ m are 11.97 cm, 34.92 cm, and 155.67 cm for 20 m$^3$/s, 60 m$^3$/s, and 300 m$^3$/s discharge rates.

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