Pak Lay Hydropower Project
Hydraulic Model Test Report

1. Overview

Pak Lay Hydropower Project is located at the middle reaches on the Mekong River in Lao P.D.R. It is the 4^{th} cascade among the 11 in the Mekong River mainstream planning (from upstream to downstream). Its upstream is Xayaburi cascade and downstream is Sanakham cascade.

The project is mainly developed for power generation, concurrently with such functionsas navigation, recreation, tourism. The power plant has a normal storage level of 240.00m, a corresponding reservoir storage of 890 million m^3, a total installed capacity of 770MW (14×55MW). It has a multi-year average power generation of 4.1248×10^3GW·h and 5357 annual utilization hours.

In this project, the water retaining structure is a normal concrete gravity dam and the powerhouse is a run-of-river type. The max.dam height is 51.00m. The flood design standard in normal service is a 2,000-year flood and the flood design standard for emergency service is a 10,000-year flood. The flood peak volume of a 2,000-year flood (P=0.05%) is 34,700m^3/s and the flood peak volume of a 10,000-year flood (P=0.01%) is 38,800m^3/s.

2. Hydraulic Model Test of Operation Period

2.1Model Scale

In accordance with SL155-2012, *Specification for normal hydraulic model test*, an undistorted model on a scale of 1:100 and designed with a gravity similarity rule is adopted. The scales of the physical parameters are shown in Table 2.1-1.

Table 2.1-1Scales of Various Model Physical Parameters

Description of Physical Parameter	Geometric Scale	Flow Velocity Scale	Flow Scale	Pressure Scale	Time Scale	Roughness Scale

Scale relation	λ_L	$\lambda_V = \lambda_L^{0.5}$	$\lambda_Q = \lambda_L^{2.5}$	$\lambda_{\frac{P}{\gamma}} = \lambda_L$	$\lambda_t = \lambda_L^{0.5}$	$\lambda_n = \lambda_L^{1/6}$
Scale	100	10	100000	100	10	2.154

In order to ensure the similarity of the water flow in the river reach modelled, the model simulates the river reach between the location 1200m upstream of the prototype dam axis and the location 1800m downstream of the prototype dam axis. The model has a total length of 30m and a total width of 11m, as detailed in Fig. 6.2.

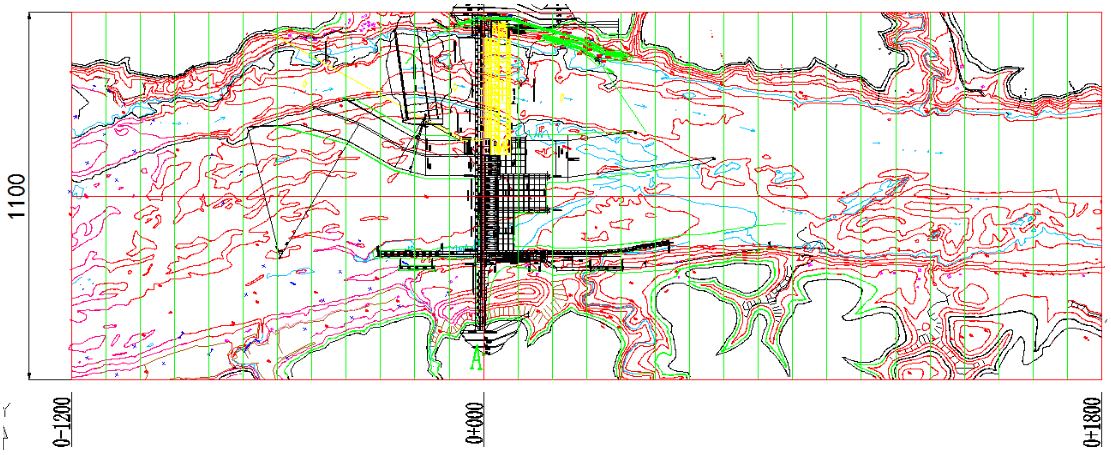


Fig.2.1-1Simulation Scope of Hydraulic Model of Pak Lay HPP

2.2 Model and Scope

Model tests under 11 operating conditions with a discharge falling between 1,870m^3/s~38,000m^3/shave been taken, including the operating condition when the navigation discharge reaches the max. 16,700m^3/s and the powerhouse stops running, the operating condition when two low-level sand flushing outlets and 14 surface bays are fully opened with a discharge of 38,000m^3/s and the operating condition when 1 to 3 gates are considered not to be opened with a discharge of 38,000m^3/s.

2.3 Test Result

2.3.1Discharge capacity

Through the comparison between the test discharge capacityand design discharge capacity, the results indicate that: when the upstream water level is less than 238.00m, the testdischarge capacity is less than the design discharge capacity; when the upstream water level is larger than 238.00m, the test discharge capacity is larger than the design discharge capacity, and the difference increases with the increase of the upstream water level, as shown in Table 2.3-1. Under the design discharge (34700m3/s) and the check discharge (38800 m^3/s), the test water level is respectively 0.05m and 0.18m lower than the design water level, where the discharge capacity of the project satisfies the requirements.

Table 2.3-1Comparison Data of Design and Test Discharge Capacities of All Flood Release Bays and Outlets Fully Opened

Upstream water level m	Designflow m^3/s	Discharge Calculated by Test Fitting Formula m^3/s	Discharge Difference $(Q_{Test}-Q_{Design})$ m^3/s	$(Q_{Test}-Q_{Design})/Q_{Design}\times100\%$
230	14337	14106	-231	-1.61%
231	16198	15986	-212	-1.31%
232	18188	17970	-218	-1.20%
233	20277	20060	-217	-1.07%
234	22485	22254	-231	-1.03%
235	24759	24554	-205	-0.83%
236	27130	26959	-171	-0.63%
237	29561	29470	-91	-0.31%
238	32086	32085	-1	0.00%
239	34658	34805	147	0.43%
240	37335	37631	296	0.79%
240.59	38800	39348	548	1.41%
241	40089	40562	473	1.18%

In order to understand the flood release characteristics passing through the dam in the actual operation process, when some bays fail to participate the operation due to abrupt fault and an extreme situation such as the check flood occurs. It is arranged in the test to verify the discharge of the check flood and the results indicate that: when 3 deep surface bays or 5 shallow surface bays fail to participate the flood release, the upstream water level will exceed 245m and overflow occur on the non-spillway dam section. Inferred with said results, except for the combination with 3 deep surface bays failing to participate the flood release, the reservoir water level under the combinations with any three of the remaining surface bays unable to participate the flood release are all below 245m, as detailed in Table 2.1-2.

