# SIMULATION OF UNSTEADY FLOW IN COMPLEX HEADRACE CHANNEL OF HYDROPOWER PROJECT

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***Abstract:*** *The present paper focus on the phenomenon of unsteady flow which happens in complex headrace channel of hydropower project when flow rate suddenly changes. In practice, this problem has not been well taken into account in design and operation process, especially when the effects of intake, lateral weir and fore-bay on the headrace channel simultaneously considered. In headrace channel, the unsteady flow usually leads to wave propagation in the channel and lateral overflows, for example, it caused considerable damages in the case of Nho Que 3 hydropower project. Therefore, this article aims to simulate this unsteady state in the headrace channel in order to explain waves and overflows induced in headrace channel of above project and to prove that simulation results are in accordance with the experimental ones. Then, some solutions and operation rules are proposed to reduce the unexpected damages provoked by this unsteady flow.*

***Keywords:*** *Unsteady flow, Complex headrace channel, HEC-RAS, Nho Que 3 Hydropower project.*

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## 1. Introduction

Hydropower is the renewable energy which has been used for a long time. In 2015, hydropower projects generated 16.6% of the total world electricity and 70% of all renewable energy, and is expected to increase about 3.1% each year for the next 25 years (Source: International Hydropower Association). In Vietnam, according to Decision N° 428/QĐ-TTg, the development of hydroelectricity will be the priority in order to increase the installed hydropower capacity from 17 000 MW in 2015 to about 21 600 MW in 2020 and 27 800 MW in 2030. Many hydropower plants are and shall be under construction for the incoming years in Vietnam.

A lot of hydropower projects use open channels to transport water from intake to penstock, called headrace channels. In design and operation process of hydropower projects, both steady and unsteady flow in the headrace channel must be considered. The unsteady flow arises when there is a sudden change of flow rate at the headrace channel boundary. The calculation of unsteady flow is based on the Saint - Venant equations [1,2]. However, the theoretical calculation of unsteady flow in headrace is limited to several simple cases, mostly when the geometry of headrace is simple. In reality, the unsteady flow in headrace channel remains complicated, especially when the effects of upstream boundary (intake with the operation rule of gate), downstream boundary (unsteady flow triggered by surge tank or turbine operation) and the lateral structures such as lateral weir and culvert are considered.

In the literature, unsteady flow attracted the attention of the scientific community, including the authors solved the Saint-Venant equations by numerical method [3-6]. Therefore, these methods principally dedicated to the derivation of these equations, the effects mentioned above were not considered. In line with the development of numerical codes and computer performance, numerical simulation became a significant tendency [7-12]. Many programs are developed, available under commercial, free or open source license, in order to simulate the unsteady flow. Among them, HEC-RAS, a free software developed by the US Army Corps of Engineers-Hydrologic Engineering Center is a robust program and suitable for analysis of unsteady flow in headrace channel. It is robust and powerful software which allows to perform one-dimensional steady

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flow, one and two-dimensional unsteady flow, sediment transport/mobile bed computations, and water temperature/water quality modeling. This software is widely used over the world and recommended by a number of famous academic and governmental organizations.

In this paper, HEC-RAS will be used to study the incidents happened at Nho Que 3 hydropower plant, located in Ha Giang province and placed in operation in 2012, where the overflows were observed in its headrace channel in operation process and up to now, the reason of this phenomenon is still unclear. This study is carried out with the aim to clarify that reason and propose some effective solutions to reduce the damages.

## 2. Governing equations of unsteady flow in HEC-RAS

Unsteady flow problems arise in hydraulic engineering in a variety of settings, ranging from waves formed in irrigation channels by gate operation to natural flood waves and dam-break surges in rivers. In hydroelectric plant headraces, this unsteady flows can be induced by gate operation or by turbine operation. The types of waves considered in these situations are called translator waves because of their continuous movement along the channel. In addition, only shallow water waves are took into account, in which water movement occurs over the full depth, the vertical velocity and acceleration can be neglected to allow the use of one-dimensional forms of the governing equations. The later involves two partial differential equations representing the continuity and momentum principles. The differential form of the energy equation could be used in cases where the flow variables are continuous but the momentum equation is required where they are discontinuous. The full differential forms of the governing equations are called the Saint-Venant equations.

