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Edited by Xiaohui Lei

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Meet the editor

Prof. Xiaohui Lei works at the China Institute of Water Resources and Hydropower Research. He is currently the leading expert in water resources allocation and scheduling in China. Over the past 10 years, he focused his work on the integrated regulation of multiple water projects. Prof. Lei solved a number of complex problems including uncertainty of hydrological forecasting, curse of dimensionality and multi-objectives of water resource scheduling, nonlinearity, and strong coupling of engineering control. Currently, he holds many university positions in China, including those at Hohai University, Taiyuan University of Technology, Chang'an University, etc. He is also the Chairman of Cascade Reservoirs and Water System Operations Working Group in the International Association for Hydro-Environment Engineering and Research (IAHR), the Director of Water Cycle and Operation Specialized Committee in the Chinese National Committee on Large Dams (CHINCOLD), and Deputy Secretary General of Water Diversion Specialized Committee in the Chinese Hydraulic Engineering Society (CHES), etc.

Contents

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Preface

The Middle Route of the South-to-North Water Diversion Project (MRP) is strategically important for China, and has made a great contribution to the sustainable development of society and economy in North China. Various potential risks of sudden water pollution accidents are distributed along the long canal, which may result in huge losses and endanger the safety of water supply. In order to provide technical support for emergent operations in the MRP, the emergency operation technologies for sudden water pollution accidents were developed. This book introduces these emergency operation technologies, including simulation technology for hydrodynamic and water quality in the main canal, traceability technology, emergency operations, and an emergency management system for sudden water pollution accidents.

This book is aimed at managers and technicians of the water diversion projects, and researchers in various fields, including hydraulics, emergency control, project management, and decision support platform.

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Chapter 1

Introduction - Emergency Operation Technologies for Sudden Water Pollution Accidents

Xiaohui Lei, Hezhen Zheng and Lingzhong Kong

1. Long-distance water diversion projects

Water is an important basic resource for human survival, social and economic development, and ecological environment stability [1]. With population growth, economic development, and climate change, many countries and regions face water shortage due to the uneven spatial-temporal distribution of water resources [2, 3]. Shortage of water resources will affect food safety, economic development, and ecological environment health. This is one of the major difficulties that the world has to face in the future and a major obstacle to realize sustainable development [4, 5]. It is an important way to solve the crisis of water resources in some areas by carrying out water diversion projects to transfer water from the areas where water is abundant to the places lacking of water [6].

1.1 Main long-distance water diversion projects in the world

It has been a long history of building water diversion projects by all the countries around the world. The earliest project can be traced back to 2400 BC in the ancient Egypt, transferring the water of the Nile River to irrigate the southern Ethiopia plateau. With the development of social economy, the distance, range, transferable water amount, benefit, construction, and management levels of water diversion projects are gradually improved [7]. Until now, 350 long-distance water diversion projects have been built in at least 39 countries [8]. The following are some typical ones in the world.

1.1.1 California north-to-south water diversion project (America)

The main work of this Project Phase I was completed in 1973, successfully providing industrial and living water for 17 million people centering around Los Angeles. The main canal is 1138 km long in total, with a multi-year average water diversion amount of 5.2 billion m^3 and a water diversion flow of 284 m^3/s.

1.1.2 Central Arizona Project (America)

This project started from 1968, from Lake Havasu in the west to Tucson in the southeast. The main canal is totally 539 km long with a multi-year average water diversion amount of 1.85 billion m^3, and the water diversion amounts of three canals are 85, 78, and 62 m^3/s, respectively.

1.1.3 Quebec water transfer project (Canada)

This project started in 1974 and was planned mainly for irrigation, with excess water for water power development. The main canal is 861 km long in total, with a multi-year average water diversion amount of 25.2 billion m^3 and a water diversion flow of 1590 m^3/s.

1.1.4 Volga to Moscow water diversion project (the former Soviet Union)

This project started in 1932, transferring water from the Volga River to Moscow. The main canal is 224 km long in total, with a multi-year average water diversion amount of 2.1 billion m^3 and a water diversion flow of 78 m^3/s.

1.1.5 West to east water transfer project (Pakistan)

This project started in 1960, transferring water from the Indus River to the east which is used mainly for irrigation and for power generation. The main canal is 622 km long in total, with a multi-year average water diversion amount of 14.8 billion m^3 and a water diversion flow of 614 m^3/s.

1.1.6 Snowy Mountains Scheme (Australia)

This project started in 1949 and is mainly used for power generation with re-regulated water for irrigation. The pipeline and tunnel are totally 224 km long, with a multi-year average water diversion amount of 1.13 billion m^3.

1.2 Main long-distance water diversion projects in China

China's total water resources are about 2812.4 billion m^3, ranking sixth in the world, but the per capita water resource quantity is only 1/4 of the world average. Moreover, the temporal and spatial distribution of water resources in China is very uneven, with more water in the south and less water in the north. This aggravates the shortage of water resources. In order to solve this problem, China began the construction of water diversion projects very early. From 486 to 219 BC, China successively carried out the Hanggou Project transferring the water in the Yangtze River into the Huaihe River, the Honggou Project transferring the water in the Yellow River into the Huaihe River, etc. Since 1949, China has completed construction of a series of water transfer projects, which play an important role in alleviating the uneven distribution and tense situation of water resources and promoting the development of local economy. There are approximately 20 main long-distance water diversion projects in China. The following are some typical ones:

1.2.1 Water diversion project from the Yangtze River to the northern plains of Jiangsu Province

This project has the water in the Yangtze River pumped at Jiangdu Station, Jiangdu County, Yangzhou, Jiangsu Province, China. Water is delivered to Xuzhou via 10 stages of lift pumping stations and regulation and storage in Hongze Lake and Luoma Lake. The route is more than 400 km in total, with a water diversion flow up to 470 m^3/s. This project can irrigate farmland of 2,799,000 hm^2 and is a major part of the eastern route of south-to-north water diversion project.

1.2.2 Dongjiang-Shenzhen water supply project

This project transfers water from Dongjiang River (a branch of Zhujiang River) and delivers water to Shenzhen Reservoir via nine stages of lift pumping stations and then to Hong Kong through pipelines. Since its construction that began in 1964, the project has been expanded two times. Now it supplies 620 million m^3 for Hong Kong each year, which accounts for 60% of total water amount used in Hong Kong and 1500 million m^3 for Shenzhen each year.

1.2.3 Luanhe-Tianjin water diversion project

This project is an inter-basin water diversion project, which supplies water from the basin of the Luanhe River to Tianjin and Tangshan in the basin of the Haihe River. It started in 1982 and ended in 1986, with an annual water diversion amount up to 1.95 billion m^3.

1.2.4 Shandong Yellow River to Qingdao Project

This project transfers water from Dayuzhang Diversion Sluice in the lower reaches of the Yellow River (with a design water diversion flow of 75 m^3/s) and delivers water Qingdao gradually via three stages of lift pumping stations (with a total head of 29.15 m). The route is totally 262 km long, among which the canal is 213 km long and the pipeline 22 km. The annual water diversion amount is 635 million m^3.

1.2.5 Datong River to Qinwangchuan Basin Project

This project is a large-scale gravity irrigation project which transfers the water in the Datong River to the Qinwangchuan Basin 60 km from the north of Lanzhou. The design water diversion flow is 32 m^3/s, and the increased water diversion flow is 36 m^3/s. The project can irrigate an area of 860,000 mu and transfer water of 443 million m^3 each year after completion. Among the aqueduct, the tunnel is 74.9 km long, and the longest tunnel, Pandaoling tunnel, is 15.7 km long. It is the longest tunnel which has been completed in China up to now.

1.2.6 South-to-north water diversion project

This project is one of China's strategic projects, with eastern, middle, and western routes planned, involving 438 million people in the planned areas and with a water transfer scale of 44.8 billion m^3. The project is built to solve water shortage in the regions of Northern China, especially in Huang-Huai-Hai River Basin. An entire layout with three main vertical routes and four main horizontal routes is formed by connecting three water transfer routes to the Yangtze River, the Yellow River, the Huaihe River, and the Haihe River. The purpose is to facilitate the realization of the reasonable allocation pattern for south-to-north water diversion and east-west mutual aid in China. The middle and eastern routes of south-to-north water diversion project (Phase I) has been completed and started transferring water to Northern China. Up to now, the Western Route is being planned, but the construction has not started yet.

1.3 Middle route of the south-to-north water diversion project (MRP)

Among the south-to-north water diversion project, MRP plays the most strategic role. It will be introduced in the following three aspects:

1.3.1 Project overview and characteristics

China has carried out the MRP with important strategic significance in order to alleviate the shortage of water resources in Henan, Hebei, Beijing, and Tianjin, optimize the allocation of water resources, improve the ecological environment, and promote sustainable development of society and economy. The MRP passed through more than 60 years from the proposal (on October 30, 1952) to exploration, survey, planning, design, and construction. It is put into formal operation on December 12, 2014.

The MRP (**Figure 1**) transfers water from Danjiangkou Reservoir, passing through Hubei, Henan, and Hebei and crossing the Yangtze River, the Huaihe River, the Yellow River, and the Haihe River. Water is transferred in an open channel by means of gravity flow to the Tuancheng Lake in Beijing and the Waihuan River in Tianjin. The MRP is characterized by (1) a long route: the general main canal is totally 1432 km, 1277 km of which is the length of the main canal, and 155 km, the length of the Beijing to Tianjin Branch; (2) a large water transfer scale: the Phase I Project of MRP is designed with an annual water diversion amount of 9.5 billion m^3, and the design discharge of Danjiangkou Reservoir into the canal head (i.e., Taocha Gate) is 350 m^3/s; (3) many buildings: the project has various kinds of buildings (totally more than 1800). Among them, the hydraulic control buildings include 63 check gates, 1 pumping station, 97 diversion gates, and 54 drainage gates; (4) strict control condition: the operation mode is constant downstream depth without a ready-to-use equalizing reservoir and with a low water head; the canals allow a small range of change in water stage, and the fluctuation of the water stage should not exceed 0.15 m/h or 0.3 m/d under normal operating conditions; (5) high water quality requirement: the supplied water should meet the Class II standard of China's Environmental Quality Standards for Surface Water (GB3838-2002); and (6) complicated operating conditions: the water division plans along the route tend to

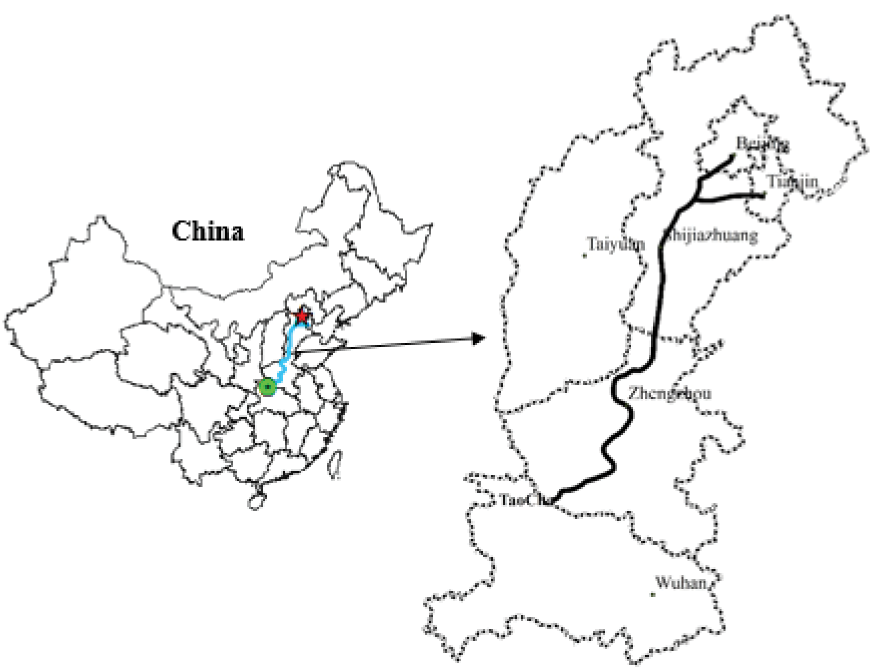

Figure 1.
Water diversion route of MRP.

change frequently. In winter, water has to be transferred in an icy period with more risks. These characteristics make it difficult to operate and control the main canal of the MRP.

1.3.2 Monitoring station network

In order to support the operation and scheduling of the MRP, the Construction and Administration Bureau of the MRP (hereinafter called "CABMRP") has arranged a lot of monitoring stations along the route, including water amount and quality, engineering safety, video camera, etc. The water amount monitoring station network is made up by monitoring devices (flowmeter, water gauge, etc.) at more than 300 gate stations. The measured data for this research include the upstream and downstream water levels, opening and discharge of all check gates, and the discharge and water supply amount of diversion gates and drainage gates (some of the drainage gates are used as diversion gates sometimes). The water quality monitoring station network is comprised of 28 fixed monitoring stations, 13 automatic monitoring stations, 2 movable laboratories, etc. However, the measured water quality data have not been obtained due to various kinds of reasons.

1.3.3 Risk of sudden water pollution accidents

The MRP passes through large regions, involving totally 2218 enterprises producing dangerous chemicals, 1238 cross canal bridges, and many rivers and roads. These may cause sudden water pollution accidents, which endanger water quality safety, for example, a vehicle loaded with toxic materials falls into the canal due to traffic accidents; someone commits malicious poisoning; or the pollutants on the earth surface are brought by rainstorm or flood into the canal.

2. Sudden water pollution accidents

A sudden water pollution accident refers to a sudden event due to artificial or natural factors, etc., which brings plenty of pollutants into a water body. It can result in deteriorated water quality, affected living and production water and huge economic loss, endangering ecological environment and lead to a bad impact on the society [9].

Over the past few decades, a large number of sudden water pollution accidents have occurred worldwide. Major cases are Chemical Pollution Accident in the Rhine in 1986 [10], Benzene Pollution Accident in the Songhua River of China in 2005 [11], Oil Spill Accident in the Gulf of Mexico in 2010 [12], Nuclear Accident in Fukushima, Japan [13], etc. The basic situation and hazards of the four above accidents are simply indicated in **Figure 2** and **Table 1**.

The challenges of real world spills and accidents are as follows: First, the spread of pollutants is too fast and complicated to be monitored accurately. Second, the emergency measures are difficult to determine quickly and reasonably. Third, how to deal with the accidents mainly depends on human experience.

Long distance water diversion projects provide an important guarantee for the development of social economy, human living, ecological environment health, and so on. Under the background of frequent occurrence of all kinds of sudden water pollution accidents occur frequently, that long distance water diversion projects suffer from water pollution accident risks has become a water supply safety factor that cannot be ignored. Once a sudden water pollution accident occurs, great harm and bad social influence will be brought.

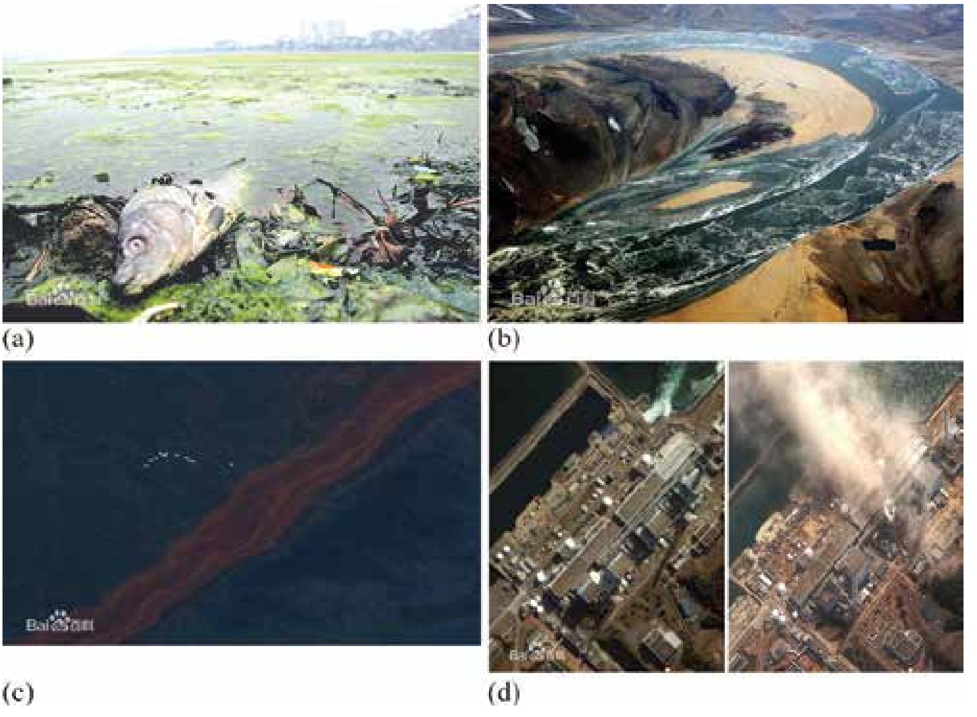

Figure 2.
Sudden water pollution accidents. (a) Chemical pollution accident in the Rhine; (b) benzene pollution accident in the Songhua River of China; (c) oil spill accident in the Gulf of Mexico; (d) nuclear accident in Fukushima, Japan.

