PAK BENG HYDROPOWER PROJECT

Numerical Simulation of Sediment Movement in the Ship Channel of Pak Beng HPP Downstream

KUNMING ENGINEERING CORPORATION LIMITED

September 2015
Numerical Simulation of Sediment Movement In the Ship Channel of Pak Beng HPP Downstream

Research Report

College of Water Resources and Hydropower Engineering, Wuhan University
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1 INTRODUCTION

1.1 BACKGROUND

Mekong River is a vital international river in the Southeast Asia. Pak Beng HPP locates on the upstream of Mekong River, which ranks the first cascade station of hydropower development planning of Mekong River. In 1957, Committee for the Coordination of Mekong River Basin (Mekong Committee, MC), which consists of Vietnam, Laos, Cambodia and Thailand, etc, was set up under the support of Economic Commission for Asia and the Far East (ECAFE, replaced by Economic and Social for Asia and Pacific in 1987). Since 1963, MC has carried out a large amount of researches for the appropriate development of Mekong River.

In 1994, 《Mekong River Development Programme of Hydropower Engineering》, in which eleven cascades hydropower station were planned, was finished by Mekong Committee Secretariat. According to this programme, Pak Beng HPP was recommended as the first cascade station. To satisfy the electric power demand of the downstream region and the shipping demand through supplying facilities for ships passing dam, such as the navigation lock, is the purpose of cascade station development of Mekong main stream. Meanwhile, it also tends to provide convenience for improvement of the riverway shipping.

However, there will be a siltation that harmful for shipping at the entrance of the lower approach channel after Pak Beng Station begin running; Therefore, the study on riverbed and channel situation of downstream of the dam should be proved to guarantee the navigation moving normally after the station begin running.

1.2 COMPILATION BASIS

(1) Specification on compiling preliminary design report of water conservancy and hydropower projects. DL5021-93;

(2) Specification on water conservancy computation of hydro- electric projects. DL/T 5105-1999;

(3) Specification for sediment design of hydropower and water conservancy project. DL/T 5089-1999.

1.3 STUDY OBJECTIVE AND SIGNIFICANCE

The approach channel safety could have some problems in the junction area of the Pak Beng Power Station due to the sediment deposition and flow velocity. Thus, the
main tasks of this research are: to study the sediment deposition in the approach channel during Pak Beng Station running, and to analyse and evaluate the scouring effect of the channel silt-releasing sluice, and then to work out a rational running pattern to provide a reference for the approach channel design and reservoir reasonable dispatching.

1.4 STUDY CONTENT

Based on Task Document of Sediment Simulation of Pak Beng hinge region delivered by Hydrochina Kunming Engineering Corporation, We used the sediment three-dimensional mathematical model to simulate the erosion and siltation of the lower approach channel, followed the study of flow pattern, flow direction and flow velocity distribution in the lower approach channel. An optimal design scheme, running pattern and engineering measures for reducing approach channel siltation should be proposed through studying the sediment siltation situation (sedimentation quantity, location, form and obstructing navigation or not) under various flows at the entrance of the lower approach channel.
2 GENERAL INFORMATION OF PROJECT AND HYDROLOGY

2.1 PROJECT INTRODUCTION

(1) Site

Pak Beng HPP as the first cascade station of the hydropower development planning of Mekong River, lies on the upstream of the river. It locates in the county named Pak Beng, which stands in Oudumxay Province north of Laos. On the upstream, which 14 kilometers away from Pak Beng County, is the dam site. It is shown in Figure 2.1.

![Pak Beng Station geographical location](image)

**Fig. 2.1** Pak Beng Station geographical location

Although the main task of this project is to generate electricity, it still also has the demand of comprehensive utilization for navigation and fish passing.

(2) Basic architecture
The project is the second type engineering which can be classified into I Class, the main hydraulic structure is level 2. And this power station was developed in the river bed. The junctional project consists of concrete gravity dam, flood-discharge and sediment-erosion sluice, power station in river bed, sediment erosion under-sluice, ship lock and fish canal (Figure 2.2). The reservoir is daily regulating, with normal pool level at 340 m and dead water level at 334 m. It has approximately $7.8 \times 10^8 \text{m}^3$ total storage capacity and 912 MW installed capacity.

**Fig. 2.2  Pak Beng Power Station horizontal design**

The diversion dam is a concrete gravity dam, with 346 m crest elevation, about maximum 69 m height, and 894.5 m crest length, the main discharge structure of which is the outlet sluice. The preliminary plan is to layout fourteen 15 m width and 23 m height sluice holes.

Besides, the type of the power house is river-bed. And the preliminary plan is to arrange sixteen tubular turbines with 57 MW unit capacity. In all 8 under-sluices will be placed between every two unites.

The navigation structure is one-way and one-step ship lock. And the ship lock level is IV, with the maximum weight of ship passing is 500 t. The maximum working head of the lock is 32.48 m. The total tonnage of one-way passing ship is about $150 \times 10^4$ t per year. In order to be convenient for sediment
erosion of approach ship channel, there arranged one hole ship channel sediment erosion sluice on the right of the ship lock.

The lower approach channel, sitting on the right of reservoir downstream, is about 700 m length, 100 m width and 302.45 m bottom elevation.

(3) Flood control constructure and operation mode

Discharge structure is composed of silt-releasing sluice and sediment erosion under-slui ce. More precisely, as the main water discharge and sediment outlet structure, the silt-releasing sluice is on the shoaly land on the right bank of the river. The sediment erosion under-slui ce that used to discharge suspended sediment silt at the water intake of the power house is on the house dam section, insuring the front of the house intake is clean. The preliminary plan is to arrange fourteen silt-releasing sluices.

According to the programme, water level should be maintained at 340 m as possible when reservoir inflow is lower than 13200m$^3$/s as the flow of three years frequency flood. Accordingly the navigation should be carried out through flood-discharge and sediment-erosion sluices (twelve holes), sediment erosion under-sluices, flow passing turbine units and ship locks.

While, the lock should stop running and still the reservoir water level should be maintained at 340 m as possible when the inflow is higher than 13200m$^3$/s, but lower than 14900m$^3$/s as five years frequency flood. Through silt-releasing sluice (twelve holes), sediment erosion under-sluices, flow passing turbine units and sediment erosion sluices (two holes) in the channel water can be discharged.

2.2 INTRODUCTION OF HYDROLOGY AND SEDIMENT CONDITIONS

2.2.1 Hygrology

Analysis based on the location of Pak Beng Station dam and the storm runoff character of Mekong River Basin shows that, in general, rainy season comes at May and the main rainy period is from June to October when rainfall is relatively concentrated. It earliest appears at the early ten days of July, and the last ten days of October. However, most of time, it appears at August. According to the order of magnitude of flood, June and November can be defined as the transition period anterior and posterior to the flood season, respectively.