Table 2.1-2 Test Data under Check Flow When Some Surface Bays Unable to Participate Flood Release

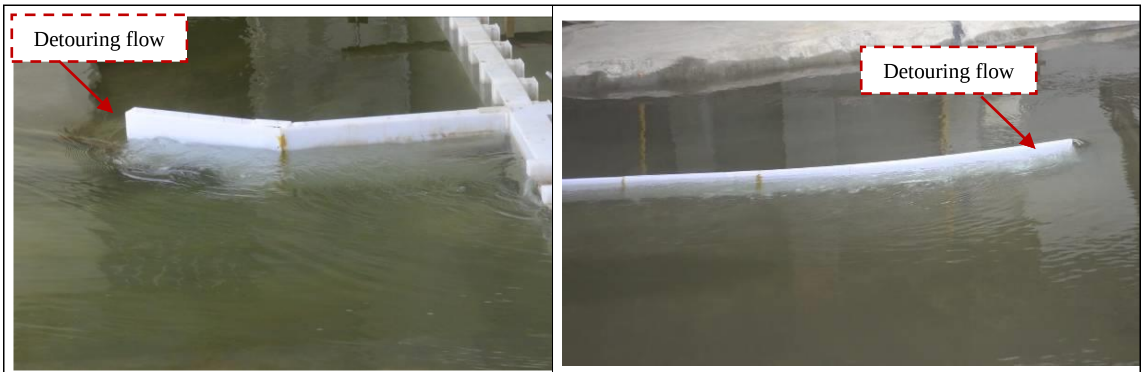
No.	Discharge m^3/s	Upstream water level m	Downstream water level m	Gate Opening Mode	Remarks
1	38890	241.37	236.47	②low-level outlets Fully opened +10 shallow surface bays fully opened +3 deep surface baysfully opened	1 shallow surface bay unable to participate the service
2		242.17	236.47	②low-level outlets fully opened +9 shallow surface bays fully opened +3 deep surface baysfully opened	2 shallow surface bays unable to participate the service
3		243.04	236.47	②low-level outlets Fully opened +8 shallow surface bays fully opened +3 deep surface baysfully opened	3 shallow surface bays unable to participate the service
4		243.90	236.47	②low-level outlets fully opened +9 shallow surface bays fully opened +2 deep surface baysfully opened	2 shallow surface bays +1 deep surface bays unable to participate the service
5		244.80	236.44	②low-level outlets fully opened +10 shallow surface bays fully opened +1 deep surface bays fully opened	1 shallow surface bay +2 deep surface bays unable to participate the service
6		245.19	236.42	②low-level outlets fully opened +10 shallow surface bays	3 deep bays unable to participate the

				fully opened	service

Note: The upstream water levels in No. 4 and 5 of the table are obtained by linear interpolation based on the available discharge capacity data.

2.3.2 Flow regime

The inflow from the upstream reservoir is smooth. In case that the flood release dam section participates the flood release, when the water flow passes by the heads of the upstream approach channel wall and the powerhouse-dam guiding wall, there exists detouring flow, which is weak with the gates partially opened and relatively obvious with the gates fully opened. However the detouring flow has limited impact on the structure and the discharge capacity. Under the plant power generation service condition, the inflow upstream of the headrace channel is smooth and the water surface steady. There exists weak recirculation zone in a small area on the left side of the powerhouse-dam guiding wall. Under various service conditions with surface bays partially opened, there are intermittent hopper vortexes on the water surface close to Chainage Dam 0+0.000m in the bay chamber, occasionally with air. The measured max.vortex has an outer diameter of about 4m, and vortex flow regimes of the bay chambers are shown in Fig. 2.3.2-1.



(a) Flow regime close to head of upstream Powerhouse-Dam guiding wall

(b) Flow regime close to head of upstream approach channel wall

Fig. 2.3.2-1Operating condition 10 (Q= 38800m^3/s, 2 low-level outlets and 14 surface bays fully opened) upstream flow regime

With all the gates fully opened, as affected by the detouring flow at the guiding wall upstream of the bay, the water surface in the two lateral bays is not so symmetric;

the inflow of the remaining bays is relatively symmetric and smooth.Submerged underflow occurs at the low-level flushing bays. Under the test service condition, the flow regime downstream of the bays at the stilling basin area is relatively symmetric and steady,and under most service conditions the water flow in the basin is steady and of submerged underflow. See Fig. 2.3.2-2 for the flow regime in the main energy dissipation area.

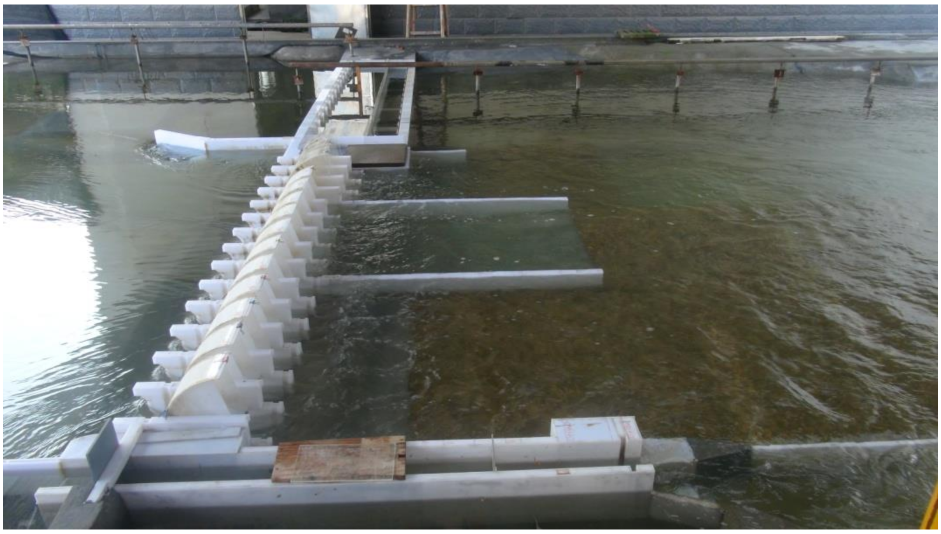


Fig. 2.3.2-2Operating condition9 (Q= 34895m^3/s, 2 low-level outlets and 14 surface bays fully opened) main energy dissipation area

Under the power generation service condition of the plant, the tailrace channel of the plant has smooth flow, where there occurs no unfavorable flow regime; the downstream main flow is in the middle of the river on the left, with smooth flow regime.When the plant does not generate power, the main flow expands longitudinally downstream of the basin, proceeding in the middle of the river lightly on the left side. See Fig. 2.3.2-3 for downstream flow regime.

Fig. 2.3.2-3 Operating condition 9 (Q= 34895m^3/s, 2 low-level outlets and 14 surface bays fully opened) downstream flow regime

2.3.3 Water surface profile

When all the sluice bays operate with the gate fully opened, the water surface profile on the right side of the upstream approach channel wall is obviously higher than that on the left side, and the water level difference between the two sides increases with the increase below the bay opening.The measured max.water level differences are 2.25m and 3.15m respectively.

Under the gate fully opened service condition, the water surface profile and the fluctuation amplitude in the lock chamber of the surface bays increase with the increase of the discharge. Affected by the detouring flow at the head of the upstream powerhouse-dam guiding wall, the water surface profile upstream Chainage Dam 0+10.00m in the lock chamber from ① surface bay to ④ surface bay appears as low in the left and high in the right; there is certain abrupt expansion space on the right side downstream of the gate pier of ⑭ surface bay (Chainage Dam 0+42.00m), thus the water surface profile downstream of Chainage Dam 0+30.00m is slightly higher than that of other bays. The water surface fluctuation at ⑨ surface bay to ⑭ surface bay is the minimum; and that at ①surface bay to ⑧ surface bay a little bit higher, with the measured max. fluctuation amplitude as 4.3m.

The longitudinal water surface profile and the fluctuation within the main energy dissipation area are related to the scale of the discharge and the operation mode at the dam section where the discharge is located. Generally, the fluctuation is small. In case of all spillway sluice bay gates are in service at full opening, the fluctuation of the water surface is relatively small, with the max. measured fluctuation as 2.2m. In case of a discharge of 16700m^3/s and below, most of the measured max.water surface fluctuation within the main energy dissipation area under various service conditions is above 3.5m, with the max. value as 4.8m.