Time	Sudden water pollution accidents	Overview and hazards of the accidents
November 1986	Chemical pollution accident in the Rhine	The warehouse of Schweizerhalle Corporation near Basel, Switzerland, caught a fire, and nearly 10,000 m^3 of fire water polluted by toxic materials flew into the Rhine, forming a reddish pollution zone of 70 km long, killing tons of fish, and causing closedown of all water works along the river.
November 2005	Benzene pollution accident in the Songhua River of China	One of the workshops of the Double Benzene Plant of Jilin Petrochemical Company exploded, and about 100 tons of benzene homologs and derivatives flew into the Songhua River, resulting in severe pollution of the water therein, and affecting the living of millions of people along the bank of the river.
April 2010	Oil spill accident in the Gulf of Mexico	A coastal oil drilling platform in Louisiana in the south of the United States exploded, leading to spill of a large quantity of oil and an economic and environmental tragedy.
March 2011	Nuclear accident in Fukushima, Japan	An earthquake happened in Japan caused Fukushima Nuclear Power Plant reactor failure, a large number of radioactive sewage was discharged into the sea, and many nearby residents evacuated.

Table 1.
Information on sudden water pollution accidents.

3. Response measures for sudden water pollution accidents

Worldwide researchers have done a lot of research on the response measures for sudden water pollution accidents, mainly involving simulation, traceability, operation and management system, and so on, which will be summarized as follows:

3.1 Simulation of sudden water pollution accidents

Grifoll et al. [14] performed simulation and risk evaluation of the sudden water pollution accident owning to oil at Port of Barcelona by applying a 3D model and then proposed control measures to prevent water quality deterioration. Galabov et al. [15] conducted simulation of oil spill pollution events that might happen at 20 locations in the Burgas Bay by using a simulation program (including a variety of wind speed and direction conditions) to assess the range of impact due to oil spill pollution. Saadatpour et al. [16] established a 2D water quality simulation model; by using this model, they carried out simulation of different degrees of pollution caused by toxic materials at the Ilam Reservoir, putting forward emergency measures. Tang et al. [17] built a 1D hydrodynamic water quality model to make analysis of the pollutant diffusion process after sudden water pollution in the Main Route Project of the MRP; set a scene with three water flow rates, four pollutants, and three pollution amounts; and recommended reasonable emergency measures. Fan et al. [18] simulated the diffusion process of pollution due to a fluorescein sodium point source in the basin of the Paraiba do Sul River under many scenes. Samuels et al. [19] performed simulation of the transfer process of pollutants in rivers.

3.2 Traceability of sudden water pollution accidents

Fulvio et al. [20] combined a one-dimensional water quality modeling river with a geostatistical method and inversely analyzed the pollutant amount on where the pollution source location was known. Zhu et al. [21] combined a water quality model and a Bayesian estimation method and obtained the location probability distribution of pollution source. Based on the mathematical theory, Wang and Xu [22] proved the uniqueness of the pollution source position, and then Wang and Qiu [23] gave the related identification method. Yang et al. [24] proposed a multi-point source identification model of sudden water pollution accidents in surface waters based on differential evolution and Metropolis-Hastings-Markov Chain Monte Carlo. Zhang and Xin [11] discussed pollution source identification for water pollution accidents in small straight rivers by using genetic algorithm.

3.3 Emergency operations of sudden water pollution accidents

Lian et al. [25] built a 2D hydrodynamic water quality model based on CE-QUAL-W2 at the Three Gorges Reservoir and put forward different scheduling rules for the Three Gorges Reservoir to inhibit the outbreak of algal blooms according to the simulation results. Long et al. [26] established a joint emergency scheduling model for canals where an accident occurred and upstream and downstream sections thereof based on the risk evaluation of sudden water pollution accidents in the MRP. Xu et al. [27] set up a rapid emergency scheduling model for canals where an accident occurred in the MRP and put forward the method to operate two check gates in such canals. Zheng et al. [28] developed a 3D hydrodynamic water quality model based on the EFDC model at the Danjiangkou Reservoir and according to the simulation result, came up with joint emergency regulation and control methods for Danjiangkou Dam and Taocha Dam. Cheng and Qian [29] established an evaluation model based on a fuzzy comprehensive evaluation method to evaluate whether the emergency measures for sudden water pollution accidents are feasible. The indexes of evaluation are completeness, operability, effectiveness, flexibility, rapidity, and reasonableness.

3.4 Emergency management system for sudden water pollution accidents

Dobbins [30] developed a decision-making support system for controlling the pollution risks arising out of inland water navigation accidents. Nine countries including Germany, Austria, etc. have developed "an emergency early warning system for Danube emergency accidents" with complete dangerous material database and accurate pollutant effect simulation capacity. The system serves in the simulation of emergency response to and other work of sudden water pollution accidents that happen in the Danube [31]. Bildstein et al. [32] integrated a water quality model in software called "SeauS," which can provide technical support for the simulation and treatment of sudden water pollution accidents. Rui et al. [33] integrated a hydrodynamic water quality model into a sudden water pollution accident response system based on GIS, and they applied it to the simulation and early warning of sudden water pollution accidents (TP and COD_{Mn}) at Xiangjia Dam.

3.5 Summary

Based on the analysis of the above literatures, simulation, traceability, regulation, control, management system, and other technologies should be included with regard to response measures for possible sudden water pollution accidents in the MRP. Among them, the hydrodynamic and water quality simulation technology can be used for predicting the pollutant transportation process after a sudden water pollution accident and for analyzing the effect of emergency operations; the sudden water pollution accident traceability technology can identify the pollution source fast and accurately; the emergency operation technology can be used to determine suitable emergency operation measures of control structures; the emergency management system can support the integration and visualization of professional model groups and provide decision-making support for engineering management companies and personnel.

4. About this book

This book provides technical support for potential sudden water pollution accidents in the MRP, with the main purpose of developing a practical emergency management system, by developing a 1D hydrodynamic and water quality model, a sudden water pollution accident source identification model, and an emergency operation model and finally integrating these models into the emergency management system. The chapters herein are arranged as follows:

Chapter 1 Introduction - Emergency Operation Technologies for Sudden Water Pollution Accidents: This chapter presents a simple introduction of the current development situation of long-distance water diversion projects in the world, focusing on the basic overview of the MRP and the existing risks that might arise from sudden water pollution accidents. The main contents of the book, the key technologies including "simulation-traceability-operation-system" for coping with sudden water pollution, are introduced through analysis of major hazards of sudden water pollution accidents and the current research of the response measures.

Chapter 2 Simulation Technology for Hydrodynamic and Water Quality in the Main Canal: This chapter provides a simple introduction of hydrodynamic water quality simulation software generally used at present, focusing on a method to build a 1D hydrodynamic and water quality model for the main canal of the

MRP. The chapter also proves the precision of the hydrodynamic and water quality model by using the measured data and comparing with MIKE11 model, respectively.

Chapter 3 Traceability Technology for Sudden Water Pollution Accidents in Rivers: This chapter simply introduces the basic principles of sudden water pollution traceability and the traceability methods used currently, focusing on the new source identification model in the canal, and provides the results of sudden water pollution traceability technology obtained from analysis of practical applications.

Chapter 4 Emergency Operations of Sudden Water Pollution Accidents: This chapter simply introduces the strategies and goals of emergency operation of sudden water pollution accidents in the MRP, focusing on the emergency control algorithm for the gates at accident pool, upstream and downstream pools of the canal, and analyzes some cases.

Chapter 5 Emergency Management System for Sudden Water Pollution Accidents: This chapter offers the introduction of the service objects, building targets, system frame, system functions, system interface design of and system safety and maintenance rules for sudden water pollution accidents emergency management.

5. Conclusions

This chapter presents the introduction of the current development situation of long-distance water diversion projects in the world, the basic overview of the MRP, and the existing risks that might arise from sudden water pollution accidents. The contents researched for coping with sudden water pollution are summarized through analysis of major hazards of sudden water pollution accidents and the research of the countermeasures therefor made by predecessors.

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Chapter 2

Simulation Technology for Hydrodynamic and Water Quality in the Main Canal

Yu Tian, Hezhen Zheng, Xiaohui Lei and Wei Dai

Abstract

The hydrodynamic and water quality simulation technology can be used for predicting the pollutant diffusion process after a sudden water pollution accident, and for analyzing the effect of emergency operation measures. The MRP features a long route, a variety of buildings, etc.; therefore, a set of hydrodynamic and water quality models that are applicable to the main canal of the MRP was independently developed based on 1-D open canal hydrodynamic and water quality theory and with various types of buildings as inner boundaries. Through calibration and verification, these models can be applied to the simulation of hydraulic and water quality response process under any operation conditions in the main canal of the MRP.

Keywords: hydrodynamic, water quality, simulation

1. Introduction

The hydraulic model is used to obtain the temporal and spatial distribution information of water level and flow by solving a numerical method, and to describe the change relations among water body quality factors with time and space under the influence of various factors [1]. After years of research and development, the hydrodynamic water quality simulation technology has become very mature and can be divided into 0-D, 1-D, 2-D, and 3-D models from the perspective of spatial dimension. The software commonly used for hydrodynamic water quality simulation are MIKE [2], EFDC [1], WASP [3], QUAL [4], DELFT3D [5], etc.

The MRP is characterized by a long route, a variety of buildings (such as inverted siphon, aqueduct, gate, etc.) and variable water distribution at dividing gates, which lead to complicated boundary conditions and the frequent operation of gate/pump groups. These characteristics result in the fact that conventional software for hydrodynamic water quality simulation cannot be well applied to the simulation of the hydraulic and water quality response process under the operation of the gate/pump groups for the main canal of the MRP [6, 7].

The water depth and water surface width of the main canal are extremely small compared with the length, and the average slope of the main canal is 1/25000 (mild slope); in case of any sudden water pollution accident, the pollutant can mix with water uniformly at a section after moving a short distance. Therefore, the pollution can be often simplified to a 1-D water quality problem for treatment, i.e., the concentration of the pollutant in the section is uniform and changes only with water flow direction. Therefore, we independently developed a set of hydrodynamic and water quality models that are applicable to the main canal of the MRP based on 1-D open canal hydrodynamic and water quality theory.

2. Hydraulic model

2.1 Control equations

When the Saint-Venant equations [8] are used to describe a 1-D unsteady canal flow model, the following assumptions should be made:

1. 1-D flow in a channel and uniform distribution of cross section of flow;

2. undulating water surface changes gradually with a very small acceleration in the vertical direction, and the pressure on cross section of flow is consistent with the distribution law of hydrostatic pressure;

3. the frictional head loss is considered only with the local head loss neglected, and the constant can be used for calculation; and

4. $\cos\theta \approx 1$ for a small channel bottom slope.

The de Saint-Venant system of equations consists of a continuity equation and a momentum equation, and with water level Z and flow Q as variables. Its specific forms are as follows:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_l \tag{1}$$

$$\frac{\partial}{\partial t}\left(\frac{Q}{A}\right) + \frac{\partial}{\partial x}\left(\beta\frac{Q^2}{2A^2}\right) + g\frac{\partial h}{\partial x} + g\left(S_f - S_0\right) = 0 \tag{2}$$

where A = cross-section area; Q = cross-section flow; S_0 = channel bottom slope; S_f = hydraulic gradient; x = spatial coordinate; t = time coordinate; q_l = lateral inflow of channel of unit length; h = water depth; β = correction factor for uniform flow rate on cross section.

S_f can be determined according to a discharge modulus:

$$S_f = \frac{Q|Q|}{K^2} \tag{3}$$

where K = discharge modulus.

2.2 Equation discretization

The finite difference scheme in de Saint-Venant system of equations, the scheme applied mostly is the four-point implicit scheme raised by Preissmann, also called a space-time eccentricity scheme (**Figure 1**) [9]. The de Saint-Venant system of equations is dispersed by using this method in this book.

See the following figure:

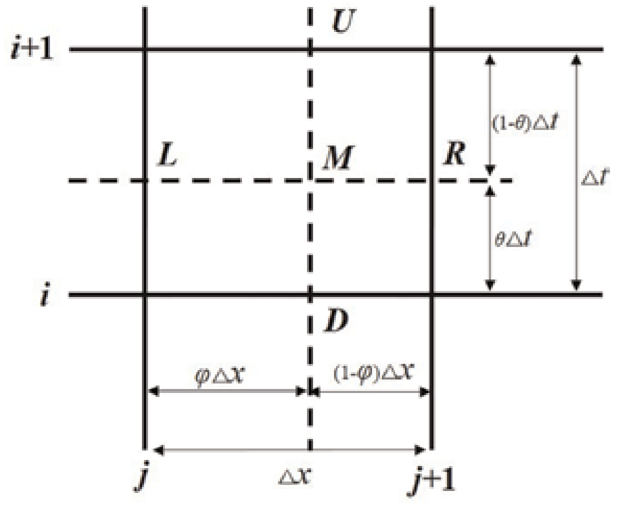

Figure 1.
Schematic diagram of Preissmann four-point implicit scheme.

where θ and φ are a time weight coefficient and a space weight coefficient, respectively, with the range of values: $0 \le \theta \le 1$, $0 \le \varphi \le 1$; j and j + 1 are the nodes in the space direction; i and i + 1 are the nodes in the time direction. Then, the functional values at points L, R, U, and D can be determined:

$$f_L = \theta f_j^{i+1} + (1-\theta) f_j^i \tag{4}$$

$$f_R = \theta f_{j+1}^{i+1} + (1-\theta) f_{j+1}^i \tag{5}$$

$$f_U = \varphi f_{j+1}^{i+1} + (1-\varphi) f_j^{i+1} \tag{6}$$

$$f_D = \varphi f_{j+1}^{i} + (1-\varphi) f_j^{i} \tag{7}$$

Then, the functional value for point M is:

$$f_M = \varphi f_R + (1-\varphi) f_L \tag{8}$$

Substituting Formulas (4) and (5) into (8), and then, we obtain:

$$f_M = \varphi\left(\theta f_{j+1}^{i+1} + (1-\theta) f_{j+1}^i\right) + (1-\varphi)\left(\theta f_j^{i+1} + (1-\theta) f_j^i\right) \tag{9}$$

Temporal discretization:

$$\begin{aligned} \left(\frac{\partial f}{\partial t}\right)_M &\approx \frac{f_U - f_D}{\Delta t} = \frac{\left(\varphi f_{j+1}^{i+1} + (1-\varphi) f_j^{i+1}\right) - \left(\varphi f_{j+1}^{i} + (1-\varphi) f_j^{i}\right)}{\Delta t} \\ &= \varphi \frac{f_{j+1}^{i+1} - f_{j+1}^{i}}{\Delta t} + (1-\varphi) \frac{f_j^{i+1} - f_j^i}{\Delta t} \end{aligned} \tag{10}$$

Spatial discretization:

$$\left(\frac{\partial f}{\partial x}\right)_M \approx \frac{f_R - f_L}{\Delta x} = \frac{\left(\theta f_{j+1}^{i+1} + (1-\theta) f_{j+1}^{i}\right) - \left(\theta f_{j}^{i+1} + (1-\theta) f_{j}^{i}\right)}{\Delta x} = \theta \frac{f_{j+1}^{i+1} - f_{j}^{i+1}}{\Delta x} + (1-\theta)\frac{f_{j+1}^{i} - f_{j}^{i}}{\Delta x} \tag{11}$$

Then, according to Formulas (9), (10) and (11), the de Saint-Venant system of equations can be dispersed into the following form:

Continuity equation:

$$\frac{\varphi}{\Delta t}\left(A_{j+1}^{i+1} - A_{j+1}^{i}\right) + \frac{1-\varphi}{\Delta t}\left(A_{j}^{i+1} - A_{j}^{i}\right) + \frac{\theta}{\Delta x}\left(Q_{j+1}^{i+1} - Q_{j}^{i+1}\right) + \frac{1-\theta}{\Delta x}\left(Q_{j+1}^{i} - Q_{j}^{i}\right) - \theta\left[\varphi q_{j+1}^{i+1} + (1-\varphi) q_{j}^{i+1}\right] - (1-\theta)\left[\varphi q_{j+1}^{i} + (1-\varphi) q_{j}^{i}\right] = 0 \tag{12}$$

Momentum equation:

$$\frac{\varphi}{\Delta t}\left(\frac{Q_{j+1}^{i+1}}{A_{j+1}^{i+1}} - \frac{Q_{j+1}^{i}}{A_{j+1}^{i}}\right) + \frac{1-\varphi}{\Delta t}\left(\frac{Q_{j}^{i+1}}{A_{j}^{i+1}} - \frac{Q_{j}^{i}}{A_{j}^{i}}\right) + \frac{\theta}{\Delta x}\left[\frac{1}{2}\left(\beta_{j+1}^{i+1}\frac{Q_{j+1}^{i+1}}{A_{j+1}^{i+1}}\right)^2 - \frac{1}{2}\left(\beta_{j}^{i+1}\frac{Q_{j}^{i+1}}{A_{j}^{i+1}}\right)^2\right]$$
$$+ \frac{1-\theta}{\Delta x}\left[\frac{\beta_{j+1}^{i}}{2}\left(\frac{Q_{j+1}^{i}}{A_{j+1}^{i}}\right)^2 - \frac{\beta_{j}^{i}}{2}\left(\frac{Q_{j}^{i}}{A_{j}^{i}}\right)^2\right] + \frac{\theta g}{\Delta x}\left(h_{j+1}^{i+1} - h_{j}^{i+1}\right) + \frac{(1-\theta) g}{\Delta x}\left(h_{j+1}^{i} - h_{j}^{i}\right)$$
$$+ \theta g\left[S_{f,j+1}^{i+1} + (1-\varphi_R) S_{f,j}^{i+1}\right] + (1-\theta) g\left[S_{f,j+1}^{i} + (1-\varphi_R) S_{f,j}^{i}\right] = 0 \tag{13}$$

where φ_R is the space weight coefficient of hydraulic gradient, with a value different from that of φ.