The series statistics of daily runoff, considering regulation of Xiaowan and Nuozhadu
reservoirs from June 1960 to May 2004 is adopted as runoff data of Pak Beng Station dam site. While the series of monthly average runoff is from 1960 to 2004 with a annual average flow 3160 m$^3$/s of Pak Beng Station dam site. Annual average flow is shown in Table 2.1. The flow of 2 years frequency flood is 11600 m$^3$/s, and 3 years frequency flood is 13200 m$^3$/s.

### Table 2.1 Monthly mean flow over years results of Pak Beng Station damsite

<table>
<thead>
<tr>
<th>Item \ Month</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average flow (m$^3$/s)</td>
<td>1350</td>
<td>1060</td>
<td>912</td>
<td>973</td>
<td>1440</td>
<td>2740</td>
<td>5450</td>
<td>7770</td>
<td>6770</td>
<td>4490</td>
<td>2950</td>
<td>1870</td>
<td>3160</td>
</tr>
<tr>
<td>Distribution ratio (%)</td>
<td>3.56</td>
<td>2.80</td>
<td>2.41</td>
<td>2.57</td>
<td>3.80</td>
<td>7.23</td>
<td>14.4</td>
<td>20.5</td>
<td>17.9</td>
<td>11.8</td>
<td>7.78</td>
<td>4.93</td>
<td>100</td>
</tr>
</tbody>
</table>

#### 2.2.2 Design flood at dam site

The dam site of Pak Beng Station is between Qingsheng Hydrologic Station and Lang Bolabang Hydrologic Station. Thus, the design flood at dam site is calculated through interpolating the results of frequency flood at interval area. Results are shown in Table 2.2.

### Table 2.2 Results of design flood at dam site of Pak Beng Station

<table>
<thead>
<tr>
<th>P (%)</th>
<th>Flood peak flow (m$^3$/s)</th>
<th>Upstream water level (m)</th>
<th>Downstream water level (m)</th>
<th>Flow of flood discharge sluice (m$^3$/s)</th>
<th>Discharge flow of sediment erosion sluice (m$^3$/s)</th>
<th>Discharge of sediment erosion under sluice (m$^3$/s)</th>
<th>Flow rate passing turbines (m$^3$/s)</th>
<th>Total discharge flow of junction (m$^3$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.05</td>
<td>30200</td>
<td>343.54</td>
<td>342.86</td>
<td>28043</td>
<td>2157</td>
<td>30200</td>
<td></td>
<td>30200</td>
</tr>
<tr>
<td>0.1</td>
<td>28700</td>
<td>342.52</td>
<td>341.87</td>
<td>26650</td>
<td>2050</td>
<td>28700</td>
<td></td>
<td>28700</td>
</tr>
<tr>
<td>0.2</td>
<td>27000</td>
<td>341.35</td>
<td>340.73</td>
<td>25071</td>
<td>1929</td>
<td>27000</td>
<td></td>
<td>27000</td>
</tr>
<tr>
<td>0.5</td>
<td>24800</td>
<td>339.81</td>
<td>339.22</td>
<td>23029</td>
<td>1771</td>
<td>24800</td>
<td></td>
<td>24800</td>
</tr>
<tr>
<td>1</td>
<td>23100</td>
<td>338.59</td>
<td>338.02</td>
<td>21450</td>
<td>1650</td>
<td>23100</td>
<td></td>
<td>23100</td>
</tr>
<tr>
<td>2</td>
<td>21400</td>
<td>337.33</td>
<td>336.79</td>
<td>19871</td>
<td>1529</td>
<td>21400</td>
<td></td>
<td>21400</td>
</tr>
<tr>
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<td>18900</td>
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<td>334.97</td>
<td>17550</td>
<td>1350</td>
<td>18900</td>
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<td>18900</td>
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<tr>
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<td>17000</td>
<td>334.02</td>
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<td>331.93</td>
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<td>14900</td>
<td></td>
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<td>33.33</td>
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<td>340.00</td>
<td>330.29</td>
<td>6672</td>
<td>808</td>
<td>5720</td>
<td>13200</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>11600</td>
<td>340.000</td>
<td>328.61</td>
<td>5004</td>
<td>876</td>
<td>5720</td>
<td></td>
<td>5720</td>
</tr>
</tbody>
</table>
2.2.3 Sediment

(1) Sediment amount at dam site

In natural condition, annual average suspended load at dam site is $14268 \times 10^4$ t. Moreover, there are no materials about bed load in the hydrologic stations on the main stream of Lancang-Mekong River. Taking account of the accordance with the results of Xiaowan, Nuozhadu and Jinghong on upstream, the bed-suspension ratio was adopted to calculate the bed load sediment amount is 3%, corresponding to the result is $428 \times 10^4$ t.

Xiaowan and Nuozhadu hydropower station are giant reservoirs with extremely remarkable sediment controlling effect among the 8 cascade stations on the middle and lower reach of Lancang River. At present, stations at Manwan and Dachaoshan have been built. And impoundment and electricity generation of Jinghong station and Xiaowan station began in 2008 and 2009, respectively. While, predictly Nuozhadu Station is going to run in 2011. Pak Beng Station will be built up and put into production after Xiaowan dan Nuozhadu from the perspect of time sequence of the stations development. Suspended sediment amount at Pak Beng Station is $3799 \times 10^4$ t that constituting 26.6% of natural situation, for the impact of sediment retaining from cascade stations upstream,. Bed load sediment is all intercepted before reach to Jinghong dam, thereby bed load sediment at Pak Beng dam site only coming from the region between Jinghong and Pak Beng. And the quantity estimating at 3% of the bed load sediment amount is $103 \times 10^4$ t, which occupying 24.2% of natural condition.

(2) Size of sediment distribution

Figure 2.4 indicates the size distribution curve of suspended sediment.
Figure 2.4 Suspended sediment size distribution curve of Pak Beng Station
3 3D MATHEMATICAL MODEL OF FLOW AND SEDIMENT

3.1 WATER FLOW MODEL

The calculation was carried out on the orthogonal curvilinear coordinate system on the plane, and $\sigma$ coordinate in the vertical direction. Controlling equations are obtained as below basing on the quasi static force assumption. (Blumberg and Mellor., 1987):

Water level equation is:

$$\partial_t \left( m \lambda \zeta \right) + \partial_x \left( m_y \lambda H u \right) + \partial_y \left( m_x \lambda H v \right) + \partial_z \left( m w \right) = 0 \quad (3-1)$$