Under the power generation service condition, the water surface fluctuation downstream of the plant tailrace water is not large, varying between 0.4m and 0.8m.

Under the service condition with a discharge of 16700m3/s or below, the water surface profile on the right side of the downstream approach channel wall is higher than that on the left side. The max.water level difference between the two sides is 1.67m.

The downstream water surface profile and water surface fluctuation decrease with the decrease of the discharge, where the water surface fluctuation is closely related to the gate operation mode. When all the sluice bays discharge in a relatively uniform way, the downstream bank shore water surface fluctuation is small. In case of a discharge of a 10000-year-return flood (Q=39040m3/s), the max. water surface fluctuation at the downstream bank shore is 1.1m; and in case of relatively concentrated discharge of some spillway bays, the max. water surface fluctuation at the downstream bank shore is 2.5m.

2.3.4 Flow Velocity

In the upstream reservoir area, the measured bottom flow velocity of the riverbed close to the dam is relatively large and increases longitudinally. All the max.bottom flow velocities within 100m to the dam of all the spillway bays under fully opened service condition are larger than 4m/s. The max.measured bottom flow velocity in case of a 100-year-return flood (Q=27366m^3/s) is 4.57m/s and the max. measured bottom flow velocity in case of a 10000-year-return flood (Q=38800m^3/s) is 5.55m/s.

In case of operation service condition with a discharge of 16700m^3/s, the bottom flow velocity of the water release structure is relatively large, and the max. measured bottom flow velocities both in the deep channel area and the shallow surface bays area are larger than 15m/s. The max.value is 16.65m/s. In case of a 10,000-year flood ($Q=38800m^3/s$) when all the flood release bays and outlets are fully opened, the max. measured bottom flow velocity is 11.16m/s. When all the flood release bays and outlets are fully opened (with a discharge of a 100-year flood and above), the max.bottom flow velocities at the ends of the aprons and the end of the stilling basin are all above 4.5m/s, most of about 6m/s.

The flow velocity at the river bed downstream of the stilling basin decreases gradually; the max. measured bottom flow velocity with a 10,000-year-return flood is 7.42m/s.The flow velocity on two downstream banks is small. The measured flow velocity is generally below 3m/s and the max. measured flow velocity is 3.06m/s.

2.3.5 Downstream Scouring

When the discharge reaches $16700m^3/s$ and the low-level outlets participate the service, the aprons are subject to scouring. Scouring is even more serious under the operating condition with big difference between upstream and downstream water level. The measured lowest point in the scour pit has an elevation of 192.21m (with a scouring depth of 11.79m). See Fig. 2.3.5-1and 2.3.5-2 for details.

Fig. 2.3.5-1Downstream in the scour pit ($Q=16700m^3/s$, ①②③surface bays all with an opening of 8.33m+④⑤⑥⑦⑧surface bays all with an opening of 11m+⑨⑩⑪⑫⑬⑭surface bays all with an opening of 2m)

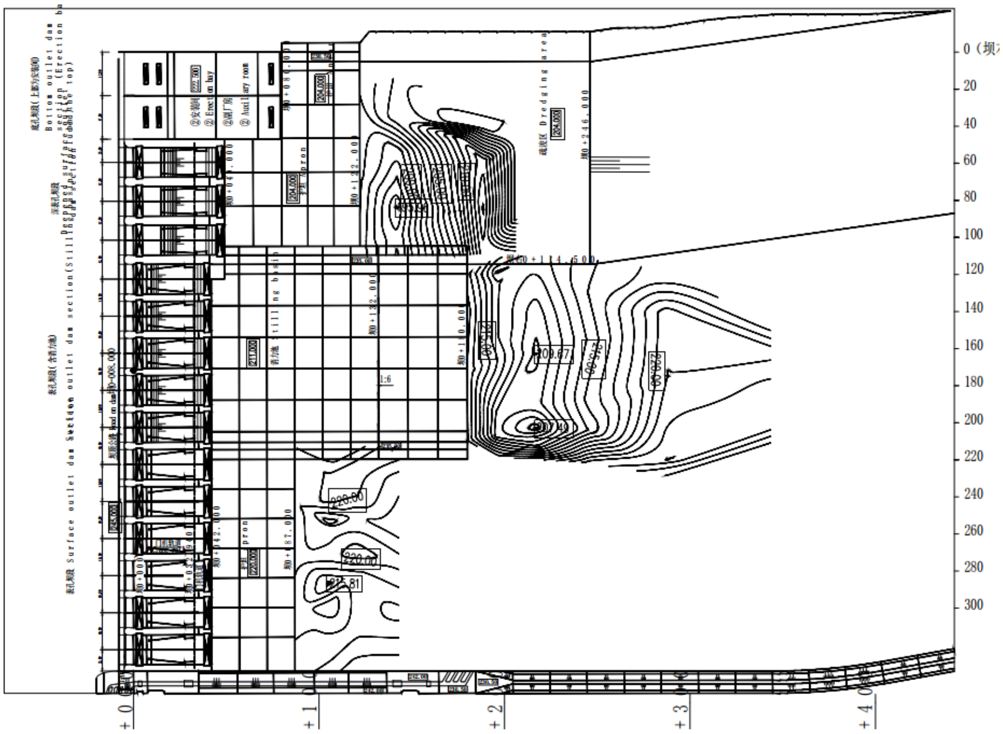


Fig. 2.3.5-2 Cross section of downstream in the scour pit (Q=16700m^3/s, ①②③surface bays all with an opening of8.33m+④⑤⑥⑦⑧surface bays all with an opening of11m+⑨⑩⑪⑫⑬⑭surface bays all with an opening of2m)

Scouring is less severe under the service condition with all the flood release bay gates fully opened. Minor scouring occurs in the downstream in case of a discharge of 100-year-return flood. The max. depth of the downstream scour pits in case of discharges of 2000-year-return and 10000-year-return floods is about 4.5m, the scour pit downstream of the shallow surface bays is a little bit deeper. The lowest point in the scour pit downstream of the shallow surface bays in case of a discharge of 10000-year-return flood has an elevation of 214.01m (with a scouring depth of 4.99m). See Fig. 2.3.5-3 and 2.3.5.2-4 for details.

Fig. 2.3.5-3Downstream in the scour pit (service condition 10, Q=38000m^3/s, 2 low-level outlets and 14 surface bays fully opened)