When the water in a channel flows slowly, the characteristic root symbols of the control equation are opposite, i.e., $\lambda_1 > 0$ or $\lambda_2 < 0$; therefore, $C_r = \lambda_1 \frac{\Delta t}{\Delta x} > 0$ or $C_r = \lambda_2 \frac{\Delta t}{\Delta x} < 0$. Because $C_r > 0$ and $C_r < 0$ exist at the same time, two discrete equations are not likely to go into an unconditional stability state at the same time. The stability condition for discrete equations is as follows:

$$C_r = \left(u - \sqrt{gh}\right)\frac{\Delta t}{\Delta x} > \frac{\varphi - 0.5}{0.5 - \theta} \tag{14}$$

2.3 Discrete equation linearization

The aforesaid (12) and (13) are the continuity equation and momentum equation that have been dispersed. The discrete equations after discretization are nonlinear, so they still need linearization and are determined. In this book, discrete equations are determined by solving water level and flow increment, i.e., Δh and ΔQ. Firstly, area and flow are transformed into an increment form:

$$A_j^{i+1} = A_j^* + \Delta A_j = A_j^* + B_j^* \Delta h_j \tag{15}$$

$$Q_j^{i+1} = Q_j^* + \Delta Q_j \tag{16}$$

where * stands for the value of the variable at the previous cycle step; ΔA, Δh, and ΔQ are the flow area, channel water depth, and flow increment, respectively; and B is the water surface width.

After substituting the aforesaid Formulas (15) and (16) into the dispersed continuity equation (12), we obtain:

$$a_j \Delta h_j + b_j \Delta Q_j + c_j \Delta h_{j+1} + d_j \Delta Q_{j+1} = p_j \tag{17}$$

where

$$a_j = (1-\varphi) B_j^* / \Delta t \tag{18}$$

$$b_j = -\theta / \Delta x \tag{19}$$

$$c_j = \varphi B_{j+1}^* / \Delta t \tag{20}$$

$$d_j = \theta / \Delta t \tag{21}$$

$$\begin{aligned} p_j = & -\frac{\varphi}{\Delta t}\left(A_{j+1}^* - A_{j+1}^i\right) - \frac{1-\varphi}{\Delta t}\left(A_j^* - A_j^i\right) - \frac{\theta}{\Delta x}\left(Q_{j+1}^* - Q_j^*\right) - \frac{1-\theta}{\Delta x}\left(Q_{j+1}^i - Q_j^i\right) \\ & + \theta\left[\varphi q_{j+1}^{i+1} + (1-\varphi) q_j^{i+1}\right] + (1-\theta)\left[\varphi q_{j+1}^i + (1-\varphi) q_j^i\right] \end{aligned} \tag{22}$$

During linearization of the momentum equation, the following formulas are needed:

$$\left(Q_j^{i+1}\right)^2 = \left(Q_j^*\right)^2 + 2Q_j^* \Delta Q_j \tag{23}$$

$$\frac{1}{\left(K_j^{i+1}\right)^2} = \frac{1}{\left(K_j^*\right)^2} - \frac{2}{\left(K_j^*\right)^3}\left(\frac{\partial K}{\partial h}\right)_j^* \Delta h_j \tag{24}$$

$$S_{f,j}^{i+1} = S_{f,j}^* + \frac{2\left|Q_j^*\right|}{\left(K_j^*\right)^2} \Delta Q_j - \frac{2S_{f,j}^*}{K_j^*}\left(\frac{\partial K}{\partial h}\right)_j^* \Delta h_j \tag{25}$$

$$\frac{Q_j^{i+1}}{A_j^{i+1}} = \frac{Q_j^*}{A_j^*} + \frac{1}{A_j^*} \Delta Q_j - \frac{Q_j^* B_j^*}{\left(A_j^*\right)^2} \Delta h_j \tag{26}$$

$$\left(\frac{Q_j^{i+1}}{A_j^{i+1}}\right)^2 = \left(\frac{Q_j^*}{A_j^*}\right)^2 + \frac{2Q_j^*}{\left(A_j^*\right)^2} \Delta Q_j - \frac{2\left(Q_j^*\right)^2 B_j^*}{\left(A_j^*\right)^3} \Delta h_j \tag{27}$$

After substituting Formulas (23)–(27) into the dispersed momentum equation (13), we obtain a linearized momentum equation:

$$a_{j+1} \Delta h_j + b_{j+1} \Delta Q_j + c_{j+1} \Delta h_{j+1} + d_{j+1} \Delta Q_{j+1} = p_{j+1} \tag{28}$$

where

$$a_{j+1} = -\frac{1-\varphi}{\Delta t}\frac{Q_j^* B_j^*}{\left(A_j^*\right)^2} + \frac{\theta}{\Delta x}\frac{\left(Q_j^*\right)^2 B_j^*}{\left(A_j^*\right)^3} - \frac{\theta g}{\Delta x} - 2\theta(1-\varphi_R) g \frac{S_{f,j}^*}{K_j^*}\left(\frac{\partial K}{\partial h}\right)_j^* \tag{29}$$

$$b_{j+1} = \frac{1-\varphi}{\Delta t}\frac{1}{A_j^*} - \frac{\theta}{\Delta x}\frac{Q_j^*}{\left(A_j^*\right)^2} + 2\theta(1-\varphi_R)g\frac{\left|Q_j^*\right|}{\left(K_j^*\right)^2} \tag{30}$$

$$c_{j+1} = -\frac{\varphi}{\Delta t}\frac{Q_{j+1}^* B_{j+1}^*}{\left(A_{j+1}^*\right)^2} - \frac{\theta}{\Delta x}\frac{\left(Q_{j+1}^*\right)^2 B_{j+1}^*}{\left(A_{j+1}^*\right)^3} + \frac{\theta g}{\Delta x} - 2\theta\varphi_R g\frac{S_{f,j+1}^*}{K_{j+1}^*}\left(\frac{\partial K}{\partial h}\right)_{j+1}^* \tag{31}$$

$$d_{j+1} = \frac{\varphi}{\Delta t}\frac{1}{A_{j+1}^*} + \frac{\theta}{\Delta x}\frac{Q_{j+1}^*}{\left(A_{j+1}^*\right)^2} + 2\theta\varphi_R g\frac{\left|Q_{j+1}^*\right|}{\left(K_{j+1}^*\right)^2} \tag{32}$$

$$\begin{aligned} p_{j+1} = &-\frac{\varphi}{\Delta t}\left(\frac{Q_{j+1}^*}{A_{j+1}^*} - \frac{Q_{j+1}^i}{A_{j+1}^i}\right) - \frac{1-\varphi}{\Delta t}\left(\frac{Q_j^*}{A_j^*} - \frac{Q_j^i}{A_j^i}\right) - \frac{\theta}{\Delta x}\left[\frac{1}{2}\left(\frac{Q_{j+1}^*}{A_{j+1}^*}\right)^2 - \frac{1}{2}\left(\frac{Q_j^*}{A_j^*}\right)^2\right] \\ &- \frac{1-\theta}{\Delta x}\left[\frac{1}{2}\left(\frac{Q_{j+1}^i}{A_{j+1}^i}\right)^2 - \frac{1}{2}\left(\frac{Q_j^i}{A_j^i}\right)^2\right] - \frac{\theta g}{\Delta x}\left(h_{j+1}^* - h_j^*\right) - \frac{(1-\theta)g}{\Delta x}\left(h_{j+1}^i - h_j^i\right) \\ &- \theta g S_{f,j+1}^* - (1-\theta)g S_{f,j}^* \end{aligned} \tag{33}$$

The aforesaid Formulas (17) and (28) are the continuity equation and the momentum equation in de Saint-Venant system of equations, respectively, after linearization via an increment method.

2.4 Boundary conditions

With the main canal of the MRP as a whole system, if all the hydraulic elements are to be determined via solving the de Saint-Venant system of equations, the various types of boundary conditions need to be defined. Normally, boundary conditions are divided into outer and inner ones. With regard to the actual situation of the general main canal of the MRP, the upstream water depth boundary and downstream flow boundary are the outer boundary conditions for value simulation, while the water level discharge process of structures such as the dividing gates, transition sections, inverted siphons, check gates, and so on in the channel system are the inner boundary conditions for value simulation.

2.4.1 Outer boundary conditions

The characteristics of upstream and downstream water connected with the general main canal are considered as the outer boundary conditions of the system. They can be roughly divided into three types: water depth boundary, flow boundary, and water depth-flow boundary. Because the system has a water connection with the upstream and downstream, the outer boundary conditions of the system should be considered, respectively.

2.4.1.1 Upstream water depth boundary

The upstream water depth generally changes with time, so it can be considered as the function of time t, namely:

$$h_1 = h(t) \tag{34}$$

where h_1 is the upstream water depth.

Transform the aforesaid Formula (34) to an increment form, and then, we obtain:

$$\Delta h_1 = h_1^{i+1} - h_1^* \tag{35}$$

where h_1^{i+1} is the upstream water depth at time (i + 1); h_1^* is the value of the variable at the previous cycle step.

Transform the aforesaid Formula (35) into a form identical to linearized de Saint-Venant system of equations, and then, we obtain:

$$c_1 \Delta h_1 + d_1 \Delta Q_1 = p_1 \tag{36}$$

where c_1 = 1.0, d_1 = 0, and p_1 = $h_1^{i+1} - h_1^*$.

2.4.1.2 Upstream flow boundary

The upstream flow generally changes with time, so it can be considered as the function of time t, namely:

$$Q_1 = Q(t) \tag{37}$$

where Q_1 is the upstream flow.

Transform the aforesaid Formula (34) to an increment form, and then, we obtain:

$$\Delta Q_1 = Q_1^{i+1} - Q_1^* \tag{38}$$

where Q_1^{i+1} is the upstream flow at time (i + 1) and Q_1^* is the value of the variable at the previous cycle step.

Transform the aforesaid Formula (38) into a form identical to linearized de Saint-Venant system of equations, and then, we obtain:

$$c_1 \Delta h_1 + d_1 \Delta Q_1 = p_1 \tag{39}$$

where c_1 = 0, d_1 = 1.0, and p_1 =$Q_1^{i+1} - Q_1^*$.

2.4.1.3 Downstream water depth boundary

Like the upstream water depth boundary, the downstream water depth generally changes with time too, so it can be considered as the function of time t, namely:

$$h_n = h(t) \tag{40}$$

where h_n is the downstream water depth.

Transform the aforesaid Formula (40) to an increment form, and then, we obtain:

$$\Delta h_n = h_n^{i+1} - h_n^* \tag{41}$$

where h_n^{i+1} is the downstream water depth at time (i + 1); h_n^* is the value of the variable at the previous cycle step.

Transform the aforesaid Formula (41) into a form identical to linearized de Saint-Venant system of equations, and then, we obtain:

$$a_n \Delta h_n + b_n \Delta Q_n = p_n \tag{42}$$

where a_n =1.0, b_n = 0, and $p_n = h_n^{i+1} - h_n^*$.

2.4.1.4 Downstream flow boundary

Like the upstream flow boundary, the downstream flow generally changes with time too, so it can be considered as the function of time t, namely:

$$Q_n = Q(t) \tag{43}$$

where Q_n is the upstream flow.

Transform the aforesaid Formula (43) to an increment form, and then, we obtain:

$$\Delta Q_n = Q_n^{i+1} - Q_n^* \tag{44}$$

where Q_n^{i+1} is the upstream flow at time (i + 1); Q_n^* is the value of the variable at the previous cycle step.

Transform the aforesaid Formula (44) into a form identical to linearized de Saint-Venant system of equations, and then, we obtain:

$$a_n \Delta h_n + b_n \Delta Q_n = p_n \tag{45}$$

where a_n = 0, b_n = 1.0, and $p_n = Q_n^{i+1} - Q_n^*$.

2.4.1.5 Upstream water depth-flow boundary

The upstream water depth and flow are considered conforming to the following functional relation:

$$Q_1 = f(h_1) \tag{46}$$

where h_1 is the upstream water depth and Q_1 is the upstream flow.

Transform the left of the aforesaid formula into an increment form, and make Taylor's series expansion of the right, and then, we obtain:

$$\Delta Q_1 + Q_1^* = f\left(h_1^*\right) + \frac{df}{dh} \Delta h_1 \tag{47}$$

Transform the aforesaid Formula (47) into a form identical to linearized de Saint-Venant system of equations, and then, we obtain:

$$c_1 \Delta h_1 + d_1 \Delta Q_1 = p_1 \tag{48}$$

where $c_1 = -\frac{df}{dh}$ and d_1 = 1.0; and $p_1 = f\left(h_1^*\right) - Q_1^*$.

2.4.1.6 Downstream water depth-flow boundary

Like the upstream treatment, the downstream water depth and flow are also considered conforming to the following functional relation:

$$Q_n = f(h_n) \tag{49}$$

where h_n is the downstream water depth and Q_n is the downstream flow.
Transform the left of the aforesaid formula into an increment form and make Taylor's series expansion of the right, and then, we obtain:

$$\Delta Q_n + Q_n^* = f\left(h_n^*\right) + \frac{df}{dh}\Delta h_n \tag{50}$$

Transform the aforesaid Formula (50) into a form identical to linearized de Saint-Venant system of equations, and then, we obtain:

$$a_n \Delta h_n + b_n \Delta Q_n = p_n \tag{51}$$

where $a_n = -\frac{df}{dh}$ and b_n = 1.0; and $p_n = f\left(h_n^*\right) - Q_n^*$.

2.4.2 Inner boundary conditions

Because the MRP involves a variety of buildings along the main route, the whole general main canal can be considered to be a channel pool system which connects different hydraulic structures in series, with all check gates used to divide different channel pools. Therefore, when considering inner boundary conditions, we mainly focus on four structures inside the channel system: dividing gate (release sluice), transition section, inverted siphon, and check gate.

2.4.2.1 Boundary conditions of dividing gate

The situation of water flow at the dividing gate (**Figure 2**) is described by using the water balance equation and the energy conservation equation:

$$Q_j = Q_{j+1} + Q_f \tag{52}$$

$$h_j + Z_j + \frac{1}{2g}\left(\frac{Q_j}{A_j}\right)^2 = h_{j+1} + Z_{j+1} + \frac{1}{2g}\left(\frac{Q_{j+1}}{A_{j+1}}\right)^2 \tag{53}$$

where j and j + 1 are the upstream and downstream cross section numbers of the dividing gate, respectively; Q_j and Q_{j+1} are the upstream and downstream cross

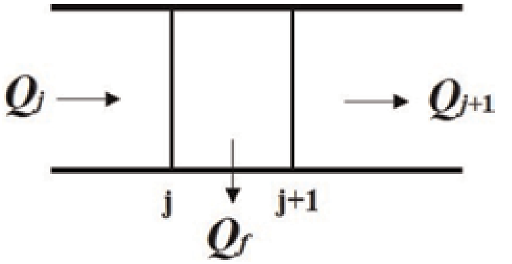

Figure 2.
Schematic diagram of dividing gate.

section flows, respectively; Q_f is the divided water flow at the dividing gate; Z_j and Z_{j+1} are the elevation of the channel bottom of upstream and downstream cross sections at the dividing gate.