Momentum equations are:

$$\partial_t \left( m \lambda Hu \right) + \partial_x \left( m_y \lambda Huu \right) + \partial_y \left( m_x \lambda Huv \right) + \partial_z \left( mwu \right) - mf_e \lambda Hv$$

$$= -m_y \lambda H \partial_x \left( g \zeta \right) - m_y \lambda H \partial_x \left( g \lambda H \int_{-z}^{1} bdz \right) - gm_y \lambda H \left( \partial_x \zeta - (1-z) \partial_x \left( \lambda H \right) \right) b + \partial_z \left( m \frac{A_y}{\lambda H} \partial_x u \right) \quad (3-2)$$

$$\partial_t \left( m \lambda Hv \right) + \partial_x \left( m_y \lambda Huv \right) + \partial_y \left( m_x \lambda Hvv \right) + \partial_z \left( mwv \right) + mf_e \lambda Hu$$

$$= -m_x \lambda H \partial_y \left( g \zeta \right) - m_x \lambda H \partial_y \left( g \lambda H \int_{-z}^{1} bdz \right)$$

$$+ gm_x \lambda H \left( \partial_y \zeta - (1-z) \partial_y \left( \lambda H \right) \right) b + \partial_z \left( m \frac{A_x}{\lambda H} \partial_y v \right) \quad (3-3)$$

Where $f_e$ is determined by the formula below:

$$mf_e = mf - u \partial_y m_x + v \partial_x m_y \quad (3-4)$$

In the formulas above,

- $t$ is time;
- $\lambda$ is transform coefficient from standard $\sigma$ coordinate to LCL coordinate;
- $H$ is total water depth;
- $u$, $v$ and $w$ are horizontal velocities in $x$, $y$ and $z$ direction under orthogonal curvilinear coordinate system;
- $m = m_x m_y$, $m_x$, $m_y$ is transform coefficient under Jacobian orthogonal
curvilinear coordinate system;

\( \zeta \) is water level;

\( f \) is Coriolis force parameter;

\( f_e \) is intermediate variable;

\( A_v \) is vertical turbulent diffusion coefficient;

\( Q_u \) and \( Q_v \) are source and sink terms of momentum equation, respectively, including horizontal diffusion and so on;

\( b \) is buoyancy as the ratio of density deviation and density reference value.

This model used \( \frac{1}{2} \) steps Mellor-Yamada turbulence model to solve vertical turbulent diffusion item, in order to effectively simulate the impact that stratification of water body has on vertical mixing intensity. The equations are:

\[
\frac{\partial}{\partial t} (m_x m_y \lambda H q^2) + \frac{\partial}{\partial x} (m_x \lambda H u q^2) + \frac{\partial}{\partial y} (m_x \lambda H v q^2) + \frac{\partial}{\partial z} (m w q^2) = \nabla \left( \frac{m_x m_y}{\lambda H} A_v \nabla q^2 \right) + Q_q + 2 \frac{m_x m_y}{\lambda H} A_v (\nabla u^2 + (\nabla v)^2) + 2 m g A_v \nabla b - 2 m \lambda H (B_l)^{-1} q^3
\]  

(3-5)

\[
\frac{\partial}{\partial t} (m_x m_y \lambda H q^2 l) + \frac{\partial}{\partial x} (m_x \lambda H u q^2 l) + \frac{\partial}{\partial y} (m_x \lambda H v q^2 l) + \frac{\partial}{\partial z} (m w q^2 l) = \nabla \left( \frac{m_x m_y}{\lambda H} A_v \nabla q^2 l \right) + Q_q + m \frac{m_x m_y}{\lambda H} E_l A_v ((\nabla u)^2 + (\nabla v)^2) + m g E_l A_v \nabla b - m \lambda H B_l^{-1} q^3 (1 + E_l (\kappa L)^{-2} l^2)
\]  

(3-6)

\[
L^{-1} = H^{-1} (z^{-1} + (1 - z)^{-1})
\]  

(3-7)

\[
A_v = \phi_v q l = 0.4 (1 + 36 R_q)^{-1} (1 + 6 R_q)^{-1} (1 + 8 R_q) q l
\]  

(3-8)

\[
A_b = \phi_b q l = 0.5 (1 + 36 R_q)^{-1} q l
\]  

(3-9)
\[ R_q = \frac{gH}{q^2} \left( \frac{\partial b}{\partial z} \right) \frac{l^2}{H^2} \]  

(3-10)

Where

- \( q^2/2 \) is turbulence intensity;
- \( \kappa \) is Karman constant;
- \( l \) is mixing turbulence length;
- \( L \) is macroscopic turbulence length;
- \( B_1, E_1, E_2 \) and \( \frac{M}{M_0} = A_0 + A_1 e^{-t/T} \) are empirical constants;
- \( Q_q \) and \( Q_f \) are source and sink terms, including horizontal diffusion item and so on;
- \( \phi_v \) and \( \phi_b \) are stability functions that reflect the promoting effect or inhibitory effect from vertical stratification of density versus vertical mixing.

Vertical diffusion coefficient \( A_q \) is usually equal to \( A_v \).

### 3.2 SEDIMENT MATHEMATICAL MODEL

#### 3.2.1 Suspended load concentration equation

\[
\partial_t (mHs_k) + \partial_x \left( m_y H u_{s_k} \right) + \partial_y \left( m_x H v_{s_k} \right) + \partial_{\sigma} \left( m_{w_k} \right) = \partial_x \left( mH^{-1} \varepsilon_s \partial_{\sigma} s_k \right) + \partial_y \left( m y \varepsilon_s H \partial_{\sigma} s_k \right) + \partial_{\sigma} \left( m \omega_{sk} s_k \right) 
\]

(3-11)

Where

- \( s_k \) is suspended load concentration need to be simulated;
- \( \omega_{sk} \) is sediment settling velocity, determined by Wushui formula;
- \( \varepsilon_s = \frac{\nu}{\sigma_s} + \nu \), \( \sigma_s \) is Schmidt number that values vary in different models.

However, the range is usually between 0.5 and 1.0. And we use 0.8, which suggested by Xia Yunfeng in 2002.
3.2.2 River bed deformation equation

River bed deformation arose by suspended sediment is determined by the deposition flux $D_b$ and sediment pick-up flux $E_b$ on the bottom interface. The equation is:

$$
\frac{\partial z_b}{\partial t} = -D_b + E_b
$$

(3-12)

where

- $z_b$ is river bed elevation;
- $\gamma_s'$ is dry unit weight;
- $D_b$ is deposition flux on river bed interface, $D_b = \omega_s s_b$;
- $E_b$ is sediment pick-up flux on this interface, $E_b = \omega_s s_b^*$, $s_b^*$ is capacity of sediment load in the vicinity of river bed surface.

3.2.3 Adjustment of grouping sediment carrying capacity and bed load grading

(1) Grouping gradation of sediment carrying capacity

Sediment carrying capacity of each group, which is near the bottom, need to be given to solve the 3D non-uniform model, as following:

$$
S^*_{bk} = P_{sk}^{-} s^*_b
$$

(3-13)

Determining how to give dealing method to the grading of grouping sediment carrying capacity on the bottom in 3D sediment mathematical models is the key point. This report used the following formula to solve this problem:

$$
P_{sk} = \frac{\alpha_b P_{uk} - \alpha_u P_{sk,u}}{\sum_{k=1}^{n} (\alpha_b P_{uk} + \alpha_u P_{sk,u})}
$$

(3-14)

where

$P_{sk}$ is grouping sediment carrying capacity grading;
\(P_{uk}\) is local bed sediment grading;

\(P_{uk,u}\) is suspended sediment grading of upwind direction of flow; \(\alpha_b\) and \(\alpha_u\) are the proportion of local grading and grading of upwind of flow, respectively. When it is scouring, \(\alpha_b\) is larger; while silting, \(\alpha_u\) is larger.