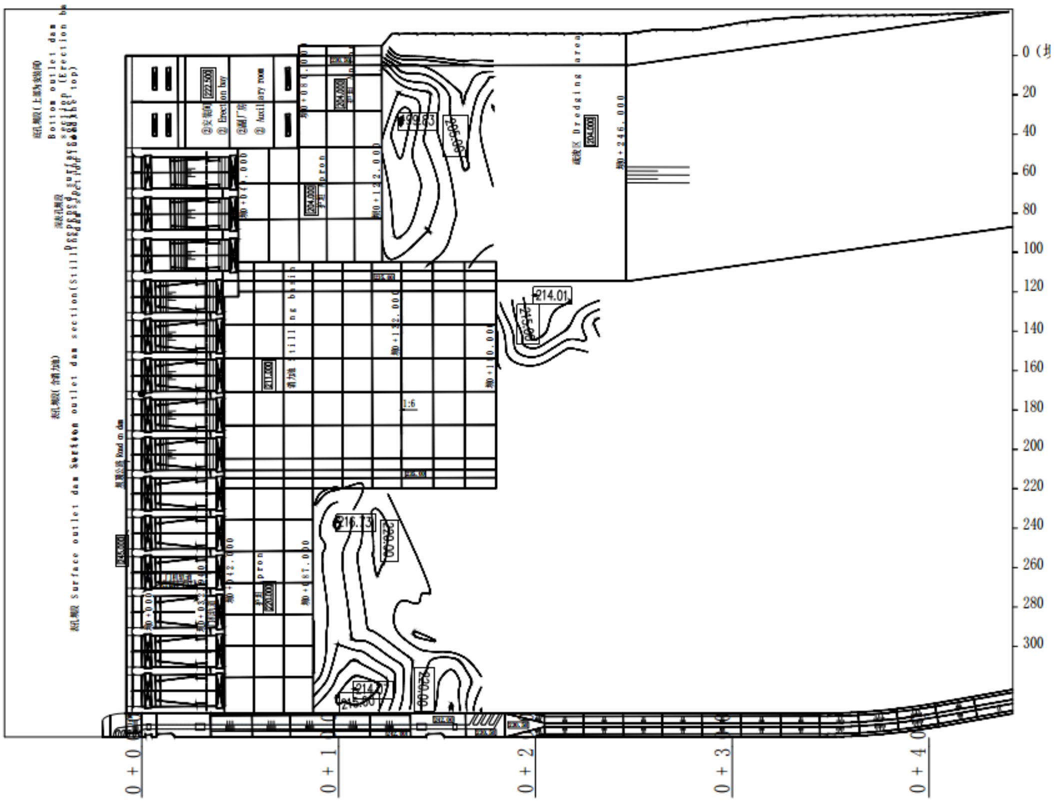


Fig. 2.3.5-4Downstream in the scour pit (service condition10, Q=38000m^3/s,2 low-level outlets and 14 surface bays fully opened)

2.3.6 Navigation conditions of upstream and downstream shiplock approach channels

Under various service conditions, the water surface within the upstream approach channel wallis flat and steady, where there exists weak recirculation. When the discharge is 16700m^3/s, see Fig. 2.3.6-1 for the flow fields of the upstream approach channel mouth area and the connection section.

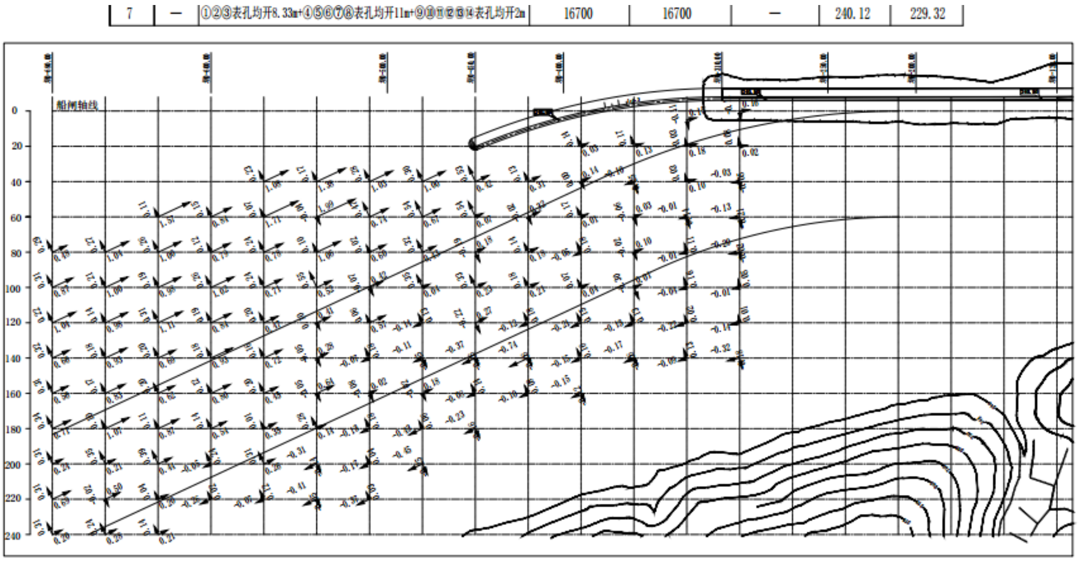


Fig. 2.3.6-1 Flow Fields of the Upstream Approach Channel Mouth Area and the Connection Section

The flow regime and the flow velocity in the downstream navigation channel mouth area is affected by both of the total discharge and the gate opening mode. In case of a discharge of Q_{total}=6101m^3/s and below, no matter whether the discharge passes through the units or the flood release dam section, theflow velocity index in the downstream navigation channel mouth area can satisfy the navigation requirements. In case of a discharge of Q_{total}=8500m^3/s~16700m^3/s, there is non-compliance of the recirculation flow velocity at some measuring points by the bankshore 70m downstream of the head of the approach channel wall, and there is non-compliance of the transversal flow velocity at some measuring points close to the river center 130m downstream of the head of the approach channel wall. Under service condition 6’ (Q_{total}=16700m^3/s, with low-level outlets participating service), there is non-compliance of the longitudinal flow velocity at some measuring points close to the river center 220m downstream of the head of the approach channel wall. It is considered that when the fleet passes in and out the downstream mouth area, the fleet proceeds with a Min. navigation channel diameter of 880m between 160m~330m downstream the head of the approach channel wall, which can basically satisfies the navigation requirements. See Fig. 2.3.6-2 for the flow regime at the downstream navigation channel mouth area.

工 况	频率 %	运行方式	总下泄流量 m³/s	泄洪闸流量 m³/s	电厂流量 m³/s	上游水位 m	下游水位 m
6	—	机组满发+①②③表孔开5m+④⑥⑧表孔开7m+⑤⑦表孔开6m+⑨⑩⑪⑫⑬⑭表孔开1m	16700	10599	6101	240.14	229.35

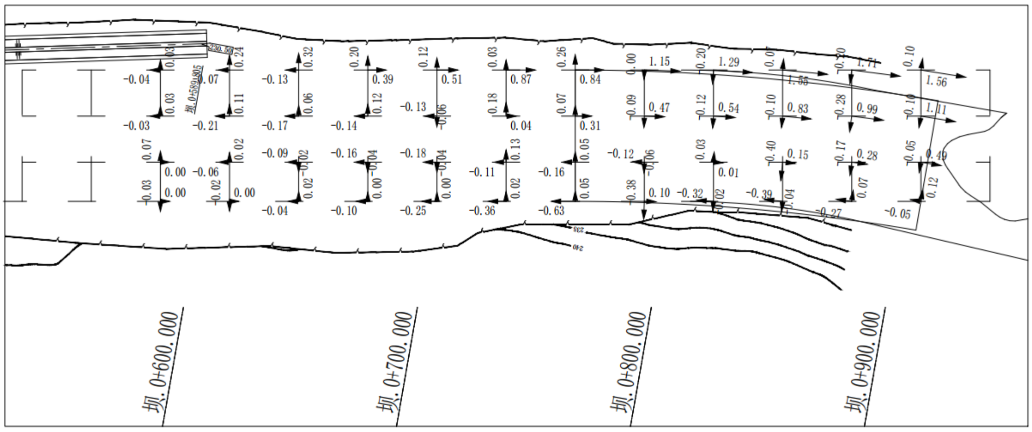


Fig. 2.3.6-2 Flow Regime at the Downstream Navigation Channel Mouth Area

3. Construction Period Model Test

3.1 Model Scale and Modelled Scope

An undistorted model on a scale of 1:100 and designed with a gravity similarity rule is adopted. The scales of the physical parameters are shown in Table 2.1-1. The model scale is the same with that in the hydraulic test of the operation period.