After linearization of the aforesaid formula, we obtain:

$$\Delta Q_j - \Delta Q_{j+1} = Q_{j+1}^* - Q_j^* + Q_f \tag{54}$$

$$\left(1 - \frac{\left(Q_j^*\right)^2 B_j^*}{g\left(A_j^*\right)^3}\right)\Delta h_j + \frac{Q_j^*}{g\left(A_j^*\right)^2}\Delta Q_j - \left(1 - \frac{\left(Q_{j+1}^*\right)^2 B_{j+1}^*}{g\left(A_{j+1}^*\right)^3}\right)\Delta h_{j+1} - \frac{Q_{j+1}^*}{g\left(A_{j+1}^*\right)^2}\Delta Q_{j+1}$$
$$= -h_j^* - Z_j - \frac{1}{2g}\left(\frac{Q_j^*}{A_j^*}\right)^2 + h_{j+1}^* + Z_{j+1} + \frac{1}{2g}\left(\frac{Q_{j+1}^*}{A_{j+1}^*}\right)^2 \tag{55}$$

Transform them into the form of Formulas (17) and (28), and all respective factors are as follows:

$$a_j = 0, b_j = 1.0, c_j = 0, d_j = -1.0, p_j = Q_{j+1}^* - Q_j^* + Q_f. \tag{56}$$

$$a_{j+1} = \left(1 - \frac{\left(Q_j^*\right)^2 B_j^*}{g\left(A_j^*\right)^3}\right), b_{j+1} = \frac{Q_j^*}{g\left(A_j^*\right)^2}, c_{j+1} = -\left(1 - \frac{\left(Q_{j+1}^*\right)^2 B_{j+1}^*}{g\left(A_{j+1}^*\right)^3}\right),$$
$$d_{j+1} = -\frac{Q_{j+1}^*}{g\left(A_{j+1}^*\right)^2} \tag{57}$$

$$p_{j+1} = -h_j^* - Z_j - \frac{1}{2g}\left(\frac{Q_j^*}{A_j^*}\right)^2 + h_{j+1}^* + Z_{j+1} + \frac{1}{2g}\left(\frac{Q_{j+1}^*}{A_{j+1}^*}\right)^2. \tag{58}$$

2.4.2.2 Boundary conditions of transition section

Like the treatment of the dividing gate, the water balance equation and energy conservation equation used to describe hydraulic characteristics at the transition section (**Figure 3**) are shown below:

$$Q_j = Q_{j+1} \tag{59}$$

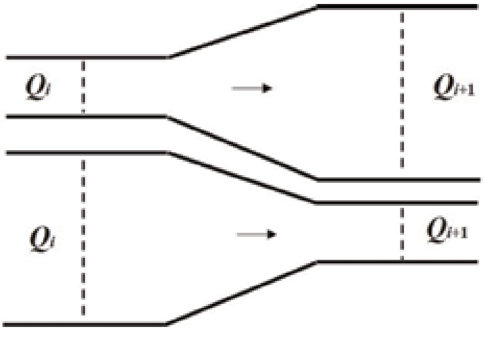

Figure 3.
Schematic diagram of transition section.

$$h_j + Z_j + \frac{1}{2g}\left(\frac{Q_j}{A_j}\right)^2 = h_{j+1} + Z_{j+1} + \frac{1}{2g}\left(\frac{Q_{j+1}}{A_{j+1}}\right)^2$$
$$+\zeta\frac{1}{2g}\left|\left(\left(\frac{Q_j}{A_j}\right)^2 - \left(\frac{Q_{j+1}}{A_{j+1}}\right)^2\right)\right| + \xi\frac{1}{2g}\left(\frac{Q_j}{A_j}\right)^2 \tag{60}$$

where ζ is a local loss coefficient and ξ is an other loss coefficient. However, when the hydraulic characteristics of the transition section are considered, the transition section should be divided into two cases: converge section and divergence section, which should be treated differently.

After linearization of the aforesaid formula, we obtain:

$$\Delta Q_j - \Delta Q_{j+1} = Q^*_{j+1} - Q^*_j \tag{61}$$

$$\left(1-(1-\zeta)\frac{\left(Q^*_j\right)^2 B^*_j}{g\left(A^*_j\right)^3}\right)\Delta h_j + \frac{(1-\zeta)Q^*_j}{g\left(A^*_j\right)^2}\Delta Q_j - \left(\frac{(1-\zeta)Q^*_{j+1}}{g\left(A^*_{j+1}\right)^2} + \frac{\xi|Q^*_{j+1}|}{g\left(A^*_{j+1}\right)^2}\right)\Delta Q_{j+1}$$
$$-\left(1-(1-\zeta)\frac{\left(Q^*_{j+1}\right)^2 B^*_{j+1}}{g\left(A^*_{j+1}\right)^3} - \frac{\xi Q^*_{j+1}|Q^*_{j+1}|B^*_{j+1}}{g\left(A^*_{j+1}\right)^3}\right)\Delta h_{j+1}$$
$$= -h^*_j - Z_j - \frac{1-\zeta}{2g}\left(\frac{Q^*_j}{A^*_j}\right)^2 + h^*_{j+1} + Z_{j+1} + \frac{1-\zeta}{2g}\left(\frac{Q^*_{j+1}}{A^*_{j+1}}\right)^2 + \frac{\xi Q^*_{j+1}|Q^*_{j+1}|}{2g\left(A^*_{j+1}\right)^2} \tag{62}$$

$$\left(1-(1+\zeta)\frac{\left(Q^*_j\right)^2 B^*_j}{g\left(A^*_j\right)^3}\right)\Delta h_j + \frac{(1+\zeta)Q^*_j}{g\left(A^*_j\right)^2}\Delta Q_j - \left(\frac{(1+\zeta)Q^*_{j+1}}{g\left(A^*_{j+1}\right)^2} + \frac{\xi|Q^*_{j+1}|}{g\left(A^*_{j+1}\right)^2}\right)\Delta Q_{j+1}$$
$$-\left(1-(1+\zeta)\frac{\left(Q^*_{j+1}\right)^2 B^*_{j+1}}{g\left(A^*_{j+1}\right)^3} - \frac{\xi Q^*_{j+1}|Q^*_{j+1}|B^*_{j+1}}{g\left(A^*_{j+1}\right)^3}\right)\Delta h_{j+1}$$
$$= -h^*_j - Z_j - \frac{1+\zeta}{2g}\left(\frac{Q^*_j}{A^*_j}\right)^2 + h^*_{j+1} + Z_{j+1} + \frac{1+\zeta}{2g}\left(\frac{Q^*_{j+1}}{A^*_{j+1}}\right)^2 + \frac{\xi Q^*_{j+1}|Q^*_{j+1}|}{2g\left(A^*_{j+1}\right)^2} \tag{63}$$

Transform them into the form of Formulas (17) and (28), and all respective factors are as follows:

$$a_j = 0, b_j = 1.0, c_j = 0, d_j = -1.0, p_j = Q^*_{j+1} - Q^*_j. \tag{64}$$

The coefficient for the converge section:

$$a_{j+1}=\left(1-(1-\zeta)\frac{\left(Q_j^*\right)^2B_j^*}{g\left(A_j^*\right)^3}\right),\ b_{j+1}=\frac{(1-\zeta)Q_j^*}{g\left(A_j^*\right)^2},\ c_{j+1}=-\left(\frac{(1-\zeta)Q_{j+1}^*}{g\left(A_{j+1}^*\right)^2}+\frac{\xi|Q_{j+1}^*|}{g\left(A_{j+1}^*\right)^2}\right),$$
$$d_{j+1}=-\left(1-(1-\zeta)\frac{\left(Q_{j+1}^*\right)^2B_{j+1}^*}{g\left(A_{j+1}^*\right)^3}-\frac{\xi Q_{j+1}^*|Q_{j+1}^*|B_{j+1}^*}{g\left(A_{j+1}^*\right)^3}\right) \tag{65}$$

$$p_{j+1}=-h_j^*-Z_j-\frac{1-\zeta}{2g}\left(\frac{Q_j^*}{A_j^*}\right)^2+h_{j+1}^*+Z_{j+1}+\frac{1-\zeta}{2g}\left(\frac{Q_{j+1}^*}{A_{j+1}^*}\right)^2+\frac{\xi Q_{j+1}^*|Q_{j+1}^*|}{2g\left(A_{j+1}^*\right)^2}. \tag{66}$$

The coefficient for the divergence section:

$$a_{j+1}=\left(1-(1+\zeta)\frac{\left(Q_j^*\right)^2B_j^*}{g\left(A_j^*\right)^3}\right),\ b_{j+1}=\frac{(1+\zeta)Q_j^*}{g\left(A_j^*\right)^2},\ c_{j+1}=-\left(\frac{(1+\zeta)Q_{j+1}^*}{g\left(A_{j+1}^*\right)^2}+\frac{\xi|Q_{j+1}^*|}{g\left(A_{j+1}^*\right)^2}\right),$$
$$d_{j+1}=-\left(1-(1+\zeta)\frac{\left(Q_{j+1}^*\right)^2B_{j+1}^*}{g\left(A_{j+1}^*\right)^3}-\frac{\xi Q_{j+1}^*|Q_{j+1}^*|B_{j+1}^*}{g\left(A_{j+1}^*\right)^3}\right) \tag{67}$$

$$p_{j+1}=-h_j^*-Z_j-\frac{1+\zeta}{2g}\left(\frac{Q_j^*}{A_j^*}\right)^2+h_{j+1}^*+Z_{j+1}+\frac{1+\zeta}{2g}\left(\frac{Q_{j+1}^*}{A_{j+1}^*}\right)^2+\frac{\xi Q_{j+1}^*|Q_{j+1}^*|}{2g\left(A_{j+1}^*\right)^2}. \tag{68}$$

2.4.2.3 Boundary conditions of inverted siphon

When describing the hydraulic characteristics at the inverted siphon (**Figure 4**), we consider that the water head difference at the inlet and outlet consists of the local head loss and middle frictional head loss; then, the water balance equation and energy conservation equation used to describe the hydraulic characteristics are indicated below:

$$Q_j=Q_{j+1} \tag{69}$$

$$\begin{aligned}h_j+Z_j+\frac{1}{2g}\left(\frac{Q_j}{A_j}\right)^2&=h_{j+1}+Z_{j+1}+\frac{1}{2g}\left(\frac{Q_{j+1}}{A_{j+1}}\right)^2\\&+\zeta_1\frac{1}{2g}\left(\frac{Q_j}{A_j}\right)^2+\zeta_2\frac{1}{2g}\left(\frac{Q_{j+1}}{A_{j+1}}\right)^2+\frac{Q_{j+1}|Q_{j+1}|}{\left(K_{j+1}\right)^2}L\end{aligned} \tag{70}$$

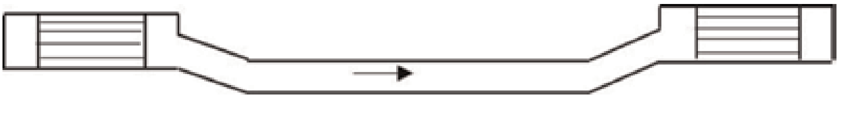

Figure 4.
Schematic diagram of inverted siphon.

where ζ_1 and ζ_2 are the loss coefficient at the inlet and outlet; and L is the middle length of the inverted siphon.

After linearization of the aforesaid formula, we obtain:

$$\Delta Q_j - \Delta Q_{j+1} = Q_{j+1}^* - Q_j^* \tag{71}$$

$$\begin{aligned}&\left(1-(1-\zeta_1)\frac{\left(Q_j^*\right)^2 B_j^*}{g\left(A_j^*\right)^3}\right)\Delta h_j - \left(\frac{(1+\zeta_2)Q_{j+1}^*}{g\left(A_{j+1}^*\right)^2} + \frac{2|Q_{j+1}^*|}{\left(K_{j+1}^*\right)^2}L\right)\Delta Q_{j+1}\\ &-\left(1-(1+\zeta_2)\frac{\left(Q_{j+1}^*\right)^2 B_{j+1}^*}{g\left(A_{j+1}^*\right)^3}\right)\Delta h_{j+1} + \frac{(1-\zeta_1)Q_j^*}{g\left(A_j^*\right)^2}\Delta Q_j\\ &= -h_j^* - Z_j - \frac{1-\zeta_1}{2g}\left(\frac{Q_j^*}{A_j^*}\right)^2 + h_{j+1}^* + Z_{j+1} + \frac{1+\zeta_2}{2g}\left(\frac{Q_{j+1}^*}{A_{j+1}^*}\right)^2 + \frac{Q_{j+1}^*|Q_{j+1}^*|}{\left(K_{j+1}^*\right)^2}L\end{aligned} \tag{72}$$

where $K = \frac{A}{n}R^{\frac{2}{3}}$.

Transform them into the form of Formulas (17) and (28), and all respective factors are as follows:

$$a_j = 0, b_j = 1.0, c_j = 0, d_j = -1.0, p_j = Q_{j+1}^* - Q_j^*. \tag{73}$$

$$\begin{aligned}&a_{j+1} = \left(1-(1-\zeta_1)\frac{\left(Q_j^*\right)^2 B_j^*}{g\left(A_j^*\right)^3}\right), b_{j+1} = \frac{(1-\zeta_1)Q_j^*}{g\left(A_j^*\right)^2}, c_{j+1} = -\left(1-(1+\zeta_2)\frac{\left(Q_{j+1}^*\right)^2 B_{j+1}^*}{g\left(A_{j+1}^*\right)^3}\right),\\ &d_{j+1} = -\left(1-(1+\zeta_2)\frac{\left(Q_{j+1}^*\right)^2 B_{j+1}^*}{g\left(A_{j+1}^*\right)^3}\right)\end{aligned} \tag{74}$$

$$p_{j+1} = -h_j^* - Z_j - \frac{1-\zeta_1}{2g}\left(\frac{Q_j^*}{A_j^*}\right)^2 + h_{j+1}^* + Z_{j+1} + \frac{1+\zeta_2}{2g}\left(\frac{Q_{j+1}^*}{A_{j+1}^*}\right)^2 + \frac{Q_{j+1}^*|Q_{j+1}^*|}{\left(K_{j+1}^*\right)^2}L \tag{75}$$

2.4.2.4 Boundary conditions of check gate

The equations used to describe the hydraulic characteristics of water flowing through the check gate (**Figure 5**) are shown below:

$$Q_j = Q_{j+1} \tag{76}$$

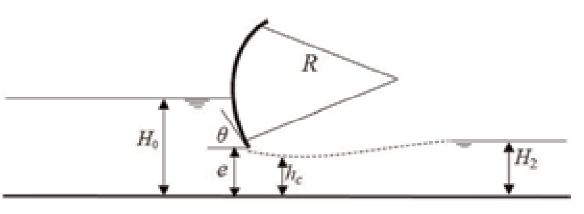

Figure 5.
Schematic diagram of arc check gate.

$$Q_j = \sigma_c \sigma_s mbf\left(H_j, H_{j+1}, e\right) \tag{77}$$

where σ_c is the side-contract coefficient; σ_s is the submergence coefficient; m is the flow coefficient; b is the width of sluice; H_j and H_{j+1} are the downstream water levels; e is the opening of sluice.

Transform Formulas (76) and (77) to an increment form, and then, we obtain:

$$\Delta Q_j - \Delta Q_{j+1} = Q_{j+1}^* - Q_j^* \tag{78}$$

$$\sigma_c \sigma_s mb \frac{\partial f^*}{\partial H_j^*} \Delta H_j - \Delta Q_j + \sigma_c \sigma_s mb \frac{\partial f^*}{\partial H_{j+1}^*} \Delta H_{j+1} = Q_j^* - \sigma_c \sigma_s mbf^* \tag{79}$$

where $f^* = \sigma_c \sigma_s mbf\left(H_j^*, H_{j+1}^*, e\right)$.

Transform them into the form of Formulas (17) and (28), and all respective factors are as follows:

$$a_j = 0, b_j = 1.0, c_j = 0, d_j = -1.0, p_j = Q_{j+1}^* - Q_j^*. \tag{80}$$

$$a_{j+1} = \sigma_c \sigma_s mb \frac{\partial f^*}{\partial H_j^*}, b_{j+1} = -1, c_{j+1} = \sigma_c \sigma_s mb \frac{\partial f^*}{\partial H_{j+1}^*}, d_{j+1} = 0 \tag{81}$$

$$p_{j+1} = Q_j^* - \sigma_c \sigma_s mbf^* \tag{82}$$

2.5 Equation solution

Assuming that a channel has n cross sections; each of which has two variables—flow and water level. That is to say, it has totally 2n variables. The channel is divided by n cross sections into n−1 channel section. According to the de Saint-Venant system of equations, two equations can be set up for each channel section, and totally 2(n−1) equations can be obtained; and after combining them with upstream and downstream boundary conditions, totally 2n equations can be obtained, forming a closed system of equations. The coefficient of each equation in the system of equations can be calculated by an initial value and a previous iterative value; therefore, the system of equations is a constant coefficient linear system of equations which can be transformed into a matrix form.