Depending on the velocity of the mean water velocity and the wave celerity with respect to still water, the Froude number may be smaller or greater than 1. Then, the flow in the channel may be respectively in subcritical regime or supercritical regime. Some assumptions have been made in order to derive the governing equations [6,8,9]:

+ The shallow water approximation is applied so that vertical accelerations are negligible, resulting in a vertical distribution that is hydrostatic; and the water depth is small compared to the wave length;

+ The channel bottom slope is small;

+ The channel bed is stable, meaning that the bed elevations do not change with time;

+ The flow can be represented as one dimensional flow with (a) a horizontal water surface across any cross section such that transverse velocities are negligible and (b) an average boundary shear stress that can be applied to the whole cross section;

+ The frictional bed resistance is the same in unsteady flow as in steady flow, so that the Manning or Chezy equations can be used to evaluate the mean boundary shear stress.

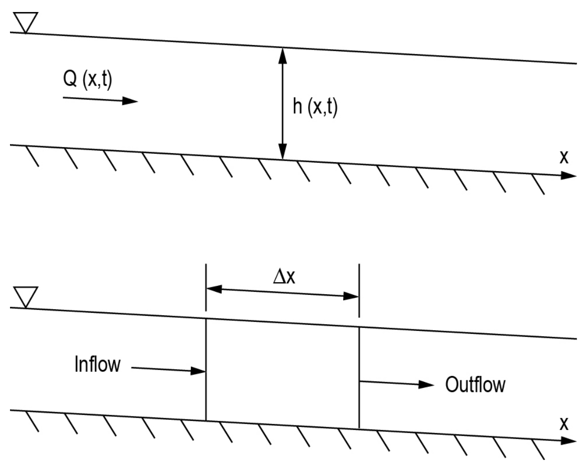


***Figure 1.*** *Elementary control volume for derivation of continuity and momentum equations (HEC-RAS)*

With the following assumptions, the continuity equation can be written for an elementary segment (Fig. 1) as:

$$\frac{\partial A_T}{\partial t}+\frac{\partial Q}{\partial t}-q_1=0 \qquad (1)$$

in which $Q$ denotes the flow, $A_T$ is the total flow area and $q_1$ is the lateral inflow per unit length.

The momentum equation can be obtained from the conservation of momentum expressed by Newton's second law as following:

$$\frac{\partial Q}{\partial t}+\frac{\partial QV}{\partial x}+gA\left(\frac{\partial z}{\partial x}+S_f\right)=0 \qquad (2)$$

where $g$ is gravity acceleration, $V$ is flow velocity, $A$ is cross-sectional area and $S_f$ is the friction slope.

While the governing equations of unsteady flow were clearly represented by Saint-Venant equations, for a long time they could not be used successfully in engineering practice. The reasons for such situation were both mathematical complexity of the equations and specific properties of open channels. Only in rather severe simplifications of the governing equations are analytical solutions available for unsteady flow. Currently, for solving the system of Saint-Venant equations, the finite difference method dominates. For instance, the following numerical schemes are able to solve the Saint-Venant equations: Lax scheme, leap-frog scheme, Abbot-Ionescu scheme, Delft Hydraulic Laboratory scheme, Vasiliev scheme, Gunaratman-Perkins scheme and Preissmann scheme [3,4]. Up to now, it commonly accepted that the most robust scheme is the four point implicit difference scheme or more precisely the Preissmann scheme. Its alternative name is the box scheme. This scheme is also used in HEC-RAS program which will be used to model the case study in the next section.

## 3. Nho Que 3 hydropower plant - A case study

### *3.1 Overview of Nho Que 3 hydropower plant*

Nho Que 3 hydropower project located on Nho Que River, Meo Vac district, Ha Giang province with the installed capacity of 110 MW using two Francis turbines. In this project, water from the reservoir is transported to the turbine hall by a number of structures in series. In the downstream direction, water flows firstly through the intake, following by a headrace channel of 2.7 kilometers long, a fore-bay then 1.8 kilometers of pressurized tunnel, a surge tank and finally the penstock (Fig. 2).

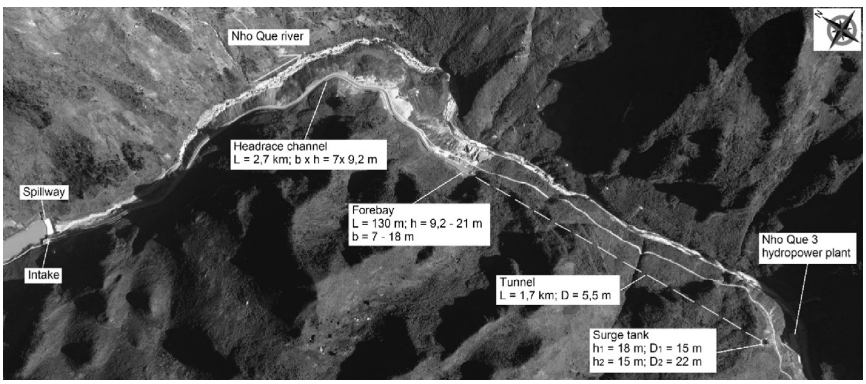


***Figure 2.*** *Layout of Nho Que 3 hydropower project (Google Earth)*

These structures transporting water of Nho Que 3 project is complex, they have both non pressurized structures and pressurized structures. The designed flow rate in the channel is 92.62m$^3$/s.