The model simulates the river reach between the location 1400m upstream of the prototype dam axis and the location 1800m downstream of the prototype dam axis . The model has a total length of 30m and a total width of 11m, as detailed in Fig. 3.1-1.

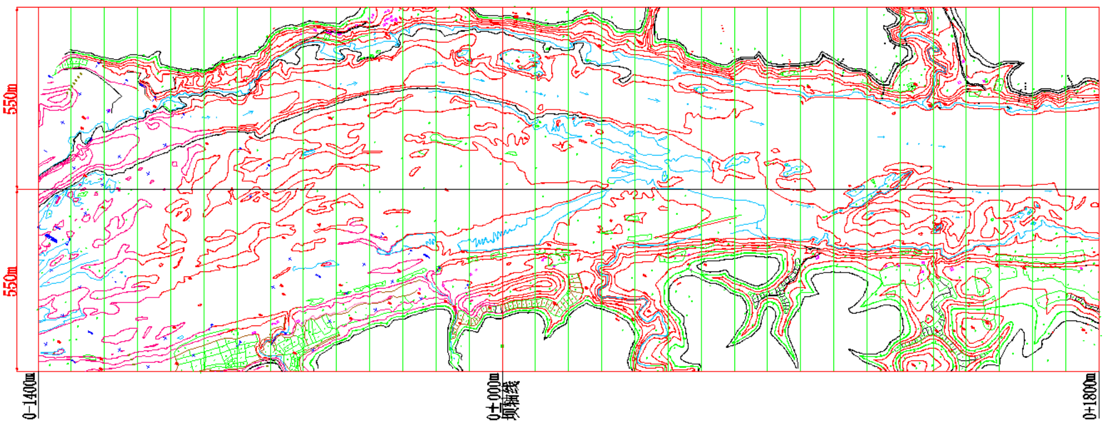


Fig. 3.1-1 **Simulation Scope of Construction Diversion Model of Pak Lay HPP**

3.2 Phase I Construction Diversion Model Test Result

3.2.1 Discharge capacity

The discharge capacity of the narrowing river course on the left is slightly small and basically meets the design requirement. In discharging a 20-year flood ($Q=23000m^3/s$), the water levels in front of the upstream and downstream earth-rock cofferdams are 234.025m and 232.265m respectively, 1.475m and 1.735m lower than the crest elevation of the two cofferdams.

3.2.2 Flow regime

Under various flow conditions, the water flow in front of the upstream and downstream earth-rock cofferdam is smooth and stable, with no unfavorable flow regime. The detouring flow occurs to the narrowing river bed section on the left bank and the head of the longitudinal concrete cofferdam; recirculation zone is detected on the side downstream of the detouring flow next to the longitudinal concrete cofferdam. The scopes and intensities of the detouring flow and recirculation zone increases along with the rise in the discharge. The main flow in the narrowing river course moves slightly to the left in the middle and diffuses to the right side gradually along the way. For the flow regime of phase I construction diversion under the operating condition of 20-year flood, see Fig. 3.2.1-1.

Fig. 3.2.2-1Flow Regime of Phase I Construction Diversion (20-year flood, Q=

23,174m^3/s)

3.2.3Water surface profile

Under various operating conditions, the water surface profile and thewater fluctuation increases with the rise in discharge.

The water surface near each cofferdam changes smoothly with no big fluctuation. The average water surface fluctuation is below 1.4m. The max.measured water surface fluctuation in front of the upstream and downstream transversal cofferdams and the longitudinal cofferdam is 0.61m, 0.55m and 1.37m respectively.

3.2.4Flow velocity

The flow velocity near the upstream and downstream earth-rock cofferdam is relatively small. The max.measured flow velocity near the upstream cofferdam is 0.69m/s and that near the upstream cofferdam is 0.20m/s.

The flow velocity in front of the head of the upstream longitudinal concrete cofferdam is relatively big and reaches 5.97m/s in case of a 20-year flood discharge. As the left side of the longitudinal cofferdam and the upstream of the transversal cofferdam is a recirculation zone, with a max. flow velocity of 4m/s; the flow velocity in the rest areas is all below 2.2m/s.

The bottom flow velocity of the flow-passing river bed (the river bed on the left side of the longitudinal concrete cofferdam) is relatively high, especially the flow velocity of the convex body in the middle of the river. The average max.measured flow velocity under various operating conditions is around 7m/s, with the biggest one standing at 7.21m/s. For the flow velocity and water level of the 20-year flood at phase I cofferdam, see Fig. 3.2.4-1.

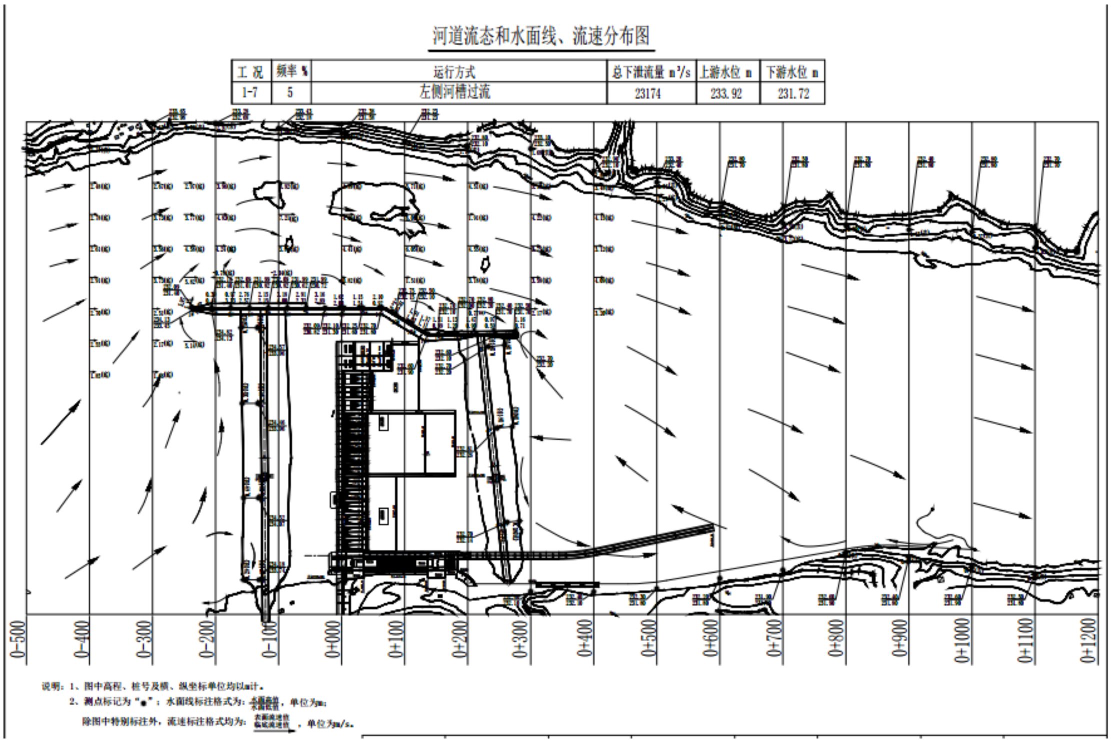


Fig. 3.2.4-1Flow Velocity and Water Level of Phase I Construction Diversion (20-year flood, Q= 23174m^3/s)

3.2.5 Navigation condition

When the fleet goes through the narrowing section of the river course (section under construction), it moves forward along the river course near the left bank (with a turning radius of around 1,050m). The condition for ship navigation is a flow velocity at the water surface below 2.50m/s. Then, when the discharge is 4000m^3/s or below, the ship can pass through the narrowing section of the river course; when the discharge is 4000m^3/s～6000 m^3/s, the flow is mostly below 3m/s and it can meet the navigation requirement with the aids to navigation. When the convex body of the narrowing river course on the left side is excavated to an elevation of 211m and the discharge reaches 8961m^3/s, the flow velocity is basically below 3.5m/s. In this case, with the aids to navigation, it meets the navigation requirement. When the discharge is 8961m^3/s, see Fig. 3.2.5-1 for the flow velocity of the river course on the left bank.