Formulas (17) and (28) are transformed into the following forms:

$$a_{2i}\Delta h_i + b_{2i}\Delta Q_i + c_{2i}\Delta h_{i+1} + d_{2i}\Delta Q_{i+1} = D_{2i} \tag{83}$$

$$a_{i+1}\Delta h_i + b_{2i+1}\Delta Q_i + c_{2i+1}\Delta h_{i+1} + d_{2i}\Delta Q_{i+1} = D_{2i+1} \tag{84}$$

where all coefficients correspond to Formulas (17) and (28); i = 0, 1, 2, 3... n−1, n, the number of each channel section; i = 0 and i = n stand for the upstream and downstream boundary conditions, respectively. Then, the amended system of equations can be transformed into the following matrix form:

$$AX = D \tag{85}$$

where

$$A = \begin{Bmatrix} c_1 & d_1 & & & & & & \\ a_2 & b_2 & c_2 & d_2 & & & & \\ a_3 & b_3 & c_3 & d_3 & & & & \\ & & a_4 & b_4 & c_4 & d_4 & & \\ & & a_5 & b_5 & c_5 & d_5 & & \\ & & \vdots & \vdots & \vdots & \vdots & & \\ & & & & a_{2n-2} & b_{2n-2} & c_{2n-2} & d_{2n-2} \\ & & & & a_{2n-1} & b_{2n-1} & c_{2n-1} & d_{2n-1} \\ & & & & & & a_{2n} & b_{2n} \end{Bmatrix}, X = \begin{bmatrix} \Delta h_1 \\ \Delta Q_1 \\ \Delta h_2 \\ \Delta Q_2 \\ \Delta h_3 \\ \vdots \\ \Delta Q_{n-1} \\ \Delta h_n \\ \Delta Q_n \end{bmatrix}, D = \begin{bmatrix} D_0 \\ D_1 \\ D_2 \\ D_3 \\ D_4 \\ \vdots \\ D_{2n-2} \\ D_{2n-1} \\ D_{2n} \end{bmatrix}. \tag{86}$$

After such transformation, the system of equations can be solved.

When calculating the water depth and flow of each cross section of a channel with a known time t_0, the following process can be used to determine the water depth and flow at time $t_0 + \Delta t$:

1. assume the initial value of the water depth and flow of each cross section of the channel at time $t_0 + \Delta t$;

2. calculate the value of all elements in matrices A and D;

3. solve equation $AX = D$ and obtain X; and

4. judge that all elements in X meet $|x_i| \le \varepsilon$ (i = 0, 1, 2 … 2n). If all elements meet the conditions, the obtained result is the solution at time $t_0 + \Delta t$; if not, use the obtained value as a new iterative value and repeat steps (2)–(4).

3. Water quality model

3.1 Control equations

When the 1-D water quality of the main canal of the MRP is described by reference to a QUAL-II comprehensive water quality model, the following equilibrium-dispersion mass equation [10]:

$$\frac{\partial AC}{\partial t} = \frac{\partial}{\partial x}\left(EA\frac{\partial C}{\partial x}\right) - \frac{\partial QC}{\partial x} + AS \tag{87}$$

where C is the average concentration of pollutant cross section; A is the cross section area; Q is the average flow of cross section; E is the longitudinal dispersion coefficient; S is the source and drain items.

3.2 Correlation between all water quality variables

When the water quality model for the main route project of the MRP is established according to the actual situation and needs of the MRP and by reference to the QUAL-II comprehensive water quality model [10], the following nine types of water quality variables are considered: (1) dissolved oxygen (DO); (2) ammonia nitrogen; (3) nitrite nitrogen; (4) nitrate nitrogen; (5) biochemical oxygen demand (BOD); (6) chlorophyll-α; (7) dissoluble phosphorus; (8) a kind of supposed degradable substance; and (9) a kind of supposed nondegradable substance. One or more water quality variables may be chosen from the model for simulation according to needs. The relation between all variables is shown in **Figure 6** given below.

The arrows on the figure are used to show the relation between seven types of water quality variables except for supposed degradable and nondegradable substances. The interaction relation between all water quality variables centers around the content of dissolved oxygen (DO) in water body; a series of chemical reactions occur with physical processes including sediment and the like at the same time. The meaning of all processes is as follows: (1) reoxygenation by water body; (2) consumption of oxygen by sediment; (3) carbonized BOD (CBOD); (4) production of oxygen by organisms including algae and the like via photosynthesis; (5) consumption of oxygen during the process of ammonia-nitrogen oxidation; (6) consumption of oxygen during the process of nitrous acid nitrogen oxidation; (7) precipitation of CBOD; (8) absorption of nitrate nitrogen by organisms including algae and the like; (9) production of dissoluble phosphorus by organisms including algae and the like via respiration action; (10) absorption of dissoluble phosphorus by organisms including algae and the like; (11) death and sediment of organisms including algae and the like; (12) production of ammonia nitrogen by organisms including algae and the like via respiration action; (13) release of ammonia nitrogen by sediment; (14) conversion of ammonia nitrogen via oxidation into nitrite nitrogen; (15) conversion of nitrite nitrogen via oxidation into nitrate nitrogen; and (16) release of dissoluble phosphorus by sediment.

The general main canal is an artificial open channel, the MRP sets high requirements for water quality, and there is no sediment in the canal. Therefore, the release of water quality variables (13 and 16) by sediment and the process of oxygen consumption by sediment (2) in the figure above are not considered when the water quality model is established for the general main canal.

For the aforesaid nine types of water quality variables, they have the same form of transport equations and the difference only in the item of growth or attenuation ($\frac{dC}{dt}$) of chemical reaction; then, for each different water quality variable, its item of growth or attenuation of chemical reaction is shown below:

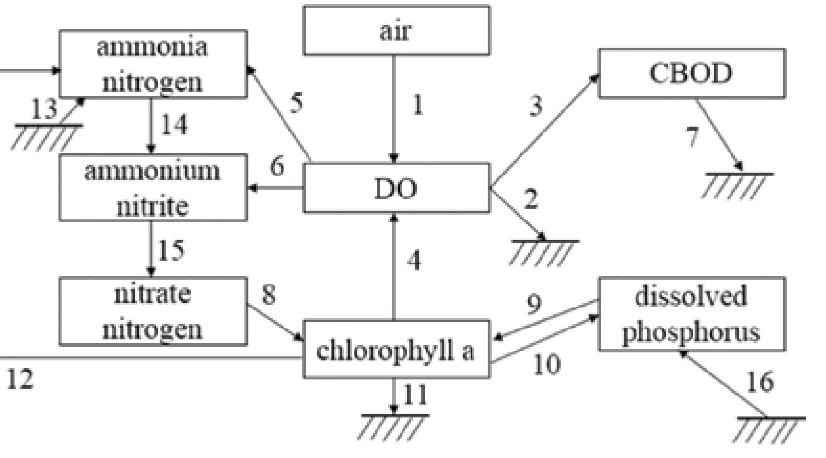

Figure 6.
Diagram of the relation between all water quality variables.

3.2.1 Chlorophyll-α

The reaction process of chlorophyll-α is indicated as follows:

$$\frac{dC_A}{dt} = \mu C_A - \rho_A C_A - \frac{\sigma_1}{H} C_A \tag{88}$$

where μ is algae growth rate; ρ_A is the algae respiratory rate; σ_1 is the algae settling velocity; and H is the average water depth.

3.2.2 Nitrogen cycle

Considering that there are normally three different forms of nitrogen in water body: ammonia nitrogen, nitrite nitrogen, and nitrate nitrogen. Ammonia nitrogen can be oxidized to nitrite nitrogen and transformed to nitrate nitrogen after oxidization. The reaction term of these three substances is shown below, respectively:

Ammonia nitrogen:

$$\frac{dC_{N1}}{dt} = \alpha_1 \rho_A C_A - K_{N1} C_{N1} + \frac{\sigma_3}{A} \tag{89}$$

where α_1 is the ratio of ammonia nitrogen in algae biomass; σ_3 is the release rate of ammonia nitrogen from underwater wildlife which is not considered in the general main canal; K_{N1} is the oxidization rate of ammonia nitrogen; and other symbols have the same meaning as before.

Nitrite nitrogen:

$$\frac{dC_{N2}}{dt} = K_{N1} C_{N1} - K_{N2} C_{N2} \tag{90}$$

where K_{N2} is the oxidization rate and other symbols have the same meaning as before.

Nitrate nitrogen:

$$\frac{dC_{N3}}{dt} = K_{N2} C_{N2} - \alpha_1 \mu C_A \tag{91}$$

The symbols have the same meaning as before.

3.2.3 Phosphorus cycle

Unlike the nitrogen cycle, the phosphorus cycle is not very complex. Therefore, only the correlation between dissoluble phosphorus and algae as well as phosphorus released by sediment is considered. The item of sediment is not considered in the main route project of the MRP. The reaction item is shown below:

$$\frac{dC_p}{dt} = \alpha_2 \rho_A C_A - \alpha_2 \mu C_A + \frac{\sigma_2}{A} \tag{92}$$

where α_2 is the ratio of phosphorus in algae biomass and σ_2 is the release rate of phosphorus from sediment which is not considered in the main route project.

3.2.4 Carbonized BOD (CBOD)

For the change rate of carbonized BOD, refer to the first-order reaction kinetics, and the reaction item is shown below:

$$\frac{dL}{dt} = -K_1L - K_3L \tag{93}$$

where K_1 is the degradation rate of carbonized BOD and K_3 is the rate of carbonized BOD consumption due to precipitation. During the change process of carbonized BOD, dissolved oxygen (DO) is consumed only for degradation than for sediment.

3.2.5 Dissolved oxygen (DO)

The reaction item of DO is shown below:

$$\frac{dO}{dt} = K_2(O_S - O) + (\alpha_3\mu - \alpha_4\rho_A)C_A - K_1L - \alpha_5K_{N1}C_{N1} - \alpha_6K_{N2}C_{N2} - \frac{K_4}{A} \tag{94}$$

where O_S is the saturation concentration of DO; O is the concentration of DO; α_3 is the rate of oxygen production by algae per unit via photosynthesis; α_4 is the rate of oxygen consumption by algae per unit via respiration action; α_5 is the rate of oxygen consumption in the oxidization of ammonia nitrogen per unit; α_6 is the rate of oxygen consumption in the oxidization of nitrite nitrogen per unit; K_2 is the reoxygenation coefficient; K_4 is the coefficient of oxygen consumption by sediment which is not considered in the main route project; and other symbols have the same meaning as before.

3.2.6 Degradable and nondegradable substances

The reaction item of a degradable substance randomly chosen is shown below:

$$\frac{dC_R}{dt} = -K_6C_R \tag{95}$$

where K_6 is the degradation rate of the degradable substance. When it is 0, the reaction equation corresponding to a nondegradable substance randomly chosen is obtained.

3.2.7 External source items of water quality

For the external source items in the model, the following two aspects are mainly considered: (1) the flow of water out of a channel from a dividing gate (release sluice) and (2) input of water quality variable point sources.

$$S_{ext} = qC + S_C \tag{96}$$

where q is the flow of per unit along the channel; C is the concentration of quality variables of water flowing out of the channel; and S_C is the concentration of water quality variables at a point source.

3.2.8 Basic equation for the model

Substitute the reaction item of the aforesaid different water quality variables into the basic control equation for water quality, and then, we obtain the basic equation for the model:

$$\begin{cases} \dfrac{\partial C}{\partial t} + u\dfrac{\partial C}{\partial x} = \dfrac{\partial}{\partial x}\left(E\dfrac{\partial C}{\partial x}\right) + \dfrac{qC}{A} + S_C \\ \dfrac{\partial C_1}{\partial t} + u\dfrac{\partial C_1}{\partial x} = \dfrac{\partial}{\partial x}\left(E\dfrac{\partial C_1}{\partial x}\right) + \dfrac{qC_1}{A} + S_{C_1} - K_6 C_1 \\ \dfrac{\partial L}{\partial t} + u\dfrac{\partial L}{\partial x} = \dfrac{\partial}{\partial x}\left(E\dfrac{\partial L}{\partial x}\right) + \dfrac{qL}{A} + S_L - K_1 L - K_3 L \\ \dfrac{\partial C_A}{\partial t} + u\dfrac{\partial C_A}{\partial x} = \dfrac{\partial}{\partial x}\left(E\dfrac{\partial C_A}{\partial x}\right) + \dfrac{qC_A}{A} + S_{C_A} + \mu C_A - \rho_A C_A - \dfrac{\sigma_1}{H} C_A \\ \dfrac{\partial C_P}{\partial t} + u\dfrac{\partial C_P}{\partial x} = \dfrac{\partial}{\partial x}\left(E\dfrac{\partial C_P}{\partial x}\right) + \dfrac{qC_P}{A} + S_{C_P} + \alpha_2 \rho_A C_A - \alpha_2 \mu C_A \\ \dfrac{\partial C_{N1}}{\partial t} + u\dfrac{\partial C_{N1}}{\partial x} = \dfrac{\partial}{\partial x}\left(E\dfrac{\partial C_{N1}}{\partial x}\right) + \dfrac{qC_{N1}}{A} + S_{C_{N1}} + \alpha_1 \rho_A C_A - K_{N1} C_{N1} \\ \dfrac{\partial C_{N2}}{\partial t} + u\dfrac{\partial C_{N2}}{\partial x} = \dfrac{\partial}{\partial x}\left(E\dfrac{\partial C_{N2}}{\partial x}\right) + \dfrac{qC_{N2}}{A} + S_{C_{N2}} + K_{N1} C_{N1} - K_{N2} C_{N2} \\ \dfrac{\partial C_{N3}}{\partial t} + u\dfrac{\partial C_{N3}}{\partial x} = \dfrac{\partial}{\partial x}\left(E\dfrac{\partial C_{N3}}{\partial x}\right) + \dfrac{qC_{N3}}{A} + S_{C_{N3}} + K_{N2} C_{N2} - \alpha_1 \mu C_A \\ \dfrac{\partial O}{\partial t} + u\dfrac{\partial O}{\partial x} = \dfrac{\partial}{\partial x}\left(E\dfrac{\partial O}{\partial x}\right) + \dfrac{qO}{A} + S_O + K_2(O_S - O) + (\alpha_3 \mu - \alpha_4 \rho_A) C_A - K_1 L \\ -\alpha_5 K_{N1} C_{N1} - \alpha_6 K_{N2} C_{N2} \end{cases} \tag{97}$$

where C, C_1, L, C_A, C_P, C_{N1}, C_{N2}, C_{N3}, O are the concentration of degradable substances, nondegradable substances, BOD, chlorophyll-α, dissoluble phosphorus, ammonia nitrogen, nitrite nitrogen, nitrate nitrogen, and DO, respectively; S_C, S_{C_1}, S_L, S_{C_A}, S_{C_P}, $S_{C_{N1}}$, $S_{C_{N2}}$, $S_{C_{N3}}$, S_O are the concentration of the aforesaid nine types of water quality variables at a point source, respectively; E is the longitudinal dispersion coefficient; and other parameters refer to all subsections above.

3.3 Discrete equation derivation

Without considering the boundary of a channel or river, a water quality equation can be derived by using the Law of Conservation of Mass in an equilibrium area, and then, a discrete equation form can be obtained. In the equilibrium area, substances are fully mixed and the concentration at all points is uniform, as shown below:

The shaded area in **Figure 7** is an equilibrium area. From the figure, the volume of the equilibrium area is:

$$V_j = \frac{1}{4}\left(A_{j-1/2} + A_j\right)\Delta x_{j-1} + \frac{1}{4}\left(A_{j+1/2} + A_j\right)\Delta x_j \tag{98}$$

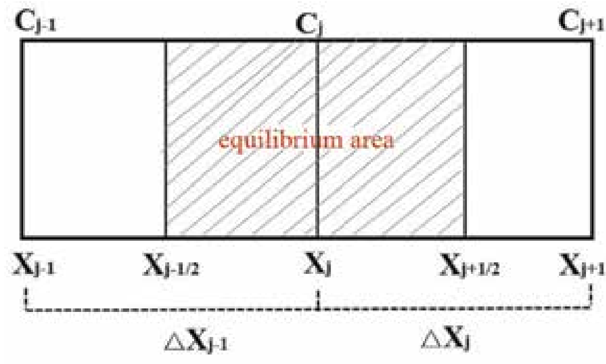

Figure 7.
Schematic diagram of equilibrium area of random nonboundary cross section of a channel.

where V_j is the volume of the equilibrium area at cross section J; $A_{j-\frac{1}{2}}$, $A_{j+\frac{1}{2}}$ are the areas of the cross sections of the inlet and outlet in the equilibrium area, respectively ($A_{j-\frac{1}{2}} = \frac{A_{j-1}+A_j}{2}$, $A_{j+\frac{1}{2}} = \frac{A_j+A_{j+1}}{2}$). Then, the volume of the equilibrium area is:

$$V_j = \frac{1}{8}\left(A_{j-1/2} + 3A_j\right)\Delta x_{j-1} + \frac{1}{8}\left(A_{j+1/2} + 3A_j\right)\Delta x_j \tag{99}$$

Then, the mass variation of water quality variables in the equilibrium area at Δt time interval:

$$\Delta m = V_j^{i+1}C_j^{i+1} - V_j^i C_j^i \tag{100}$$

where i and i + 1 stand for layer i and layer i + 1, respectively.