(2) Bed sediment grading adjustment

River bed sediment was generalized into surface, middle and bottom three layers using adjustment mode of Weizhilin(1997), which use stratification storage of bed sediment grading. The surface layer is sediment switching layer, and the middle is transitional layer, and the bottom is limit layer of sediment scouring. The thickness of each layer and average gradation are \(h_u\), \(h_m\), and \(h_b\) and \(P_{uk}\), \(P_{mk}\) and \(P_{bk}\). Scouring limitation happens on the surface layer. After a certain period of time, the surface and middle layer should move up and down to realize the height maintained at \(h_u\) and \(h_m\). The calculating process is as below:

Assuming surface sediment gradation is \(P_{uk}^{(0)}\) at the initial time of a certain period. And during this period, the thickness of siltation are \(\Delta Z_b\) and \(\Delta Z_{bk}\). Then at the end of this period, the gradation above the surface layer changed into:

\[
P_{uk}^{(1)} = \frac{(h^{(1)}_{ek} P_{uk}^{(0)} + \Delta Z_{bk})}{(h_u + \Delta Z_b)}
\]  \hspace{1cm} (3-15)

At the end of this period, adjust bed sediment grading of each group according to the silting and scouring conditions, basing on the equation above.

1) Deposition condition

① surface layer:

\[
P_{uk} = P_{uk}^{(1)}
\]  \hspace{1cm} (3-16)

② middle layer:
If $\Delta Z_b \geq h_m$, the new middle layer is upon the primary surface:

$$P_{mk} = P_{uk}^{(1)} \quad (3-17)$$

If $\Delta Z_b < h_m$, then

$$P_{mk} = (\Delta Z_b P_{uk}^{(1)} + (h_m - \Delta Z_b) P_{mk}^{(0)}) / h_m \quad (3-18)$$

3. bottom layer:

Thickness of the bottom layer: $h_b = h_b^{(0)} + \Delta Z_b \quad (3-19)$

If $\Delta Z_b \geq h_m$, then

$$P_{bk} = [(\Delta Z_b - h_m) P_{uk}^{(1)} + h_m P_{mk}^{(0)} + h_m (0) P_{bk}^{(0)}] / h_b \quad (3-20)$$

If $\Delta Z_b < h_m$, then

$$P_{bk} = (\Delta Z_b P_{uk}^{(0)} + (h_b P_{bk}^{(0)}) / h_b \quad (3-21)$$

2) Erosion

① surface layer:

$$P_{uk} = [(h_u + \Delta Z_b) P_{uk}^{(1)} - \Delta Z_b P_{uk}^{(0)}] / h_u \quad (3-22)$$

② middle layer:

$$P_{mk} = [(h_m + \Delta Z_b) P_{mk}^{(1)} - \Delta Z_b P_{mk}^{(0)}] / h_m \quad (3-23)$$

③ bottom layer:

$$h_b = h_b^{(0)} + \Delta Z_b \quad (3-24)$$

$$P_{bk} = P_{bk}^{(0)} \quad (3-25)$$

### 3.3 Computational Grid of Model

In order to simulate natural boundary of the river channel better, this model used curve orthogonal grid. Grids of the region are shown in Figure 3.1. There are 129 vertical and 130 horizontal grids. Another two factors of spacing between grids were considered: 1) change value of the mainstream flow direction, flow and sediment parameter; 2) calculation amount.

From the graph: 1) the boundaries of grids and approach ship channel are simulated well; 2) grids of main flow region are consistent with talweg of the channel; 3) grid
spacing is 6 to 17 m, however, as the mainly concern area of approach channel, the grids of this region are partially refined. While they are sparse on the upland between approach and silt-releasing sluice upper stream; 4) angle of the grid of the entire region is 80 to 90, meeting the requirement of orthogonal.

![Fig. 3.1 Sketch of calculation area and grids](image)

### 3.4 COMPUTATIONAL BOUNDARY CONDITIONS OF MODEL

1. **Water and sediment boundary condition of upstream**

   Computational water and sediment materials of Pak Beng upstream are from the sediment one-dimensional calculation of Hydrochina Kunming Engineering Corporation. In this calculation, the series of water and sediment are series of circular combination from 1984 to 1988 five years. It includes five years materials of ample, normal and dry period of water and sediment. And annual average sediment discharge, sediment concentration and sediment concentration of flood season of representative series are close to annual average values. Figure 3.2 and Figure 3.3 show the process of discharge (station entrance of lower stream) and sediment concentration of the station.

   The average values of the velocity and sediment concentration at the cross section of the inlet were adopted. It added approximately 100 m long region as transition area on the upper reach of the calculating area, to make sure that
the velocity and sediment concentration naturally distribute along the cross sections till the dam site.

![Fig. 3.2](image1.png) **Fig. 3.2** Curve of changing process of station discharge flow

![Fig. 3.3](image2.png) **Fig. 3.3** Curve of changing process of station sediment discharge

(2) Water and sediment boundary condition of downstream

Water level boundary: According to the *Research on Hydroelectricity Engineering in the River Bed Development of the Main Stream of Mekong*
River, in the dry season, The water level of Langbolabang Hydropower Station will be higher of 1m than the downstream water level of Pak Beng Power Station. Further more, their commissioning dates are close. Most part of the running period of Pak Beng Station after it built will be affected by Langbolabang Station, leading to the hard determination of the calculation of outlet water level. This time we used one-dimentional water and sediment mathematical model from the Hydrochina Kunming Engineering Corporation, with the results of water level process that considering the impact from Langbolabang Station. Figure 3.4 shows the water level process diagram of the outlet of the calculation area.

![Water level process diagram](image)

**Fig. 3.4  Curve of water level changing process at the outlet downstream**

Velocity and sediment concentration: set 0 gradient of the velocity and sediment concentration downstream along the flow direction.

(3) Soild wall boundary

The model used solid wall no slip condition, setting the velocity at the solid wall boundary nought. We used minimum water depth to determine the moving boundary through circular definition.

(4) Free surface boundary
The vertical gradient of flow variable on the free surface along the water surface is setted to zero. And boundary of sediment concentration is determined by the following formula: \( \omega_s s_k + \varepsilon_s \frac{\partial s_k}{\partial z} \bigg|_{z=z_s} = 0 \).