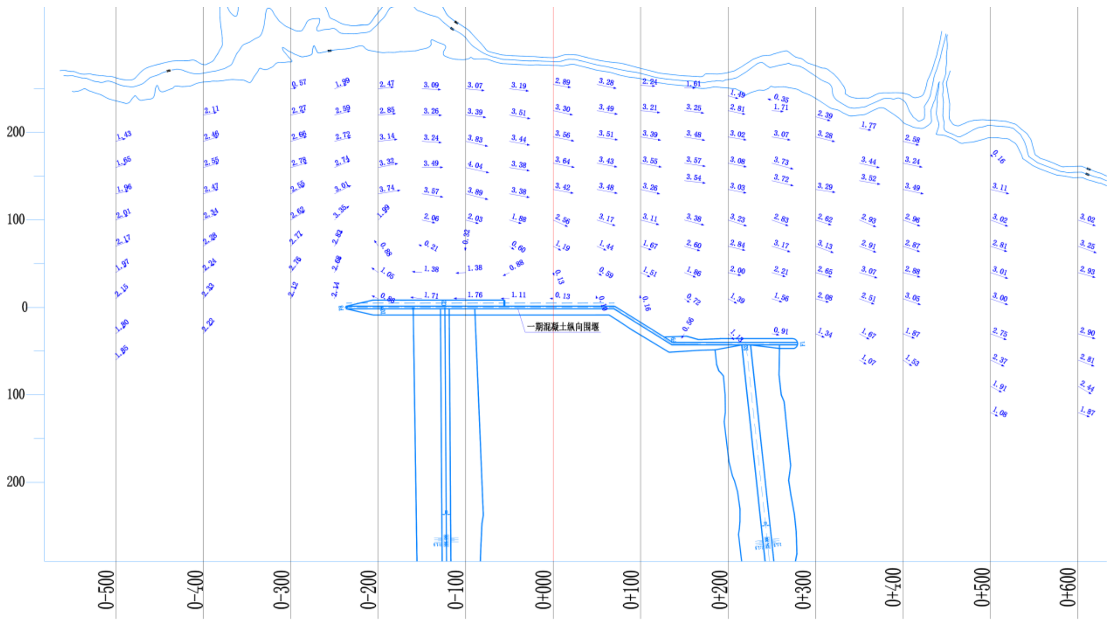


Fig. 3.2.5-1Q=8961m^3/s, river course flow field (combined flow velocity)

3.3Phase II Construction Diversion Model Test Result

3.3.1Discharge capacity

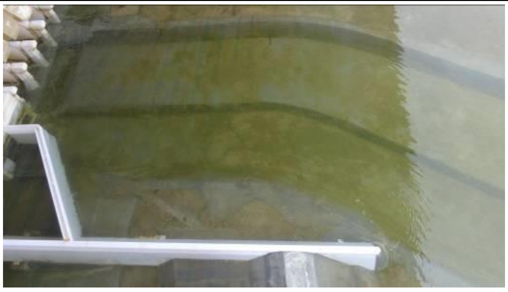
The discharge capacity with 14 surface bays (including 11 shallow surface bays and 3 deep surface bays) fully opened can meet the design requirement on Phase II construction diversion. In discharging a 20-year flood (Q=23000m^3/s), the water levels in front of the upstream and downstream earth-rock cofferdams are 234.551m and 231.899m respectively, 3.949m and 2.101m lower than the crest elevation of the two cofferdams.

3.3.2Flow regime

The inflow from the upstream reservoir is smooth.Weak detouring flow exists at the head of the upstream Powerhouse-Dam guiding wall and the head of the upstream Phase II longitudinal concrete cofferdam. The flow regime behind the gate at the flood sluicing dam section is symmetric and stable and connects smoothly with the downstream water flow. The water surface near each cofferdam is smooth and stable and no unfavorable flow regime has been detected. When the phase I upstream earth-rock cofferdam is demolished to an elevation of 218m and the discharge reaches 4,300m^3/s, the water surface above the residual cofferdam will have fierce fluctuation.

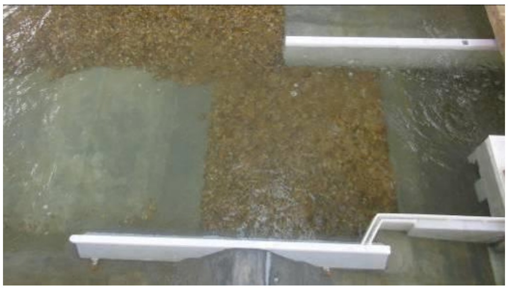
No other unfavorable flow regime will occur. For the flow regime of phase II diversion model, see Fig. 3.3.2-1.

Upstream earth-rock cofferdam



Upstream longitudinal cofferdam

Downstream earth-rock cofferdam



Downstream longitudinal cofferdam

Fig. 3.3.2-1Flow Regime of Phase II Diversion Model (P=2%, Q= 25704m^3/s)

3.3.3 Water surface profile

The water surface profile and water fluctuation are affected jointly by the discharge and gate opening pattern. The water surface fluctuation under various working condition is not very big, with the max. fluctuation of 2.5m. The max.measured water surface fluctuation in front of the upstream and downstream earth-rock cofferdams is 0.67m and 0.73m respectively.

The water surface fluctuation near the upstream and downstream longitudinal concrete cofferdam with the gates fully opened is both below 1.2m. When the gate is partially opened, the water surface fluctuation near the downstream longitudinal concrete cofferdam is considerably higher than that in other operating conditions (generally above 1.5m) and the max. measured water surface fluctuation is 2.5m.

The water surface fluctuation next to the bank at the downstream is below 1.5m.

3.3.4 Flow velocity

Except for the water releasing structures where the flow velocity is relatively high, the flow velocity of the rest parts is below 4m/s.

The max.measured flow velocity near the upstream cofferdam is 0.96m/s and that near the downstream cofferdam is 0.92m/s.

The max.measured flow velocity near the upstream longitudinal concrete cofferdam is 3.35m/s. Except for the pier head and the measuring point 9# where the flow velocity is slightly high, the flow velocity at the rest of the measuring points is below 2m/s. The max.measured flow velocity near the downstream longitudinal concrete cofferdam is 3.69m/s. Except for the case with partially opened gate when the flow velocity is relatively high, the flow velocity in other operating conditions is below 2.2m/s.

In the operating condition with all the 14 surface bays fully opened, the flow velocity at the end of aprons at each flood releasing section basically increases with the rise in discharge and the bottom flow velocity is above 4.5m/s, with the max. one being 6.66m/s; in releasing a 20-year flood, the bottom flow velocity at the end of each apron is around 6m/s.

The flow velocity of the bank side at the downstream is relatively small, with the max. measured flow velocity being 1.64m/s.

For the flow velocity and water level at phase I cofferdam in a 20-year flood, see Fig. 3.3.4-1.