The change of the concentration of water quality variables in a water body is a comprehensive physical, chemical, and biological processes that are very complicated. The physical process includes advection, molecular diffusion, turbulent diffusion, dispersion, absorption and desorption, sedimentation and resuspension, etc.; and the chemical and biological processes include oxidation, respiration action, photosynthesis, etc. Besides, the concentration of water quality variables in the water body might be affected by external sources such as lateral inflow, sudden point source, etc.

3.3.1 The effect of advection and dispersion on the water quality variables in an equilibrium area

3.3.1.1 Advection

The advection flux F_x of water quality variables at some point in the x direction is:

$$F_x = uC \tag{101}$$

where u is the time average velocity of some point in the x direction and C is the time average concentration of water quality variables at some point.

Then, the mass of water quality variables that passes some cross section at Δt time interval due to advection is:

$$m = AF_x\Delta t = A\overline{u}\overline{C}\Delta t = \overline{Q}\overline{C}\Delta t \tag{102}$$

where the variables with "—" are the average value of the cross section with the superscript omitted below.

Then, consider that the masses of water quality variables that flow into and out of an equilibrium area by means of advection at Δt time interval are, respectively, as follows:

$$m_{11} = Q_{j-\frac{1}{2}} C_{j-\frac{1}{2}} \Delta t \tag{103}$$

$$m_{12} = Q_{j+\frac{1}{2}} C_{j+\frac{1}{2}} \Delta t \tag{104}$$

where $Q_{j-\frac{1}{2}} = \frac{Q_{j-1}+Q_j}{2}$, $Q_{j+\frac{1}{2}} = \frac{Q_j+Q_{j+1}}{2}$; $C_{j-\frac{1}{2}} = \theta C_{j-1} + (1-\theta)C_j$, $C_{j+\frac{1}{2}} = \theta C_j + (1-\theta)C_{j+1}$; θ is the upwind factor $(0 \le \theta \le 1)$. If $Q_{j-\frac{1}{2}} > 0$, $\theta \ge \frac{1}{2}$;

If $Q_{j-\frac{1}{2}} \le 0$, $\theta \le \frac{1}{2}$; if $Q_{j-\frac{1}{2}} > 0$, $\theta = 1$; then this case is called a complete upwind scheme, and the concentration of water quality variables flowing into the equilibrium area under such case is the concentration at an upstream inflow node.

3.3.1.2 Molecular diffusion

Molecular diffusion conforms to Fick's first law:

$$M_m = -E_m \frac{\partial C}{\partial x} \tag{105}$$

where M_m is the molecule diffusive flux at some point in the x direction and E_m is the molecular diffusion coefficient and its range of values generally is $10^{-9} \sim 10^{-8} m^2/s$.

Then, consider that the masses of water quality variables that flow into and out of an equilibrium area by means of molecular diffusion at Δt time interval are, respectively, as follows:

$$m_{21} = -E_{m,j-\frac{1}{2}} A_{j-\frac{1}{2}} \frac{C_j - C_{j-1}}{\Delta x_{j-1}} \Delta t \tag{106}$$

$$m_{22} = -E_{m,j+\frac{1}{2}} A_{j+\frac{1}{2}} \frac{C_{j+1} - C_j}{\Delta x_j} \Delta t \tag{107}$$

where $E_{m,j-\frac{1}{2}}$ and $E_{m,j+\frac{1}{2}}$ are the molecular diffusion coefficients at $x_{j-\frac{1}{2}}$ and $x_{j+\frac{1}{2}}$, respectively; $E_{m,j-\frac{1}{2}} = \frac{E_{m,j-1}+E_{m,j}}{2}$; and $E_{m,j+\frac{1}{2}} = \frac{E_{m,j}+E_{m,j+1}}{2}$.

3.3.1.3 Turbulent diffusion

Like the molecular diffusion, turbulent diffusion also can be expressed by using the form of Fick's first law:

$$M_t = -E_{tx} \frac{\partial C}{\partial x} \tag{108}$$

where M_t is the turbulent diffusion flux at some point in the x direction; E_{tx} is the turbulent diffusion coefficient; for a turbulent flow field whose Reynolds number is about $\mathrm{Re} = 10^4$, the turbulent diffusion coefficient is approximately 3.36×10^{-4} m^2/s.

Then, consider that the masses of water quality variables that flow into and out of an equilibrium area by means of turbulent diffusion at Δt time interval are, respectively, as follows:

$$m_{31} = -E_{\mathrm{tx},j-\frac{1}{2}} A_{j-\frac{1}{2}} \frac{C_j - C_{j-1}}{\Delta x_{j-1}} \Delta t \tag{109}$$

$$m_{32} = -E_{\mathrm{tx},j+\frac{1}{2}} A_{j+\frac{1}{2}} \frac{C_{j+1} - C_j}{\Delta x_j} \Delta t \tag{110}$$

where $E_{\mathrm{tx},j-\frac{1}{2}}$ and $E_{\mathrm{tx},j+\frac{1}{2}}$ are the turbulent diffusion coefficients at $x_{j-\frac{1}{2}}$ and $x_{j+\frac{1}{2}}$, respectively; $E_{\mathrm{tx},j-\frac{1}{2}} = \frac{E_{\mathrm{tx},j-1}+E_{\mathrm{tx},j}}{2}$; and $E_{\mathrm{tx},j+\frac{1}{2}} = \frac{E_{\mathrm{tx},j}+E_{\mathrm{tx},j+1}}{2}$.

3.3.1.4 Dispersion

The expression of dispersion is as follows:

$$M_d = -E_d \frac{\partial C}{\partial x} \tag{111}$$

where M_d is the discrete transport flux at some point in the x direction and E_d is the dispersion coefficient with a range of values of $10 \sim 10^3$ $\mathrm{m^2/s}$.

Then, consider that the masses of pollutants that flow into and out of an equilibrium area by means of dispersion at Δt time interval are, respectively, as follows:

$$m_{41} = -E_{\mathrm{d},j-\frac{1}{2}} A_{j-\frac{1}{2}} \frac{C_j - C_{j-1}}{\Delta x_{j-1}} \Delta t \tag{112}$$

$$m_{42} = -E_{\mathrm{d},j+\frac{1}{2}} A_{j+\frac{1}{2}} \frac{C_{j+1} - C_j}{\Delta x_j} \Delta t \tag{113}$$

where $E_{\mathrm{d},j-\frac{1}{2}}$ and $E_{\mathrm{d},j+\frac{1}{2}}$ are the dispersion coefficients at $x_{j-\frac{1}{2}}$ and $x_{j+\frac{1}{2}}$, respectively; $E_{\mathrm{d},j-\frac{1}{2}} = \frac{E_{\mathrm{d},j-1}+E_{\mathrm{d},j}}{2}$; and $E_{\mathrm{d},j+\frac{1}{2}} = \frac{E_{\mathrm{d},j}+E_{\mathrm{d},j+1}}{2}$.

3.3.1.5 The effect of absorption and desorption and of sedimentation and resuspension on the water quality variables in an equilibrium area

Because the water body in the Danjiangkou Reservoir within the water source region is clear, contains less sand, and has high requirements for water quality, the content of solid matters in the channel such as sediment and the like is extremely low. Therefore, the action of absorption and desorption and of sedimentation and resuspension arising therefrom is omitted during water quality simulation.

3.3.1.6 The effect of advection and dispersion on the water quality variables in an equilibrium area

From the above, we learn that the diffusion coefficient is far greater than the molecular diffusion coefficient and the turbulent diffusion coefficient. Therefore, when a 1-D water quality problem of a channel is solved, advection and dispersion are considered only, with molecular diffusion and turbulent diffusion often omitted. Then, the mass change of water quality variables in an equilibrium area caused by advection and dispersion is:

$$\begin{aligned}\Delta m_1 &= m_{11} - m_{12} + m_{41} - m_{42} \\ &= \Delta t\left\{C_{j-1}\left(\theta Q_{j-1/2} + \frac{E_{d,j-1/2}A_{j-1/2}}{\Delta x_{j-1}}\right)\right. \\ &\quad + C_j\left[(1-\theta)Q_{j-1/2} - \theta Q_{j+1/2} - \frac{E_{d,j-1/2}A_{j-1/2}}{\Delta x_{j-1}} - \frac{E_{d,j+1/2}A_{j+1/2}}{\Delta x_j}\right] \\ &\quad \left.+ C_{j+1}\left[-(1-\theta)Q_{j+1/2} + \frac{E_{d,j+1/2}A_{j+1/2}}{\Delta x_j}\right]\right\}\end{aligned} \tag{114}$$

3.3.2 The effect of a source-sink term on the water quality variables in an equilibrium area

3.3.2.1 The effect of water quality variable degradation and the correlation between water quality variables on the water quality variables in an equilibrium area

According to section 3.2 correlation between all water quality variables conforms to the first-order reaction kinetics equation. Therefore, when the water quality variable degradation and the correlation between water quality variables are solved, zero- and first-order rates are used to characterize, and the coefficients corresponding to them are α_0 and α_1, respectively; at the same time, "+" (plus sign) means reduction of water quality variables, and then, consider the concentration increment of water quality variables in an equilibrium area because of water quality variable degradation and the correlation between water quality variables at Δt time interval is:

$$\Delta m_2 = \Delta t V_j\left(-\alpha_0 - \alpha_1 C_j\right) \tag{115}$$

3.3.2.2 The effect of lateral inflow and a sudden point source on the water quality variables in an equilibrium area

When lateral inflow containing corresponding water quality variables exists along the route of an open channel, or when a sudden point source in the open channel is input, this will bring effect to water quality variables in the open channel. Assuming that the lateral inflow per unit is q, the water quality concentration corresponding thereto is C_q, and the mass of water quality variables input at a sudden point source per unit time is mm, then, consider the concentration change of water quality variables in an equilibrium area owning to lateral inflow or the sudden point source at Δt time interval is:

$$\Delta m_3 = \Delta t C_q q\left(\frac{\Delta x_{j-1}}{2} + \frac{\Delta x_j}{2}\right) + mm\Delta t \tag{116}$$

3.3.3 Final form of discrete equation

Considering the conservation of mass in an equilibrium area, the mass change of water quality variables in an equilibrium area at Δt time interval is equal to the sum of the mass of water quality variables flowing into and out of the equilibrium area at such time interval.

$$\Delta m = \Delta m_1 + \Delta m_2 + \Delta m_3 \tag{117}$$

Substitute the formulas above into this formula, and then, we obtain:

$$\begin{aligned} V_j C_j - V_j^i C_j^i = \Delta t \Bigg\{ & C_{j-1}\left(\theta Q_{j-1/2} + \frac{E_{d,j-1/2}A_{j-1/2}}{\Delta x_{j-1}}\right) \\ & + C_j\left[(1-\theta)Q_{j-1/2} - \theta Q_{j+1/2} - \frac{E_{d,j-1/2}A_{j-1/2}}{\Delta x_{j-1}} - \frac{E_{d,j+1/2}A_{j+1/2}}{\Delta x_j}\right] \\ & + C_{j+1}\left[-(1-\theta)Q_{j+1/2} + \frac{E_{d,j+1/2}A_{j+1/2}}{\Delta x_j}\right]\Bigg\} + \Delta t V_j(-\alpha_0 - \alpha_1 C_j) \\ & + \Delta t C_q q\left(\frac{\Delta x_{j-1}}{2} + \frac{\Delta x_j}{2}\right) + mm\Delta t \end{aligned} \tag{118}$$

The items without a superscript are the variable values on layer i + 1.
After arrangement, we obtain:

$$\begin{aligned} & C_{j-1}\left(\theta Q_{j-1/2} + \frac{E_{d,j-1/2}A_{j-1/2}}{\Delta x_{j-1}}\right) + \\ & C_j\left[(1-\theta)Q_{j-1/2} - \theta Q_{j+1/2} - \frac{E_{d,j-1/2}A_{j-1/2}}{\Delta x_{j-1}} - \frac{E_{d,j+1/2}A_{j+1/2}}{\Delta x_j} - \alpha_1 V_j - \frac{V_j}{\Delta t}\right] \\ & + C_{j+1}\left[-(1-\theta)Q_{j+1/2} + \frac{E_{d,j+1/2}A_{j+1/2}}{\Delta x_j}\right] = \\ & -\frac{V_j^i}{\Delta t}C_j^i + \alpha_0 V_j - C_q q\left(\frac{\Delta x_{j-1}}{2} + \frac{\Delta x_j}{2}\right) - mm \end{aligned} \tag{119}$$

By reference to the form of Formulas (17) and (28), the formula above can be transformed to:

$$A_j C_{j-1} + B_j C_j + D_j C_{j+1} = E_j \tag{120}$$

where

$$A_j = \theta Q_{j-1/2} + \frac{E_{d,j-1/2}A_{j-1/2}}{\Delta x_{j-1}} \tag{121}$$

$$B_j = (1-\theta)Q_{j-1/2} - \theta Q_{j+1/2} - \frac{E_{d,j-1/2}A_{j-1/2}}{\Delta x_{j-1}} - \frac{E_{d,j+1/2}A_{j+1/2}}{\Delta x_j} - \alpha_1 V_j - \frac{V_j}{\Delta t} \tag{122}$$

$$D_j = -(1-\theta)Q_{j+1/2} + \frac{E_{d,j+1/2}A_{j+1/2}}{\Delta x_j} \tag{123}$$

$$E_j = -\frac{V_j^i}{\Delta t}C_j^i + \alpha_0 V_j - C_q q\left(\frac{\Delta x_{j-1}}{2} + \frac{\Delta x_j}{2}\right) - mm \tag{124}$$

3.4 Boundary conditions

Like considering boundary conditions during hydrodynamic calculation, boundary conditions also can be divided into external and internal boundary conditions according to the actual situation of the general main canal when water quality is simulated. The external boundary conditions are mainly about the upstream and downstream channel system; while the internal boundary conditions mainly involve the effect of hydraulic structures in the channel system on water quality variables during the operation of such structures. Only the dividing gate is discussed here according to the actual situation of the general main canal.

3.4.1 Generalization of channels of the middle route

As described above, there are multiple types of structures along the route of the MRP, including 64 check gates, 88 dividing gates, release sluices, inverted siphons, aqueducts, culverts, etc., and those structures are combined in series, with a close hydraulic relation among upstream and downstream structures. Therefore, generalization should be performed in combination of the features of all structures along the route.

Among many structures (excluding lateral outflow such as dividing gate, release sluice, and the like) along the route, other structures do not cause the water body in the channel to flow out of the channel, and at the same time, there is almost no possibility of lateral inflow under the MRP. Therefore, other structures (excluding lateral outflow such as dividing gate, release sluice, and the like) can be generalized to channels during the generalization. Then, by reference to the classification of nodes in the QUAL—II model, the MRP totally includes the following types of nodes: (1) source node; (2) normal channel node; (3) the node containing a point source; (4) upstream lateral outflow node; (5) downstream lateral outflow node; and (6) the node at a channel end. The information of generalization is shown in **Figure 8** below:

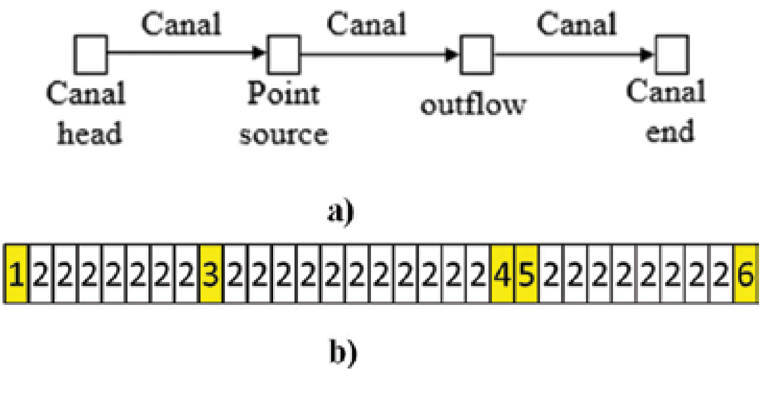

Figure 8.
(a) Channel generalization diagram and (b) node type distribution diagram.

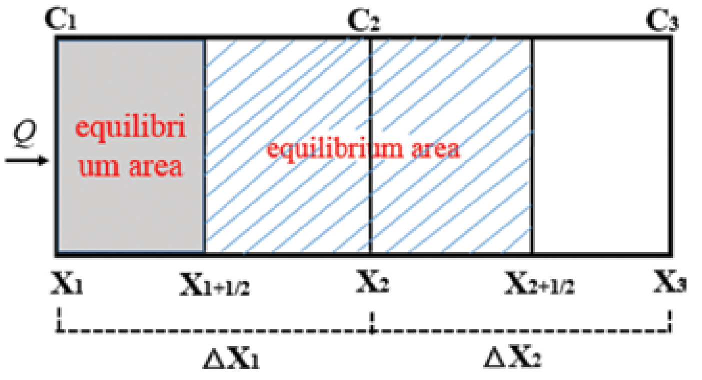

Figure 9.
Schematic diagram for upstream boundary conditions.