(5) Bed boundary

The key factor determining the results of sediment mathematical model is the sediment exchange between river bed and water flow. Nevertheless, at present, the exchange is expressed by sediment carrying capacity \( s_{bs} \) and sediment concentration near the river bed \( s_b \). We used

\[
s_b = s_p + s_{bs} \left(1 - e^{-\frac{a}{a}}\right)
\]

We used average sediment carrying capacity \( S_s \) of cross section to express bottom sediment carrying capacity, which assuming

\[
s_{bs} = \alpha S_s
\]

Assuming the distribution of sediment concentration in vertical direction obeys Rouse Equation, then:

\[
s_z = \left(\frac{a}{H-a}\right)^{\frac{\varepsilon}{\varepsilon_s}} \left(\frac{H-z}{z}\right)^{\frac{\varepsilon}{\varepsilon_s}}
\]

where \( \frac{\varepsilon}{\varepsilon_s} \) is suspended index, and \( H \) is water depth, and \( s_z \) is sediment concentration at \( z \), parameter \( a \) usually is \( a = 0.01H \sim 0.05H \). Average sediment carrying capacity is obtained by the vertical integration of the equation above:

\[
S_s = \frac{1}{H-a} \int_a^H \left(\frac{a}{H-a}\right)^{\frac{\varepsilon}{\varepsilon_s}} \left(\frac{H-z}{z}\right)^{\frac{\varepsilon}{\varepsilon_s}} s_{bs} dz
\]

through (3-27) and (3-29) we have:

\[
\alpha = \frac{H-a}{\int_a^H \left(\frac{a}{H-a}\right)^{\frac{\varepsilon}{\varepsilon_s}} \left(\frac{H-z}{z}\right)^{\frac{\varepsilon}{\varepsilon_s}} dz}
\]
through (3-27) and (3-30) we have:

\[ s_{b*} = \alpha S_* = \frac{H - a}{\int_a^H \left( \frac{a}{H - a} \right)^{5/3} \left( \frac{H - z}{z} \right)^{5/3} \, dz} \]  

(3-31)
4 CALCULATION ANALYSIS OF HYDRODYNAMIC PROCESS

Velocity of water flow in the approach channel is not only directly related to the security of navigation, but also to the silt condition of the channel. It is the chiefly premise of approach channel optimized design and station suitable dispatching to study velocity distribution regulation in different inflows from upper reach and different dispatching patterns of the reservoir.

Analyse from these two points below:

(1) It will supply references for the analysis of sediment silt condition in the approach channel to analyse the flow field of the approach channel under different flows and different dispatching patterns.

(2) It will supply references for the safe navigation of ships to analyse the variety of velocities in horizontal and vertical directions under different flows and different dispatching patterns.

4.1 ANALYSIS OF 3D FLOW FIELD CALCULATION IN THE WHOLE BASIN

Figure 4.1 shows velocity distribution on surface and at the place that has one fifth of the water depth distance to the bottom when reservoir inflows are 3160 m$^3$/s (annual average flow) and 13200 m$^3$/s (three years frequency flood flow), two types of working conditions.

Overall, in the calculated region, it shows downward trend from the surface to the bottom. However, the character of flow pattern in the river channel is different from that in the approach channel. Velocity distribution in the main channel is relatively regular; While, by contrast, there is relatively large reflux at the entrance when the velocity in the approach ship channel is relatively small; It results from the larger velocity of discharge flow rate in the channel. Then, the reflux will affect the normal running of the ship channel. The ship channel begins to discharge flood through closing the ship lock when the flow rate reaches to three years frequency flood. At this time, for the larger flow rate, the reflux becomes not obvious at the entrance and it vanishes to the end.
(a) Distribution of surface velocity under annual average flow

(b) Distribution of the place 1/5 of water depth near the bottom under annual average flow
Numerical Simulation of Sediment Movement in the Ship Channel of PAK BENG Hydropower Station Downstream

(c) Surface velocity distribution under three years frequency flood flow

(d) Flow velocity distribution at the place 1/5 of water depth near the bottom under three years frequency flood flow

Fig. 4.1 Calculation results of 3D velocity distribution
4.2 ANALYSIS OF FLOW FIELD IN NAVIGATION CHANNEL

It will be divided into two situations: normally navigate and discharge flood and erode sediment, in order to analyse the aquatic dynamic condition of the approach ship channel detailedly,

4.2.1 Flow Field under normal Navigation

According to the design scheme, when the reservoir inflow is no more than the flow of three years frequency flood 13200m$^3$/s, then the ship lock will run normally; Otherwise, the lower approach ship channel will discharge the flood and erode sediment. We took the minimum flow for navigation, annual average flow, 2 years frequency and 3 years frequency flow as the typical flow rates, analysed the velocity variation under different typical flow rates when the ship lock is running normally. The typical flow rates condition is shown in Table 4.1, and accompanying dispatching mode is in Table 4.2.

<table>
<thead>
<tr>
<th>Serial number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Character of the flow</td>
<td>Minimum navigation flow</td>
<td>Annual average flow</td>
<td>2 years frequency flow</td>
<td>3 years frequency flow</td>
</tr>
<tr>
<td>Measured data</td>
<td>1260</td>
<td>3160</td>
<td>11600</td>
<td>13200</td>
</tr>
</tbody>
</table>

Table 4.1  Typical flow rate condition

<table>
<thead>
<tr>
<th>Serial number</th>
<th>Flow rate (m$^3$/s)</th>
<th>Downstream water level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1260</td>
<td>311.60</td>
</tr>
<tr>
<td>2</td>
<td>3160</td>
<td>315.10</td>
</tr>
<tr>
<td>3</td>
<td>11600</td>
<td>328.61</td>
</tr>
<tr>
<td>4</td>
<td>13200</td>
<td>330.29</td>
</tr>
</tbody>
</table>
(1) Analysis of flow field in approach ship channel

The results show that when the ship lock is working normally, there are reflux at the entrance of the ship channel under all different flow rates. The ship channel can be approximately divided into three regions along the vertical direction: A. quasi-hydrostatic area which locates 300 meters away from the dam; B. central secondary reverse weak reflux area, which locates in the middle part of the ship channel; C. reflux area at the entrance, which locates in the range of 100 to 300 meters upper the ship channel. Figure 4.2 shows velocity distribution of the computation area under different flow rates when the ship channel is working.

When the ship lock is working, different flow rates have small effects on part of the ship channel in A. The velocity is usually lower than 0.1 m/s.

The secondary reverse reflux of B area locates in the middle part of the ship channel because of the shearing function of the reflux at the entrance. The range relatively fluctuates with the size of reflux at the entrance, and the velocity fluctuates between 0.1 and 0.3 m/s.

There is a backward flow near the entrance in C area. The region of the reflux is the largest when the discharge flow in the main river channel is from 3000 to 6000 m$^3$/s and the velocity of the reflux is around 0.2 to 0.5 m/s. The velocity of the reflux at the entrance increases with the increase of the discharge flow of the upstream of the channel. And the maximum velocity will reach to 0.8 m/s at the entrance, when the flow of the upstream reaches to 13200 m$^3$/s.
(a) Minimum navigation flow (1260 m³/s)

(b) Annual average flow rate (3160 m³/s)
Numerical Simulation of Sediment Movement in the Ship Channel of PAK BENG Hydropower Station Downstream

(c) 2 years frequency flow rate (11600 m³/s)

(d) Maximum navigation flow rate (13200 m³/s)

Fig.4.2 The velocity distribution in the approach ship channel under different typical flow rate

(2) Analysis of the horizontal velocity in the approach channel
The horizontal velocity, which is perpendicular to the main navigation line in the ship channel, directly impacts the security of ship navigation when the ship lock is working. According to the minimum navigation flow, annual average flow rate, 2 years frequency flow and maximum navigation flow four conditions mentioned above, we analysed the horizontal velocity in the approach channel as below. The maximum velocity limit of the water surface in the entrance area is shown in Table 4.3. The navigation construction is one-way and one-step ship lock, and is level IV.