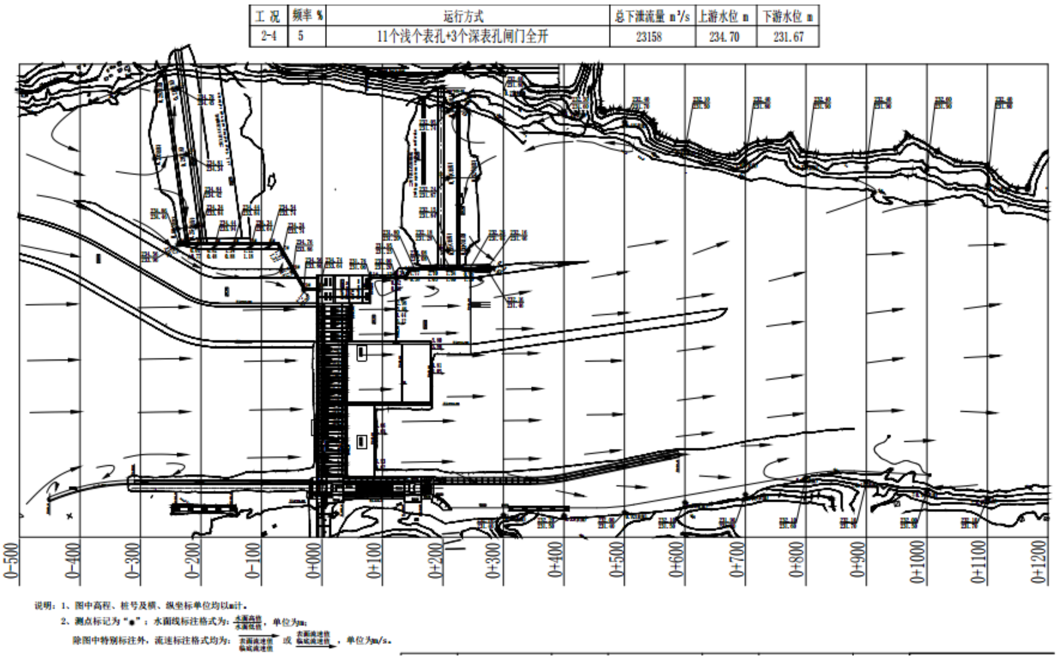


Fig. 3.3.4-1 Phase II Construction Diversion Flow Velocity and Water Level (20-year flood, Q= 23158m^3/s)

3.3.5Downstream scouring

The scouring to the area behind the aprons of the water releasing structures will be affected more by the gate scheduling mode. In case of releasing a 50-year flood or below with the gates of the 14 surface bays fully opened, scouring is not detected in the area behind the water releasing structure. However, obvious scouring is detected in the case when the upstream water level reaches 236.5m and the gates are partially opened. The scouring pit behind the deep trench area stands at an elevation of 194.14m at its lowest point (the scouring depth is 9.86m). The scouring pit behind the stilling basin stands at an elevation of 213.04m at its lowest point (the scouring depth is 5.96m). The scouring pit behind the surface bays ⑨ to ⑭ stands at an elevation of 215.81m at its lowest point (the scouring depth is 3.19m). See Fig. 3.5.5-1 for the downstream scouring pit of phase II diversion.

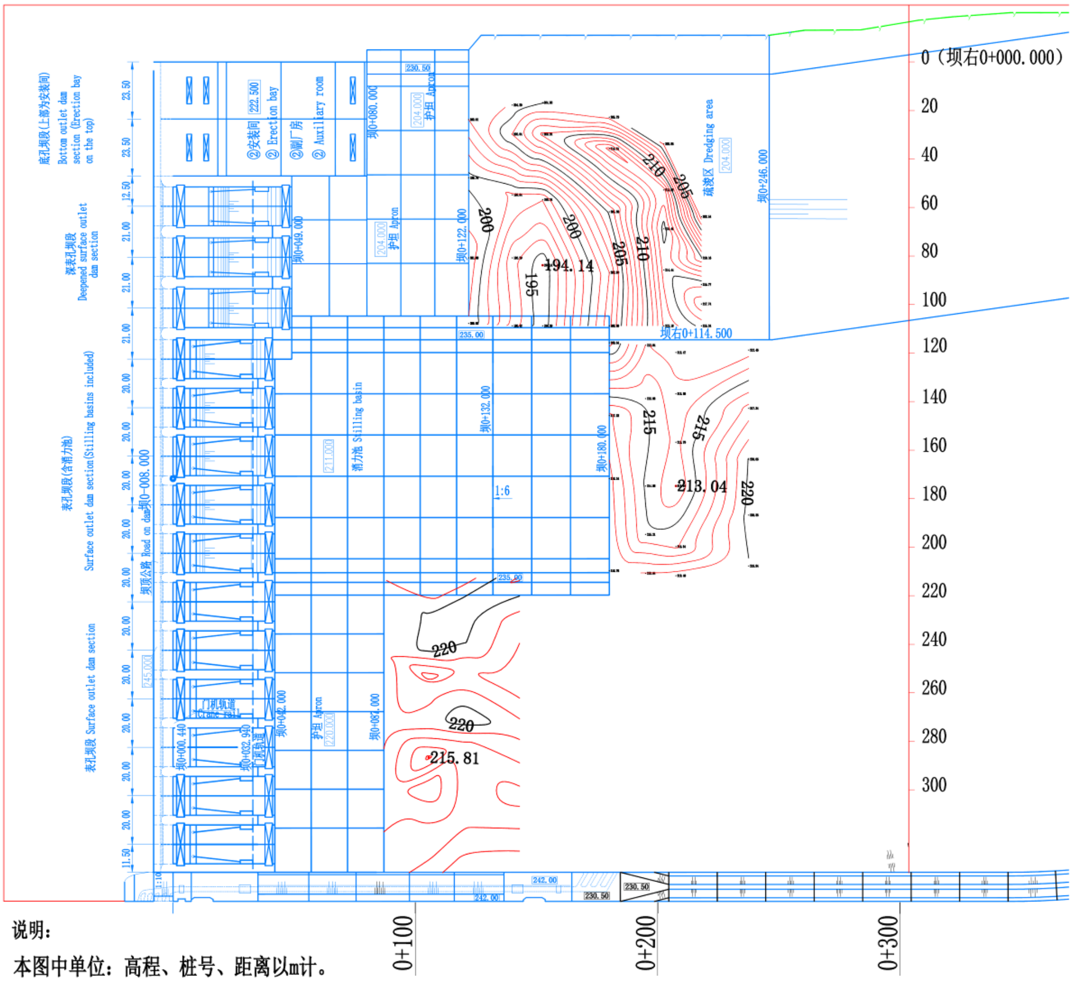


Fig. 3.3.5-1Phase II Diversion Downstream Pit

3.3.6Navigation condition

With the max. navigation flow of 16700m^3/s, the flow velocity in the gate area of the upstream and downstream approach channel is as follows.

1) When the central line of the shiplock is taken as the benchmark, the transversal flow velocity for quite a large area on the upstream side of the head of the guiding wall in the upstream approach channel is higher than 0.3m/s, see Fig. 3.3.6-1. After the ship fleet moves out of the shiplock and go straight ahead for 240m and moves towards the direction to the right bank as per a min. turning radius of 333m and a turning angle of 25°, the indicators of flow velocity at the gate entrance area can meet the requirement on navigation. See Fig. 3.3.6-2.