3.4.2 Outer boundary conditions

3.4.2.1 Upstream boundary conditions

When upstream boundary conditions are considered, this mainly targets canal head cross sections and their equilibrium area, as indicated by the gray area in **Figure 9** above.

3.4.2.1.1 Upstream boundary conditions with a known upstream concentration

The upstream concentration is considered to change with time, namely:

$$C = C_1(t) \tag{125}$$

From the formula above, we obtain the concentration $C_{1,0}$ at a canal head cross section x_1 at any time; after transforming it into the form of the above formula (120), we obtain:

$$B_1C_1 + D_1C_2 = E_1 \tag{126}$$

where $B_1 = 1.0$, $D_1 = 0$, and $E_1 = C_{1,0}$.

3.4.2.1.2 Upstream boundary conditions with a known upstream water quality variable flux

A known flux means that the mass of water quality variables passing through cross section x_1 per unit time is known, i.e.,

$$F_1 = Q_1C \tag{127}$$

Then, according to the conservation of mass in an equilibrium area and by reference to the process of derivation of the discrete equation in the last section, we know:

$$C_1\left[-\theta Q_{1+1/2} - \frac{E_{d,1+1/2}A_{1+1/2}}{\Delta x_1} - \alpha_1 V_1 - \frac{V_1}{\Delta t}\right] + C_2\left[-(1-\theta)Q_{1+1/2} + \frac{E_{d,1+1/2}A_{1+1/2}}{\Delta x_1}\right]$$
$$= -Q_1C - \frac{V_1^i}{\Delta t}C_1^i + \alpha_0 V_1 - \frac{\Delta x_1}{2}C_q q - mm \tag{128}$$

After transforming it into the form of the above formula (120), we obtain:

$$B_1C_1 + D_1C_2 = E_1 \tag{129}$$

where $B_1 = -\theta Q_{1+1/2} - \frac{E_{d,1+1/2}A_{1+1/2}}{\Delta x_1} - \alpha_1 V_1 - \frac{V_1}{\Delta t}$, $D_1 = -(1-\theta)Q_{1+1/2} + \frac{E_{d,1+1/2}A_{1+1/2}}{\Delta x_1}$, and $E_1 = -Q_1 C - \frac{V_1^i}{\Delta t}C_1^i + \alpha_0 V_1 - \frac{\Delta x_1}{2}C_q q - mm$; all the other parameters have the same meaning as before.

3.4.2.2 Downstream boundary conditions

Like the upstream boundary conditions, when downstream boundary conditions are considered, this mainly targets canal end cross sections and their equilibrium area, as indicated by the gray area in **Figure 10** below.

3.4.2.2.1 Downstream boundary conditions with a known downstream concentration

The downstream concentration is also considered to change with time, namely:

$$C = C_m(t) \tag{130}$$

From the formula above, we obtain the concentration $C_{m,0}$ at a canal end cross section x_m at any time. After transforming it into the form of the above formula (120), we obtain:

$$A_m C_{m-1} + B_m C_m = E_m \tag{131}$$

where $A_m = 0$, $B_m = 1.0$, and $E_m = C_{m,0}$.

3.4.2.2.2 Downstream boundary conditions with a known downstream water quality variable flux

For a canal, the concentration of water quality variables in the water body passing through the canal end cross section is the concentration of external water quality variables. Therefore, the mass of water quality variables passing through cross section x_m per unit time is:

$$F_m = Q_m C_m \tag{132}$$

Then, according to the conservation of mass in an equilibrium area and by reference to the process of derivation of the discrete equation in the last section, we know:

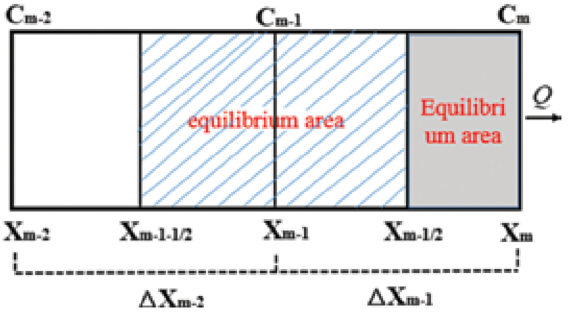

Figure 10.
Schematic diagram for downstream boundary conditions.

$$C_{m-1}\left(\theta Q_{m-1/2}+\frac{E_{d,\,\mathrm{m}-1/2}A_{m-1/2}}{\Delta x_{m-1}}\right)+$$
$$C_m\left[(1-\theta)Q_{m-1/2}-Q_m-\frac{E_{d,\,\mathrm{m}-1/2}A_{m-1/2}}{\Delta x_{m-1}}-\alpha_1 V_m-\frac{V_m}{\Delta t}\right] \quad (133)$$
$$=-\frac{V_m^i}{\Delta t}C_m^i+\alpha_0 V_m-C_q q\frac{\Delta x_{m-1}}{2}-mm$$

After transforming it into the form of the above formula (120), we obtain:

$$A_m C_{m-1}+B_m C_m=E_m \quad (134)$$

where $A_m=\theta Q_{m-1/2}+\frac{E_{d,\,\mathrm{m}-1/2}A_{m-1/2}}{\Delta x_{m-1}}$, $B_m=(1-\theta)Q_{m-1/2}-Q_m-\frac{E_{d,\,\mathrm{m}-1/2}A_{m-1/2}}{\Delta x_{m-1}}-\alpha_1 V_m-\frac{V_m}{\Delta t}$, and $E_m=-\frac{V_m^i}{\Delta t}C_m^i+\alpha_0 V_m-C_q q\frac{\Delta x_{m-1}}{2}-mm$; all the other parameters have the same meaning as before.

3.4.3 Inner boundary conditions

As mentioned above, there are various kinds of structures along the main route project under the MRP. Most of them, however, have less impact on the concentration of water quality variables in the water body. Therefore, when water quality is simulated, the change law of water quality variables at the dividing gate is analyzed only.

The dividing gate along the main route is mainly used to lead the water body out of the channel, dividing it to each water-receiving area. If the water body in the channel gets polluted and the group of polluted water passes through the dividing gate, a certain mass of pollutants will surely pass through the dividing gate and go out of the channel, resulting in the change in the total amount of pollutants in the water body. When treating the pollutants at the dividing gate, we herein think that the dividing gate divides the channel into two sections as shown in **Figure 11** below, where the upstream cross section of the dividing gate is x_j and the downstream cross-section is x_{j+1}.

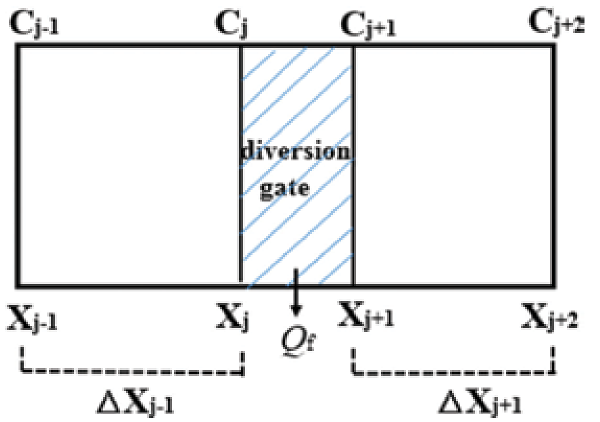

Figure 11.
Schematic diagram for boundary conditions of dividing gate.

3.4.3.1 Upstream of dividing gate

When the upstream channel section of the dividing gate is treated, the treatment method is similar to external boundary conditions of the known downstream water quality variable flux within the external boundary of the channel; then by imitating the above formula (133), we obtain:

$$\begin{aligned}&C_{j-1}\left(\theta Q_{j-1/2}+\frac{E_{d,\mathrm{j}-1/2}A_{j-1/2}}{\Delta x_{j-1}}\right)+\\&C_j\left[(1-\theta)Q_{j-1/2}-Q_j-\frac{E_{d,\mathrm{j}-1/2}A_{j-1/2}}{\Delta x_{j-1}}-\alpha_1 V_j-\frac{V_j}{\Delta t}\right]\\&=-\frac{V_j^i}{\Delta t}C_j^i+\alpha_0 V_j-C_q q\frac{\Delta x_{j-1}}{2}-mm\end{aligned}\tag{135}$$

After transforming it into the form of the above formula (120), we obtain:

$$A_j C_{j-1}+B_j C_j=E_j \tag{136}$$

where $A_j=\theta Q_{j-1/2}+\frac{E_{d,\mathrm{j}-1/2}A_{j-1/2}}{\Delta x_{j-1}}$, $B_j=(1-\theta)Q_{j-1/2}-Q_j-\frac{E_{d,\mathrm{j}-1/2}A_{j-1/2}}{\Delta x_{j-1}}-\alpha_1 V_j-\frac{V_j}{\Delta t}$, and $E_j=-\frac{V_j^i}{\Delta t}C_j^i+\alpha_0 V_j-C_q q\frac{\Delta x_{j-1}}{2}-mm$; all the other parameters have the same meaning as before.

3.4.3.2 Downstream of dividing gate

When the downstream channel section of the dividing gate is treated, this may be considered similar to external boundary conditions of the known upstream water quality variable flux within the external boundary of the channel, and the concentration of water quality variables is C_j; then by imitating the above formula (128), we obtain:

$$\begin{aligned}&C_j Q_{j+1}+C_{j+1}\left(-\theta Q_{j+1+1/2}-\frac{V_{j+1}}{\Delta t}-\frac{E_{d,\mathrm{j}+1+1/2}A_{j+1+1/2}}{\Delta x_{j+1}}-\alpha_1 V_{j+1}\right)+\\&C_{j+2}\left[-(1-\theta)Q_{j+1+1/2}+\frac{E_{d,\mathrm{j}+1+1/2}A_{j+1+1/2}}{\Delta x_{j+1}}\right]=\\&-\frac{V_{j+1}^i}{\Delta t}C_{j+1}^i+\alpha_0 V_{j+1}-\frac{\Delta x_{j+1}}{2}C_q q-mm\end{aligned}\tag{137}$$

After transforming it into the form of the above formula (120), we obtain:

$$A_{j+1}C_j+B_{j+1}C_j+D_{j+1}C_{j+2}=E_{j+1} \tag{138}$$

where $A_{j+1}=Q_{j+1}$, $B_{j+1}=-\theta Q_{j+1+1/2}-\frac{V_{j+1}}{\Delta t}-\frac{E_{d,\mathrm{j}+1+1/2}A_{j+1+1/2}}{\Delta x_{j+1}}-\alpha_1 V_{j+1}$, $D_{j+1}=-(1-\theta)Q_{j+1+1/2}+\frac{E_{d,\mathrm{j}+1+1/2}A_{j+1+1/2}}{\Delta x_{j+1}}$, and $E_{j+1}=-\frac{V_{j+1}^i}{\Delta t}C_{j+1}^i+\alpha_0 V_{j+1}-\frac{\Delta x_{j+1}}{2}C_q q-mm$; all the other parameters have the same meaning as before.

3.5 Determination of model coefficient

Because of a complicated transport and diffusion law of water quality variables in a water body, the form of the equation combining coefficient and concentration is often used to describe such law. This also makes determining the coefficient of the equation becomes a very important step to describe the law.

There are many methods to determine a coefficient. The common ones are as follows—Method 1: set up a mathematical model and assume a group of coefficients; compare the result of numerical simulation and the real test result and make adjustment to the coefficients according to the result of such comparison; repeat this procedure a number of times to determine the values of all the coefficients; Method 2 (a site test method): put a tracer into a simulative water body, trace and monitor the tracer and then calculate the coefficients by using test information; Method 3: estimate the coefficients by using experimental formulas.

Owing to the specificity of the MRP, it is difficult to put a tracer and conduct a site test. However, water flew into the canal not long before and water quality monitoring data are incomplete, so determining parameters in the book is mainly based on research of the same time and experimental formulas.

The determination of a longitudinal dispersion coefficient is mainly based on experimental formulas in the book. The experimental formula proposed by Henry Liu is used:

$$E_d = \gamma \frac{u_* A^2}{H^3} \tag{139}$$

where γ is an empirical coefficient generally having a range of values of 0.5–0.6; u_* is the frictional velocity; $u_* = \sqrt{gHJ}$; J is the hydraulic gradient; A is the cross-section area; and H is the cross section water depth.

The values of other coefficients are indicated in **Table 1** below:

Pollutant	Reaction coefficient	Value	Unit	Range of values
Nondegradable substances	u_0	8	day^{-1}	—
	u_1	0.8	$gm^{-3}\ day^{-1}$	—
Algae	μ	0.075	day^{-1}	1.0–3.0
	ρ_A	0.1	day^{-1}	0.005–0.5
	σ_1	1	$mday^{-1}$	0.153–1.83
Nitrogen	α_1	0.085	gg^{-1}	0.08–0.09
	K_{N1}	0.03	day^{-1}	0.01–0.5
	K_{N2}	1.5	day^{-1}	0.5–2.0
Phosphorus	α_2	0.0135	gg^{-1}	0.012–0.015
CBOD	K_1	0.15	day^{-1}	0.1–0.2
	K_3	0.18	day^{-1}	—
Oxygen	O_S	9.08	gg^{-1}	—
	α_3	1.6	gg^{-1}	1.4–1.8
	α_4	2	gg^{-1}	1.6–2.3
	α_5	3.5	gg^{-1}	3.0–4.0
	α_6	1.07	gg^{-1}	1.0–1.14
	K_2	5	day^{-1}	0–100

Table 1.
Values of parameters corresponding to all water quality variables of the model.

3.6 Model solving

Assuming that the channel totally has m cross sections to be calculated; excluding the cross sections at the head and end of the channel, there are m−2 cross sections; then, for each type of water quality variable, m−2 equations whose form is similar to that of Formula (120) for the whole channel are set up; and two equations for cross sections at the head and end of the channel by using the method to treat boundary conditions are established; and finally, m equations are obtained. By simultaneously solving such m equations, the concentration of corresponding water quality variables can be obtained.

4. Model calibration and verification

4.1 Hydraulic model

A design value of 0.015 is taken as the roughness coefficient of the channel. The local head loss coefficient is adjusted gradually in a reasonable range of values to minimize the error of the simulation and actually measured values of water level. The operation data actually measured at AM 8:00, December 25, 2015 were selected and input the model for calculation; the first 56 check gates were selected, and the actually measured and simulation values of water level of downstream check gate were compared, and the average error is 3.42 cm (**Figure 12**). The results indicate that the model was rational and appropriate for simulating the water surface profile and hydrodynamic change process along channels.

4.2 Water quality model

Because actually measured data cannot be obtained, this model and MIKE11 model [2] are compared with the same calculation example to verify the precision of this model.

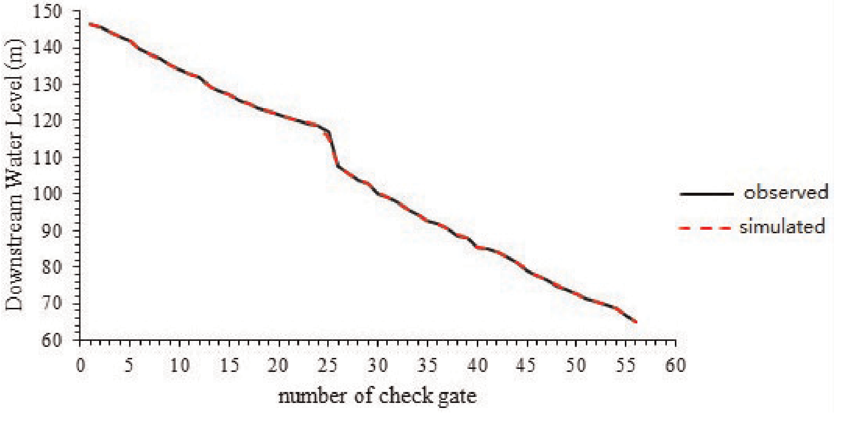

Figure 12.
Comparison between simulation value and actually measured value of water level of downstream check gate.

Assume that there is a channel with a prismatic trapezium cross section, which is 10 km long and has a bottom width of 67.5 m, a slope of 2.5, a bottom slope of about 0.00015, and a roughness coefficient of 0.027; and that there is a reservoir with a constant flow rate of 2000 m^3/s at a place far from the channel upstream. Suppose that 1 ton of soluble but refractory pollutants were leaked suddenly on the right bank of at an upstream cross section of a channel at some time, then try to estimate the process of pollutant concentration change at the downstream outflow cross section after pollution as well as the pollutant concentration distribution in the channel after 0.5, 1, 1.5, and 2 h.