### *3.2 Summary of incidents at Nho Que 3 hydropower project*

In the shop drawings of the project, a lateral weir of 100 meters long was designed at the beginning of the headrace channel. Its elevation is equal to normal water level in reservoir and equal to 360.0 m. Therefore, this structure was rarely used. The elevation of the top of the fore-bay is 360.0 m.

***Figure 3.*** *Water waves observed in the headrace channel of Nho Que 3 hydropower project (h = 9.0m; b = 7.2m), viewed from the intake*

In reality, the operational staff observed that when the flow rate Q suddenly changed due to the turbine operation or the intake gate operation, an overflow arises on about 1100m along the headrace channel from the fore-bay. In order to reduce the overflow, the owner of Nho Que 3 project cut a part of left wall of the fore-bay to form a lateral weir of 110m long to the elevation 359.3m and reinforced the right wall to the elevation 360.7m. However, this solution cannot reduce the observed overflow. Especially, on 21/05/2015, a flood arrived to the reservoir causing the water level increased to 362m. At the same time, due to a technical incident, the plant was placed outside the distribution network, so that the overflow was violent and destroyed the road below the fore-bay on the left side. Fig. 3 shows the waves observed in the headrace channel when the turbine suddenly closes.

## 4. Simulation of the unsteady flow in headrace channel by HEC-RAS

In this section, the one-dimensional unsteady flow in headrace channel will be modelled by HEC-RAS with the consideration of lateral weir at the beginning of headrace channel, at the fore-bay; with the gate operation rules at the intake and the oscillating flows induced by the surge tank through the tunnel. The headrace channel of Nho Que 3 hydropower plant is modeled in HEC-RAS as shown in Fig. 4.

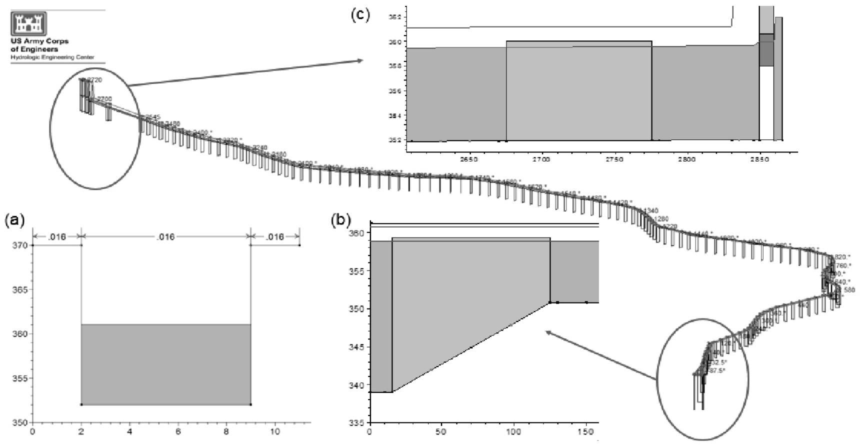


***Figure 4.*** *Simulation of headrace channel with auxiliary structures.*
*(a) Typical headrace channel cross-section b × h = 7 × 9,2 m; (b) Fore-bay with lateral weir of 110 m long; (c) Intake with gate and lateral weir of 100 m long*

It is necessary to have the initialization and the boundary conditions for the simulation of unsteady flow. The upstream initialization condition is the water level in reservoir and the downstream one is the water surface level at the fore-bay. These initial conditions are obtained from the steady flow simulation. The upstream boundary conditions are the inflow of reservoir, the elevation versus volume curve of the reservoir and the gate operation rules at the intake. To have a flow rate of 92.62m$^3$/s in headrace channel, the gate opening is equal to 2.44m when water level in reservoir equal to 360.0m (normal water level) and this opening must be 6.02m when the water level is at the dead level (358.0m). The downstream boundary conditions are the oscillating flows induced by the surge tank. These oscillating flows damp with time (Fig. 5). The lateral weirs rules are represented by a coefficient provided in HEC-RAS.