2) During Phase II construction, the power plant has not been put into operation

yet. Therefore, the discharged water from the flood releasing dam section will moves forward along the guiding wall in the middle and slightly to the left. A weak clockwise recirculation zone will be formed near the entrance area at the downstream approach channel and the indicators of the flow velocity can all meet the requirement on navigation. See Fig. 3.3.6-3.

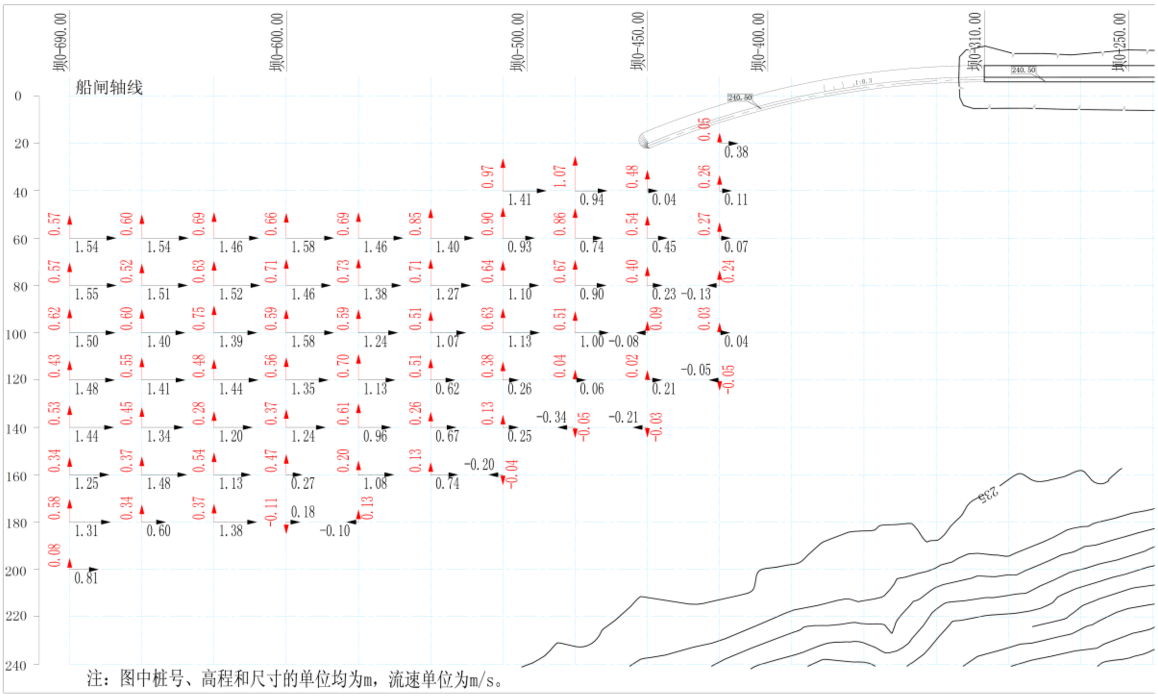


Fig. 3.3.6-1 Flow Field of Gate area at Upstream Approach Channel

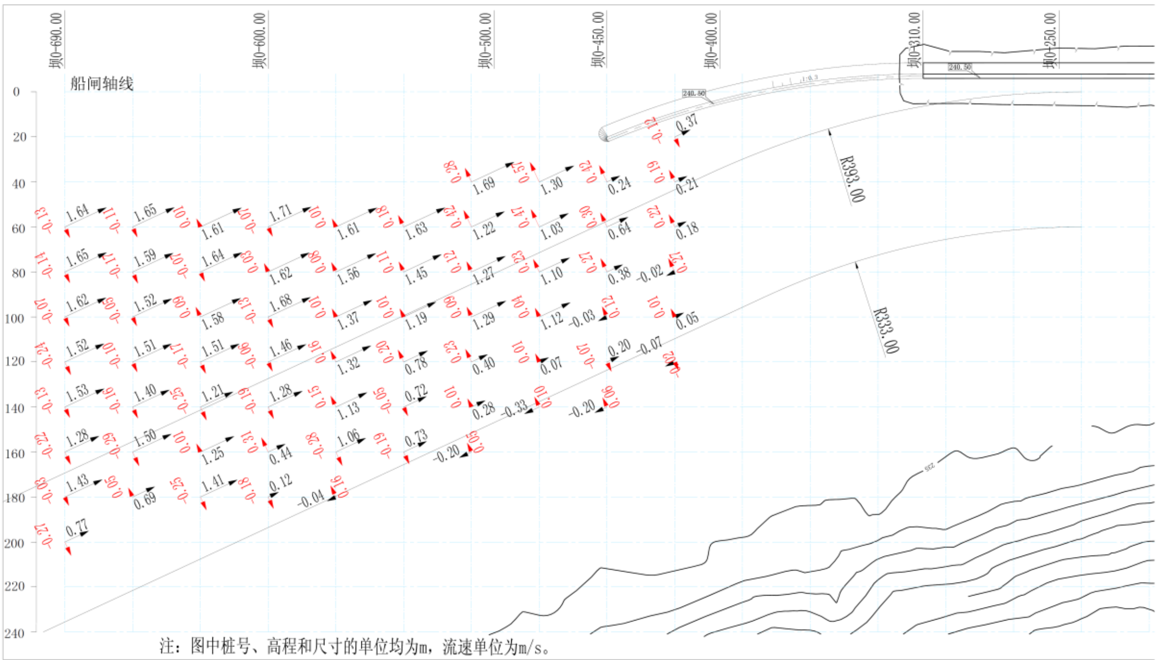


Fig. 3.3.6-2 Flow Field of Gate area at Upstream Approach Channel (with

adjusted navigation direction)

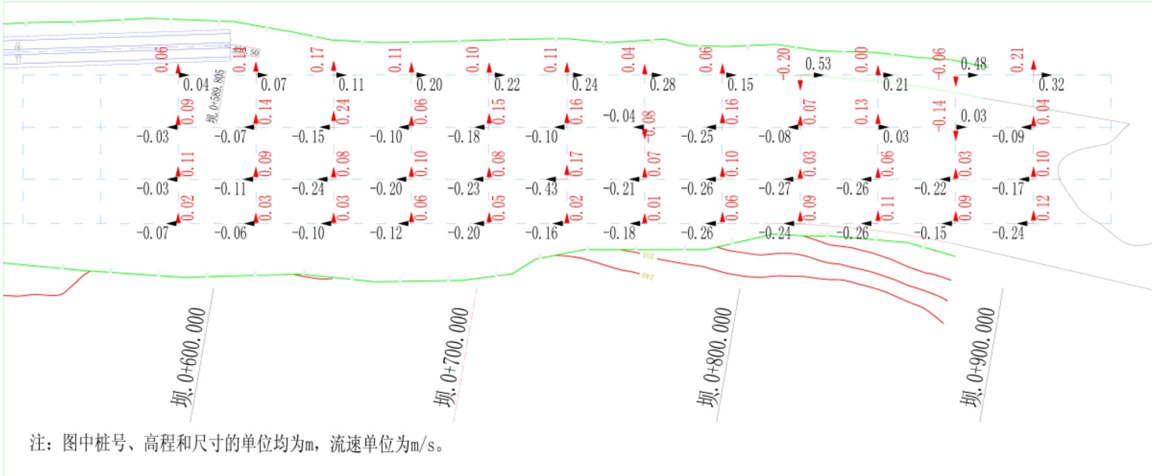


Fig. 3.3.6-3　Flow Field of Gate area at Downstream Approach Channel