### Table 4.3 Maximum velocity limit of the water surface in the entrance

<table>
<thead>
<tr>
<th>Ship lock level</th>
<th>Vertical velocity parallel the navigation line (m/s)</th>
<th>Horizontal velocity perpendicular to the navigation line (m/s)</th>
<th>Reflux velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I ~ IV</td>
<td>≤2.0</td>
<td>≤0.30</td>
<td>≤0.4</td>
</tr>
<tr>
<td>V ~ VII</td>
<td>≤1.5</td>
<td>≤0.25</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.3 shows the horizontal velocity distribution in the approach channel under four flow rates. The results indicate that the horizontal velocity distribution can be approximately divided into two regions, they are near dam and near the entrance in the approach ship channel. More precisely, the area near dam is the region from downstream of dam to the upstream of the entrance. And from 100 to 300 m on the upstream of the inner-entrance of the ship channel is the near entrance region. There is no adverse effect on ship navigation, for the velocity in the near dam region is small. Furthermore, there is reflux in this area; Thus, we only analysed the region near the entrance as below.

From Figure 4.3 we can see that the horizontal velocity in the ship channel increases with the rise of discharge flow in the channel. The velocity near the entrance is about 0.1 m/s under the minimum navigation flow and annual average flow rate. This velocity has no adverse effects on the security of navigation. Although the velocity of the reflux is lower than 0.4 m/s, it still can meet the requirement of navigation. But it shows the trend of navigation obstruction. While, the maximum horizontal velocity near the entrance can reach to between 0.5 and 0.6 m/s when the flow reaches 2 years frequency flow.
11600 m/s, which has exceed 0.3 m/s, the limit horizontal velocity near the entrance. Thus, it can not meet the navigation requirement.
(a) Minimum navigation flow rate (1260 m³/s)

(b) Annual average flow rate (3160 m³/s)
Fig. 4.3  Horizontal velocity distribution in the approach channel under different flow rate

(c) 2 years frequency flood flow rate (11600 m$^3$/s)

(d) Maximum navigation flow rate (13200 m$^3$/s)
4.2.2 Ship channel flow field analysis during sediment erosion period

According to the power station design programme, it totally arranged sixteen tubular turbines, which the minimum flow rate that can support all the hydroelectric units running is 6424 m$^3$/s. The ship lock will stop working, and the navigation scouring sluice will be opened when the reservoir inflow is more than 3 years frequency flood flow 13200 m$^3$/s.

As in Table 4.4, in order to study scouring effect under different flows, the following scouring calculation selected different flows of four levels which higher than the minimum flow rate that can support all the hydroelectric units running to comparatively analyse the scouring effect in the navigation channel under different flows. Detail working conditions are shown in Table 4.4.

**Table 4.4 Reservoir dispatch methods under different typical flow rates when approach ship channel is discharging flood and eroding sediment**

<table>
<thead>
<tr>
<th>Serial Number</th>
<th>Flow rate (m$^3$/s)</th>
<th>Sediment discharge amount (kg/s)</th>
<th>Downstream Water level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total discharge flow rate</td>
<td>Flow rate passing turbines</td>
<td>sediment erosion under sluice</td>
</tr>
<tr>
<td>1</td>
<td>7764</td>
<td>6424</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>9000</td>
<td>6424</td>
<td>335</td>
</tr>
<tr>
<td>3</td>
<td>11600</td>
<td>6424</td>
<td>876</td>
</tr>
<tr>
<td>4</td>
<td>13200</td>
<td>6424</td>
<td>808</td>
</tr>
</tbody>
</table>

(1) Flow field analysis in approach ship channel under different typical flows

The velocity is rising with the reflux at the ship channel entrance declining until vanish when the ship channel is under flood discharging and sediment scouring, in response. Figure 4.4 shows the flow field distribution in the approach ship channel. It illustrates that when the scouring flows are the same, the velocity is decreasing with the increase of discharge flow in the channel, for the backwater effect, which is derived from elevation of the water level at the exit of the approach ship channel, minished the velocity.
Numerical Simulation of Sediment Movement in the Ship Channel of PAK BENG Hydropower Station Downstream

(a) $Q=7764 \text{ m}^3/\text{s}$

(b) $Q=9000 \text{ m}^3/\text{s}$
(c) $Q=11600 \text{ m}^3/\text{s}$ (2 years frequency flood flow)

(d) $Q=13200 \text{ m}^3/\text{s}$ (3 years frequency flood flow)

Fig 4.4 Flow velocity distribution in approach ship channel under different flow rate when the channel is scouring

(2) Velocity variation regulation at typical cross section in the approach ship channel under different flow working conditions
Flow rate will have a key impact on scouring effect when the ship channel is under flood discharging and sediment scouring. Selected four typical sections (as in Figure 4.5) for the further analysis of the velocity variation in different discharge flow conditions.

Figure 4.6 shows flow rate distribution at the sections under four different flows, where $V$ is velocity, $L$ is distance to left bank of the channel, and $L_0$ is width of the channel.

The results show that average velocity at each section is between 0.55 and 0.82 m/s under four kinds of flow rates. To the same section, the average velocity decreases with the increase of discharge flow $Q$ in river channel. And every there is an increase of 2000 m$^3$/s in flow, there is a decrease of approximately 0.1 m in average velocity at the approach ship channel section when the flow in the river channel is between 7000 and 11600 m$^3$/s. Under the same flow rate, the patterns of different section velocity distribution are different. To be more precise, the velocities between I# and II# sections distributes uniformly along sections. However, there is a great difference in velocity distribution from III# to IV#, showing descending trend from left to the right bank. The difference is about 0.8 m/s.

**Fig. 4.5   Typical cross sections of approach navigation channel**
Numerical Simulation of Sediment Movement in the Ship Channel of PAK BENG Hydropower Station Downstream

(a) Section I #

(b) Section II #
Fig. 4.6  Velocity distribution on navigation channel under different flow conditions
4.3 SUMMARY

This chapter analysed dynamic conditions in the river channel of the reservoir downstream and the approach ship channel under different flow rates. Among this, it mainly analysed flow current conditions in the ship channel under normal navigation and flood discharging and sediment scouring.