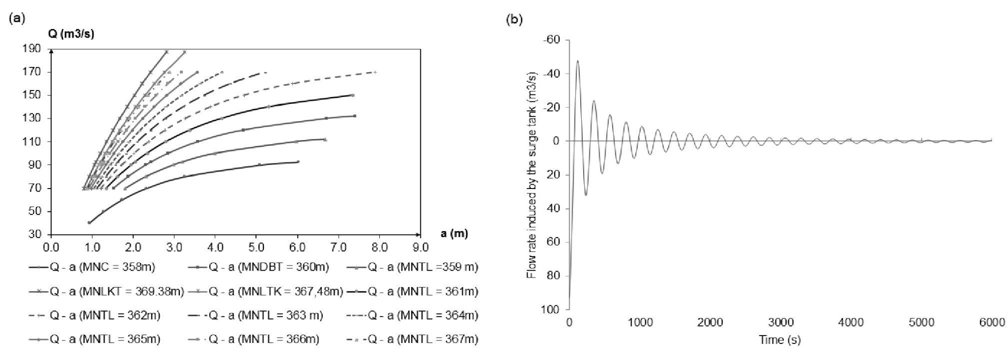


***Figure 5.*** *(a) Relation between flow rate and gate opening for several water levels in reservoir; (b) Flow rate induced by the surge tank coming to the fore-bay*

Many scenarios are built in order to investigate the reasons of overflow phenomenon in the study site. We focus on the following cases in operation process (Table 1).

The simulation results show that in the normal operation state (CS1 and CS2), the water surface elevation and the flow rate in headrace channel is in accord with the theoretical ones provided by the design consultant.

In CS3, the flow rate is 128m$^3$/s, greater than the designed flow rate of turbine which is 92.62m$^3$/s. The maximal water level at the lateral weir at the beginning of the headrace channel is 359.71m, does not exceed the top the weir. The water level in the fore-bay increases to 359.55m, exceeds the top of lateral weir

***Table 1.*** *Case studies in operation process of Nho Que 3 hydropower plant*

| Scenarios | State of hydropower plant | Description |
|---|---|---|
| CS1 | Normal state | Generated capacity 110 MW; normal water level (360.0m) in reservoir. flow rate in headrace channel is equal to 92.62m$^3$/s |
| CS2 | Normal state | Generated capacity 110 MW; dead water level (358.0m) in reservoir. flow rate in headrace channel is equal to 92.62m$^3$/s |
| CS3 | Incident state: normal water level, gate at the intake is blocked | Generated capacity 110 MW; the gate is blocked at 6.02m. and the water level in reservoir increases at 360.0m. The required time to totally close the gate is about 60 minutes. |
| CS4 | Incident state: maximal flood water level, gate at the intake is blocked | Generated capacity 110 MW; the gate is blocked at 2.44m. the maximal flood arrives and the water level in reservoir increases at 367.0m. The required time to totally close the gate is about 60 minutes. |
| CS5 | Incident state: normal water level, hydropower plant suddenly leaves the network | Generated capacity rapidly becomes zero; the initial gate opening is 6.02m. The reservoir is at normal level (360.0m). The gate starts closing at 18 minutes after the incident and the gate is totally closed after 20 minutes as in the real operation process. |
| CS6 | Incident state: water level at +362,0 m, hydropower plant suddenly leaves the network | Generated capacity rapidly becomes zero; the initial gate opening is 6.02m. The water level in the reservoir is 362.0m. The gate starts closing at 18 minutes after the incident and the gate is totally closed after 20 minutes as in the real operation process. This case study represents the event happened on 21/05/2015. |

at 359.3m. The maximal overflow rate is 10.12m$^3$/s. In CS4, the flow rate is 155m$^3$/s, greater than the designed flow rate of turbine which is 92.62m$^3$/s. The maximal water level of the lateral weir at the beginning of the headrace channel is 360.67m, exceeds the top of the weir with a maximal overflow rate is 39.41m$^3$/s. The water level increases in the fore-bay to 359.95m, exceeds the top of the lateral weir at 359.3m. The maximal overflow rate is 51.2m$^3$/s. In CS5, the flow rate is 128m$^3$/s, greater than the designed flow rate of turbine which is 92.62m$^3$/s. The maximal water level of the lateral weir at the beginning of the headrace channel is 360.69m at 8 minutes after the incident, exceeds the top of the this weir with a maximal overflow rate is 29.39m$^3$/s. The water level increases in the fore-bay to 360.1m at 13 minutes after the incident, exceeds the top of the lateral weir at 359.3m. The maximal overflow rate is 82.2m$^3$/s. There is not overflow in the rest of the headrace channel.