(1) Velocity distribution under normal navigation condition. Different river channel discharge flow rates have a slight effect on the straight channel on the ship lock downstream when the channel is under navigating. And velocity is usually lower than 0.1 m/s. Meanwhile, there are refluxes of different intensities in the area that 400 m away from the entrance upstream. Nevertheless, there is no adverse effect on ship navigation when the flow is lower than 3160 m$^3$/s, reflux rate is smaller than 0.4 m/s. Whereas, there will be a negative effect on navigation when the flow is larger than 11600 m$^3$/s, and the reflux rate is higher than 0.4 m/s. Especially when the flow reaches to 13000 m$^3$/s, the maximum reflux rate can reach to 0.8 m/s.

(2) Horizontal velocity under normal navigation condition. Overall, horizontal velocity grows with the increase of the discharge flow rate in the river channel. There will be no adverse effects on ship navigation for the maximum horizontal velocity is lower than 0.3 m/s when the flow is lower than one year frequency flood flow. On the contrary, the maximum horizontal velocity will be higher than 0.3 m/s and there may be negative effects on ship navigation subsequently when the flow reaches one year frequency flood flow.

(3) Velocity changing regulation under sediment scouring. The average velocities at each sections range between 0.55 and 0.82 m/s when the flow rate is between 7000 and 13000 m$^3$/s. To the same section, average velocity decreases with the increase of discharge flow Q in the river channel. And under the same flow, different sections have different velocity distribution pattern.
5 ANALYSIS OF EROSION AND DEPOSITION CALCULATION IN RIVER CHANNEL AND APPROACH SHIP CHANNEL

We computed the erosion and deposition situation of the inner-river channel and approach ship channel over the five years after the power station start running. In the computing process, we used the water and sediment series that circularly combined from one-dimensional results by Hydrochina Kunming Engineering Corporation, whose data were from the water and sediment process between 1984 to 1988.

5.1 EROSION AND DEPOSITION DISTRIBUTION IN RIVER CHANNEL AND APPROACH SHIP CHANNEL

(1) Erosion and deposition distribution in river channel

Figure 5.1 illustrates erosion and deposition distribution of the inner-river channel and approach ship channel over the five years after the power station start to run, respectively. From this graph, it indicates that the river channel on the reservoir downstream has been eroded in different degree along the talweg since the power station starts running. And the erosion of the section decreases from the talweg to two banks. Thereinto, the maximum scour depth is at deep trough that 300 m away from the dam downstream. The scour rate gradually slows down when the maximum scour depth reaches about 1.0 m after the reservoir running for one year. During the period of time from the 1\textsuperscript{st} to the 9\textsuperscript{th} year, annual average scour depth at this point is 0.2 m, and by the ninth year, the erosion and deposition is basically balanced. Talweg of the river channel has been eroded about 1.4 to 2.0 m in average, while some parts have reached 3.0 m.

Local reflux and deposition form on the sluice downstream when the silt-releasing sluice is closed. And the depositional thickness was about 0.7 m.

(2) Erosion and deposition distribution in approach ship channel

It can be seen from Figure 5.1 that in the five years that the power station running, it appeared cumulative deposition trend on lower approach ship channel. And the degree decreased from the entrance to the ship lock. At the end of the 5\textsuperscript{th} year, the depositional thickness near the ship lock is about 1.0 m. The maximum is around 3.6 m. Thus, there will be navigation obstruction.
(a) Erosion and deposition distribution in the 1st running year of power station

(b) Erosion and deposition distribution in the 2nd running year of power station
(c) Erosion and deposition distribution in the 3\textsuperscript{rd} running year of power station

(d) Erosion and deposition distribution in the 4\textsuperscript{th} running year of power station
(e) Erosion and deposition distribution in the 5th running year of power station

Fig. 5.1  Erosion and deposition distribution in the different running period of the power station
5.2 ANALYSIS OF EROSION AND DEPOSITION PROCESS IN THE APPROACH SHIP CHANNEL

We selected 4 typical cross sections to analyse the changing process of the ship channel erosion and deposition. The locations of the sections are shown in Figure 5.2.

![Location of different typical cross sections](image)

**Fig. 5.2** Location of different typical cross sections

Figure 5.3 presents curves of erosion and deposition process at each typical cross section. The results show that after the station start running, deposition gradually showed up at all the cross sections, and the depositional thickness decreased from the entrance to the front of the dam. The first year of the station operation, average depositional thickness is about 0.35 m at section I#, and about 1.0 m at section IV#. And the deposition rate declined at the same time. The average thickness is around 0.15 m at section I# until the fifth year, and about 0.27 m at IV#.

During this five years, from ship lock to the approach ship channel, the average depositional thickness is around 1.1 m at cross section I#, and II# about 1.8 m, and sections from III to IV# is about 3.1–3.5 m.
Numerical Simulation of Sediment Movement in the Ship Channel of PAK BENG Hydropower Station Downstream

(a) Cross section I#

(b) Cross section II#
5.3 SUMMARY

(1) Variation of erosion and deposition in the river channel. There is erosion in different degrees along the talweg in all reservoir downstream river channels after the power station start running. And the erosion at cross sections...
decreased from talweg to the two banks. The average erosion of the talweg is about 1.4 to 2.0 m, and it can locally reach to 3.0 m.

(2) Variation of erosion and deposition in approach ship channel. There was a cumulative depositional trend in the lower approach ship channel after the reservoir running for five years. And the deposition rate gradually decreased during this time. From the entrance to the ship lock, the depositional thickness showed decline trend. At the end of the fifth year, thickness near the ship lock was approximately 1.0 m. The maximum depositional thickness was around 3.6 m that would have adverse effect on navigation.
6 OPTIMIZE DISPATCH OF APPROACH SHIP CHANNEL
SEDIMENT EROSION

From the analysis above, there was a cumulative depositional trend in the lower approach navigation channel during the five years of the reservoir running. Especially at the end of the fifth year, the maximum depositional thickness at the entrance was around 3.6 m that would have adverse effects on navigation. Thus reasonable dispatching method and proper engineering measures should be taken to guarantee the ship channel can meet the requirement of navigation. In this part we will study the effect of flood discharge and sediment erosion under different discharge flow rates. Aim to supply references for the optimization of dispatch.

6.1 SCHEME DESIGN OF APPROACH NAVIGATION CHANNEL
SEDIMENT EROSION

According to the design programme, the power station has sixteen tubular turbines, and the minimum flow rate that can support all the hydroelectric units running is 6424 m$^3$/s. The ship lock will stop working and the erosion sluice of the approach ship channel will be opened when the reservoir inflow rate is more than 3 years frequency flood flow 13200m$^3$/s..

Following we selected five typical flow rates to discharge flood and to scour sediment, then analysed the erosion effect of approach ship channel. Thereinto, the total discharge flow rate of from 2 to 5 units were larger than the minimum flow rate that can support all the generators running. While, the average flow rate of the first unit was smaller than the minimum flow rate that can support all the generators running. The detail information of flow rates and reservoir dispatching methods has been list in Table 6.1.
Table 6.1  Reservoir dispatch methods under different typical flow rates when approach ship channel is discharging flood and scouring sediment

<table>
<thead>
<tr>
<th>Serial number</th>
<th>Flow rate (m³/s)</th>
<th>Amount of sediment discharge (10³kg/s)</th>
<th>Downstream Water level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total discharge flow rate</td>
<td>Power Station 1-4</td>
<td>Power Station 5-8</td>
</tr>
<tr>
<td>1</td>
<td>3160</td>
<td>1606</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>7764</td>
<td>1606</td>
<td>1606</td>
</tr>
<tr>
<td>3</td>
<td>9000</td>
<td>1606</td>
<td>1606</td>
</tr>
<tr>
<td>4</td>
<td>11616</td>
<td>1606</td>
<td>1606</td>
</tr>
<tr>
<td>5</td>
<td>13200</td>
<td>1606</td>
<td>1606</td>
</tr>
</tbody>
</table>

In the calculation we took the landform above that formed after the station running for five years as the initial landform, and the time of erosion is 24 hours.

6.2 RESULTS ANALYSIS OF THE RIVER CHANNEL SEDIMENT EROSION UNDER DIFFERENT FLOW RATES

Figure 6.2 presents river bed elevation variation curves when the approach ship channel is discharging flood and scouring sediment for 24 hours under four typical flow rates. From the graphs, it is apparent that there is close relationship between erosion effect of the approach ship channel and total discharge flow rate when the sediment eroding rate is 1340 m³/s. More specifically, the erosion effect is not well when the total discharge flow rate was higher than 7000 m³/s. The maximum erosion depth in the straight ship channel near the dam was approximately between 0.1 and 0.3 m. While, it was about 0.7 m near the entrance. And the erosion effect is obvious when the discharge flow rate decreased to 3160 m³/s. To be more precise, the maximum erosion depth near the entrance could reach to 3.3 m, and average erosion depth in straight ship channel near entrance was about 3m. And the average erosion depth near the front of the dam could reach to 0.9~1.3 m more or less. After being eroded for 24 hours, the average thickness of the initial depositional sediment is smaller than 0.5 m in the entire lower approach ship channel. And the maximum elevation of the ship channel is lower than 303 m.
Numerical Simulation of Sediment Movement in the Ship Channel of PAK BENG Hydropower Station Downstream

(a) Cross section I

(b) Cross section II
Fig. 6.1 comparing curves of elevation variation of the ship channel when the approach ship channel is discharging flood and scouring sediment under four typical flow rates.
6.3 OPTIMIZATIONAL DISPATCHING PROGRAMME OF THE APPROACH SHIP CHANNEL SEDIMENT EROSION

According to the sediment erosion calculation, the erosion effect of the lower approach ship channel was not well when the flow rate was between 7000 and 13200 m$^3$/s. And the maximum erosion depth was smaller than 0.7 m, still the maximum elevation of the lower approach ship channel was 305.3 m. By contrast, the erosion effect was relative obvious when the flow rate was 3160 m$^3$/s. After scouring for 24 hours, maximum erosion depth was close to 3.3 m. And the thickness of the initial depositional sediment was smaller than 0.5m, and maximum elevation of the ship channel is lower than 303 m. Hence, we suggests that scouring sediment in the approach ship channel should under small flow rate discharge condition.

6.4 SUMMARY

(1) Erosion effect of the approach ship channel. If sediment erosion flow rates are the same, the erosion depth decreased with the discharge flow increase. While, the erosion depth rose gradually from the ship lock to the entrance. If take the landform in Chapter 5, which the maximum deposition elevation is 3.6 m, as the initial landform, and erode the ship channel for 24 hours under different discharge flow rates, then, maximum erosion depth was about 0.5 m and maximum elevation of the approach ship channel after been eroded was about 305.6 m when the flow rate was between 7000 and 13200 m$^3$/s; While, the erosion effect was better when the flow rate was 3160 m$^3$/s, and the maximum erosion depth was close to 3.3 m; Whereas, the maximum elevation dropped to 303 m.

(2) Optimizational dispatch. We suggests that scouring sediment in the approach ship channel should under small flow rate condition.
7 CONCLUSIONS AND SUGGESTIONS

7.1 CONCLUSIONS

(1) Velocity distribution when approach ship channel was navigating normally. The velocity in the straight ship channel that near the ship lock was usually smaller than 0.1 m/s when the ship channel was under navigation. And there were refluxes of different intensities in the region from 400 m away from the entrance upstream to the entrance. The maximum horizontal velocity was smaller than 0.3 m/s when the flow rate was lower than that of 2 years frequency flood; Therefore there was no adverse effects on navigation. Otherwise, the maximum horizontal velocity was more than 0.3 m/s, which might have negative effects on navigation.

(2) Erosion and deposition variation in the river channel. The river channel on the lower reach of reservoir has been eroded in different degrees along the talweg since the power station start running. The cross section erosion decreased from the talweg to two banks. The average erosion depth of the talweg was about 1.4 to 2.0 m. It could reach to 3.0m in local part.

(3) Erosion and deposition variation in lower approach ship channel. During the five running years, if the ship channel was not scour ed when flow rate was lower than 13200 m$^3$/s, then cumulative depositional trend would show up in the lower approach ship channel, with decreasing deposition rate. Thickness of deposition had a descending trend from the entrance to the ship lock. At the end of the fifth year, thickness near the ship lock was about 1.0 m. The maximum thickness near the entrance was about 3.6 m, that would have adverse effects on navigation.

(4) Flood discharge and sediment erosion in lower approach ship channel. The larger the discharge flow rate in the river channel was, the less apparent the erosion effect of the approach ship channel was. The erosion effect was not well when the flow rate exceeded 7000 m$^3$/s. And the maximum erosion depth was smaller than 0.5 m/s; While, the erosion effect was apparent when flow rate was 3160 m$^3$/s, for the depositional sediment formed during this five years nearly all be cleaned up in 24 hours.

7.2 SUGGESTIONS

(1) The local horizontal velocity at the entrance of the approach ship channel will
over 0.4 m/s when the discharge flow rate exceed 2 years frequency flood. Thus, it will have adverse effects on navigation. The horizontal velocity at the entrance is larger, for the intersection angle between velocity in approach ship channel and that in main river channel is not smooth enough and the reflux that caused by concentration. This can be solved through adjusting the angle of guide walls in approach ship channel, smoothing the intersection angle, or setting up incomplete permeable wall, and so on.

(2) For there is adverse effect on navigation at the depositional place at the entrance after the reservoir start running, we suggest to take the following two measures to solve this problem.

Flood discharging and sediment scouring. Because of the backwater effect that impact the discharge of approach ship channel, which caused by the larger discharge flow rate in river channel, we suggest scour the channel when the discharge flow rate is smaller. And we can choose to scouring at night and can repeat it for several times considering about the electricity loss caused by discharge and the stop of navigation during the sediment scouring period.

Mechanical dredging. Dredge the entrance of approach ship channel regularly to reduce deposition in the channel.