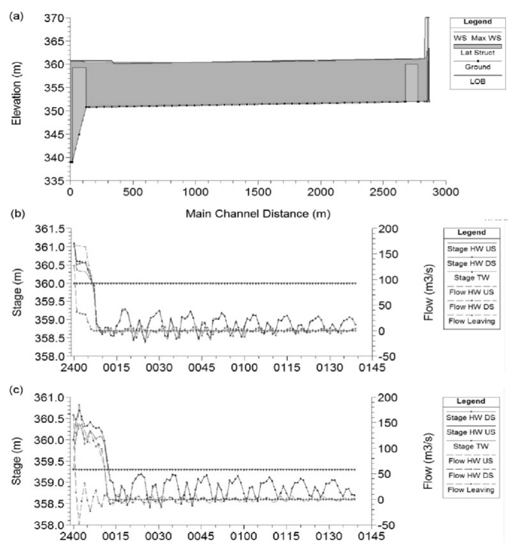


***Figure 6.*** *Simulation results of CS6. (a) Profile of maximal surface elevation; (b) Stage and flow rate hydrographs at the lateral weir at the beginning of the headrace channel; (c) Stage and flow rate hydrographs of the lateral weir at the fore-bay*

In CS6, the flow rate is 165m$^3$/s, greater than the designed flow rate of turbine which is 92.62m$^3$/s. The maximal water level of the lateral weir at the beginning of the headrace channel is 360.87m at 8 minutes after the incident, exceeds the top of this weir with a maximal overflow rate is 87.29m$^3$/s (Fig. 6). The water level increases in the fore-bay to 360.64m at 14 minutes after the incident, exceeds the top of the lateral weir at 359.3m. The maximal overflow rate is 175.6m$^3$/s. An overflow is observed on 1100m along the headrace channel from the fore-bay as in the reality (Fig. 6).

In addition, several cases of different required time to close the gate have been carried out. The results show that the overflow is proportional with the time of gate closing process. To prevent the maximal water surface elevation arise in the headrace channel, the gate closing process must be finished before 8 minutes from the beginning of the incident.

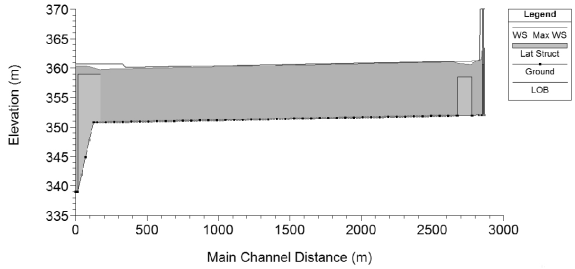


***Figure 7.*** *Maximal water surface elevation after modification of lateral weirs with the same conditions as CS6*

We have a good agreement between the simulation results and the observations in-situ. According to these results, the overflow in headrace channel is strongly influenced by the operation rules of intake gate and turbine through the oscillating flow induced by the surge tank. In addition, it is clear that the efficiency of the lateral weir at the beginning of the headrace channel is not efficient. In reality, this structure was rarely used. From these remarks, we try to propose a solution in order to reduce the overflow along the headrace channel. This idea consists to improve the efficiency of the lateral weir at the beginning of this channel. We propose to lower the lateral weir at the beginning of the headrace channel to 385.5m. Simultaneously, it is necessary to lower the lateral weir at the fore-bay to 359.0m and extend its large from 110m actually to 160m. The simulation results shows that the overflows only arise at the lateral weirs at the beginning of the headrace channel and at the fore-bay with the same conditions in the case CS6. There is no overflow in the rest of the headrace channel (Fig. 7), meaning that the reason of overflow are adequate and the proposed solutions took effects.

## 5. Conclusion

The present paper deals with the unsteady phenomena in the headrace channel of hydropower project. This unsteady flow is very complex when we simultaneously consider the effects of other structures such that the lateral weirs, the operation rules of the gate and of the turbine. With the consideration of the whole scheme from the intake to the turbine hall, several phenomena in the headrace channel have been observed and explained. HEC-RAS is a free software, but its efficiency and performance were proved.

For the case of Nho Que 3 hydropower project, this research helped to find the reason of the overflow which arisen in the headrace channel in unsteady state. The operation rules of gate and turbine had a strong impact in the unsteady state. The proposed solutions helped to reduce the overflow in headrace channel, including two measures: reduce the close time of the gate and lower the lateral weirs peak elevation at the headrace channel, which are submitted to the owner of the project for approval.

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