Firstly, calculate the following hydraulic elements of the channel:

the water depth h estimated according to the open channel uniform flow formula $Q = AR^{2/3}\sqrt{i}n^{-1}$ = 11.20 m;
the area of flow section A 1069.60 m;
average velocity of cross section u = 1.87 m/s;
hydraulic radius of cross section R = 8.34 m;
frictional velocity of cross section $u^* = \sqrt{gRi}$ = 0.11 m/s;

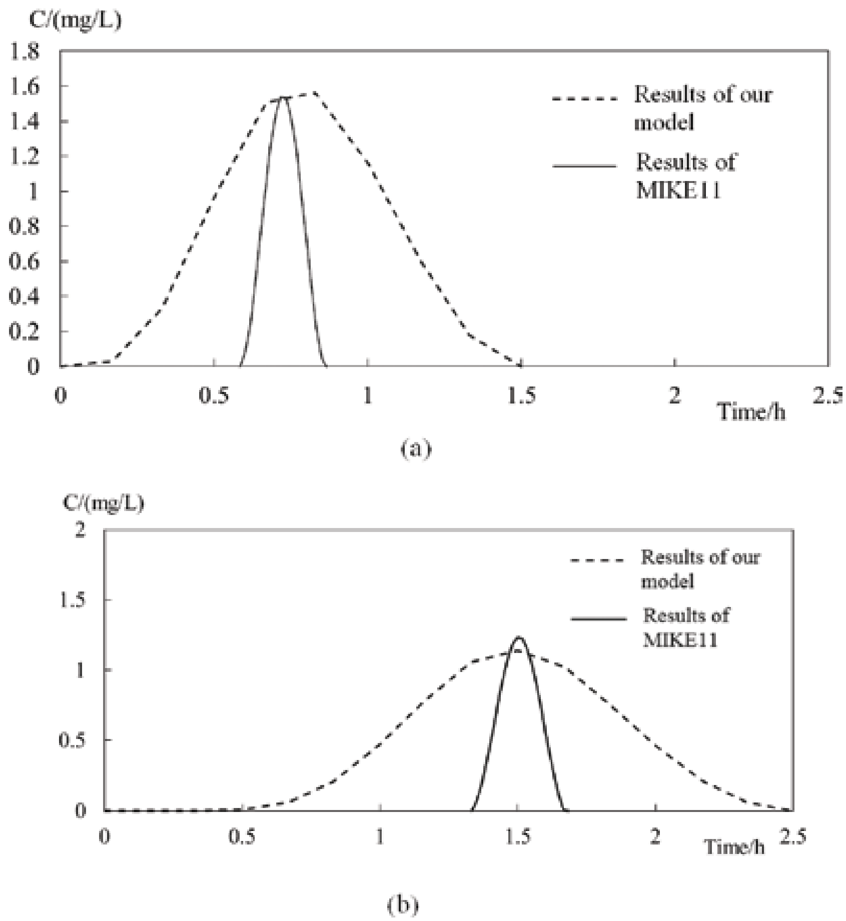

Figure 13.
The process of calculating channel pollutant concentration by using a water quality calculation method. (a) The concentration process at the place 5 km downstream of the channel and (b) the concentration process at the place 10 km downstream of the channel.

estimated channel diffusion coefficient $D_x = D_y = 0.15hu^*$ = 0.18 m^2/s; and estimated longitudinal diffusion (including dispersion) coefficient $D = 6.01hu^*$ = 7.4 m^2/s.

After obtaining the aforesaid hydraulic elements, estimate the pollutant concentration change process at the places (outflow section) 5 and 10 km downstream of the channel (See **Figure 13**), by using the water quality model and MIKE11 AD module mentioned in 2.3.

From the above figure, we see that no matter whether a numerical model under general conditions or MIKE11 is used for simulation, the result of simulation is near to the concentration peak or the time when such peak occurs, well-reflecting transport law of pollutants with water flow in the channel. Moreover, the numerical method applied under this chapter adopts the assumption of equilibrium areas. Under such assumption, if a group of sudden polluted water does not diffuse fully, but gathers in some space, the numerical method will lead to a low simulated concentration; on the other hand, for pollution due to a sudden point source, the treatment technology used in this chapter is on the basis of instantaneous and uniform mixing of concentration and omitted recession process of pollutants in the first channel segment. Therefore, the simulation by the numerical method results in a low-concentration peak.

5. Application of typical project cases

After calibration verification, the aforesaid hydrodynamic water quality model can be applied to contingent operation of the MRP; the application of the hydrodynamic model and water quality model is further verified by taking channel drainage under common operating conditions and sudden water pollution accidents for examples.

5.1 Drainage simulation

In case of a sudden event under the MRP, the release sluice is the main structure for drainage. Therefore, the 36th channel pool is selected for application to analyze the hydraulic response process when the release sluice opens. The basic parameters of the channel pool are as follows:

Assuming that the 36th channel pool undergoes a sudden event, the Anyanghe and Zhanghe check gates are closed within 15 min. Assuming that the canal discharge is 30% of the design discharge, and that the upstream water level of the check gates is the design water level. To analyze the effect arising from different opening speeds of drainage gate, we assume three cases (no opening, opened to 60 m^3/s within 15 min, and opened to 120 m^3/s within 15 min). The time of simulation is 24 h with a step of 10 min.

After simulation of the drainage process of the 36th channel pool by using the hydraulic model, the change processes of the downstream water level of Anyanghe check gate and the upstream water level of Zhanghe check gate are shown in **Figures 14** and **15**, respectively. From these figures, we can see that (1) the downstream water level of Anyanghe check gate goes down first and then up, and the water level fluctuation range becomes small gradually and finally steady without opening the release sluice; while the release sluice is opened, the water level

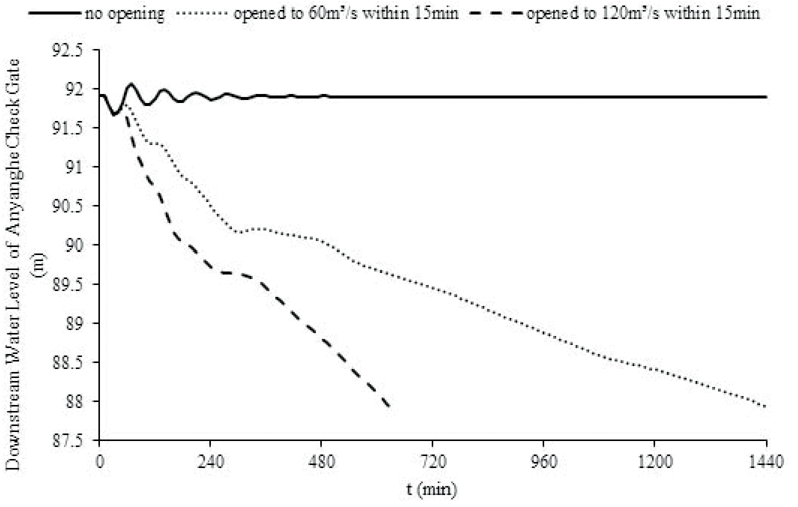

Figure 14.
Change process of downstream water level of Anyanghe check gate during drainage.

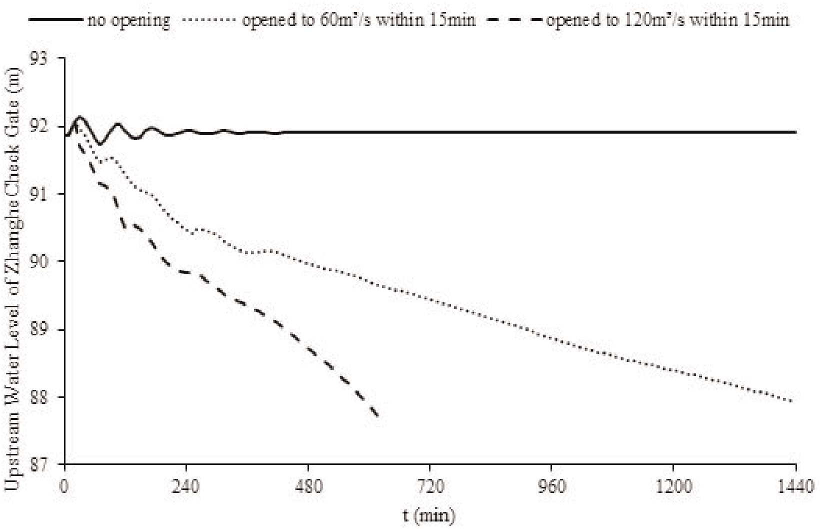

Figure 15.
Change process of upstream water level of Zhanghe check gate during drainage.

fluctuates slightly, but has a trend to become small on the whole until the channel pool is drained; and that (2) the upstream water level of Zhanghe check gate goes up first and then down, and the water level fluctuation range becomes small gradually and finally steady without opening the release sluice; while the release sluice is opened, the water level fluctuates slightly, but has a trend to become small on the whole until the channel pool is drained. Therefore, opening the release sluice makes it easy to reduce the likelihood of high water level in the channel due to fast closing of two check gates in the channel pool.

Channel pool no.	Control structure	Pile no. (km)	Length of channel pool (km)	Elevation of sluice bottom (m)	Design flow (m^3/s)	Design water level (m)
36	Check gate in Anyang River	717.045	14.321	85.60	235	92.67
	Release sluice in Zhanghe River	730.623		85.19	120	/
	Check gate in Zhanghe River	731.366		82.50	235	91.87

'/' means that there is not Design Water Level of Release sluice in Zhanghe River.

Table 2.
Table of basic information on the 36th channel pool.

Item	Content	Quantity
Pollutant category	Nondegradable substance	1
Mode of occurrence	An instantaneous point source	1
The channel pool where an accident occurs	The 36th channel pool	2
Pollutant mass	1, 5, and 10t	3
Place of occurrence	10, 30, 50, 70, and 90% of channel pool length	5
Channel pool flow	30, 50, and 70% of design flow	3
Simulation time/step	24 h/10 min	1

Table 3.
Sudden water pollution accident cases.

5.2 Simulation of sudden water pollution accidents

Since a sudden water pollution event happens generally at some place, assume that water pollution is instantaneously caused by a point source (**Tables 2** and **3**) and that pollutants are nondegradable. In order to analyze the effect caused by different pollutant amount levels, assume that there are three types of pollutant masses (1, 5, and 10 t). In order to analyze the effect of different channel operating conditions, assume that there are three types of channel pool flows (30, 50, and 70% of design flow) but that the upstream water level of each check gate is design flow. In order to analyze the effect of sudden water pollution at different places, assume that there are five accident places (10, 30, 50, 70, and 90% of channel pool length). Then, we obtain 45 cases, which have a simulation time of 24 h with a step of 10 min.

Considering the water supply features of the MRP, the most concern, after a sudden water pollution accident in the channel, is the water quality state at the dividing gate and the upstream water quality state of the lower check gate. This is because, these have a direct effect on the safety of supplied water quality at such dividing gate and in the downstream areas. Three characteristic parameters, pollutant arrival time (T_0), concentration peak ($C_{\max}$), and the occurrence time of concentration peak ($T_{C\max}$), are used to analyze the process of pollutant transport and diffusion. The pollutant arrival time is the time at which the concentration of pollutants at a water quality control point (dividing gate or upstream of lower check gate) exceeds 0.001 mg/L, because toxic effect will occur when the concentration of part of substances (for example, Hg and Cd) in a natural water body is above 0.001 mg/L [11].

In combination of the simulation results of the aforesaid cases, we can find that the concentration of the pollutants at each water quality control point goes up first and then down. These are shown by taking the concentration change process of the pollutants upstream of lower Zhanghe River check gate in case that the 36th channel pool undergoes a sudden water pollution accident at the place 10% of channel pool length and with a flow of 30% of design flow for example (**Figure 16**). The statistics of three characteristic parameters during the concentration change process of pollutants at each water quality control point under each case is made, as shown in **Tables 2–4**; and contrastive analysis of these three parameters is performed.

5.2.1 Pollutant arrival time

When a sudden water pollution accident happens at the same place with the same pollutant mass, a greater flow in the channel pool means that the pollutants arrive at the Zhanghe River check gate at an earlier time. Because the calculation step is 10 min, pollutants under some cases will diffuse fast to the check gate; all the statistical results are 10 min. When a sudden water pollution accident happens at the same place at the same channel pool flow, the greater the pollutant mass is, the earlier the pollutants arrive the check gate. When pollutant mass and channel pool flow are the same, the farther the place of a sudden water pollution accident is from Anyang River check gate, the earlier the pollutants arrive the Zhanghe River check gate.

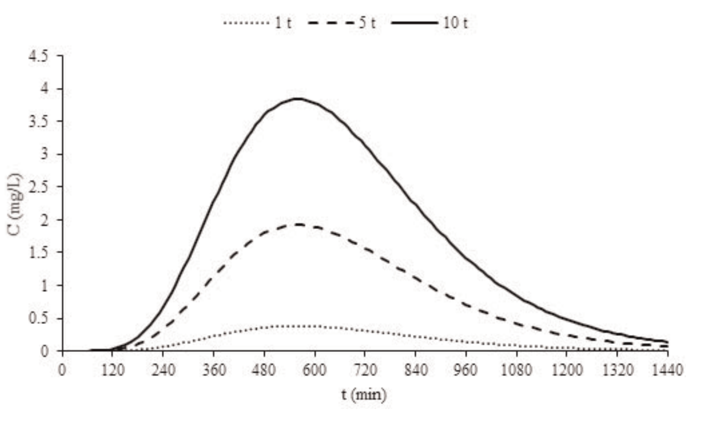

Figure 16.
Concentration change process of the pollutants upstream of Zhanghe River check gate in case that the 36th channel pool undergoes a sudden water pollution accident at the place 10% of channel pool length and with a flow of 30% of design flow.

Place of accident	Pollutant mass (t)	30% of design flow			50% of design flow			70% of design flow		
		T_0 (min)	C_{max} (mg/L)	T_{Cmax} (min)	T_0 (min)	C_{max} (mg/L)	T_{Cmax} (min)	T_0 (min)	C_{max} (mg/L)	T_{Cmax} (min)
0.1 L	1	100	0.3843	560	60	0.3679	380	40	0.3507	260
	5	70	1.9212	560	40	1.8397	380	30	1.7535	260
	10	60	3.8425	560	40	3.6793	380	20	3.507	260
0.3 L	1	60	0.4107	450	40	0.3939	310	20	0.3771	210
	5	40	2.0535	450	20	1.9693	310	10	1.8856	210
	10	30	4.1071	450	20	3.9385	310	10	3.7711	210
0.5 L	1	20	0.4965	280	10	0.475	200	10	0.4577	130
	5	10	2.4823	280	10	2.3752	200	10	2.2887	130
	10	10	4.9647	280	10	4.7504	200	10	4.5773	130
0.7 L	1	10	0.5387	220	10	0.5146	150	10	0.4975	100
	5	10	2.6935	220	10	2.5729	150	10	2.4873	100
	10	10	5.3871	220	10	5.1457	150	10	4.9745	100
0.9 L	1	10	0.8862	110	10	0.8199	80	10	0.7963	50
	5	10	4.4308	110	10	4.0996	80	10	3.9816	50
	10	10	8.8615	110	10	8.1993	80	10	7.9633	50

Table 4.
Characteristic parameters of concentration change process of the pollutants upstream of Zhanghe River check gate in all sudden water pollution accident cases that the 36th channel pool undergoes.

5.2.2 Concentration peak

When a sudden water pollution accident happens at the same place at the same pollutant mass, the greater the channel pool flow is, the smaller the concentration peak at Zhanghe River check gate is. When a sudden water pollution accident happens at the same place at the same channel pool flow, the greater the pollutant mass is, the greater the concentration peak at Zhanghe River check gate is; and the peak has a proportional relation with the pollutant mass. When pollutant mass and channel pool flow are the same, the farther the place of a sudden water pollution accident is from Anyang River check gate, the greater the concentration peak at Zhanghe River check gate is.

5.2.3 Occurrence time of concentration peak

When a sudden water pollution accident happens at the same place at the same pollutant mass, the greater the channel pool flow is, the earlier the concentration peak at Zhanghe River check gate occurs. When a sudden water pollution accident happens at the same place with the same channel pool flow but with different pollutant mass, the concentration peak at Zhanghe River check gate occurs at the same time. When pollutant mass and channel pool flow are the same, the farther the place of a sudden water pollution accident is from

Anyang River check gate, the earlier the concentration peak at Zhanghe River check gate occurs.

6. Summary

For needs predicted via simulation of sudden water pollution accidents in the main canal, this chapter develops the 1-D hydrodynamic water quality coupling simulation model, which can realize the comprehensive simulation of the hydraulic response process under normal operating conditions, emergency operation operating conditions, etc., of the main canal and of nine types of water quality variables. Then, the chapter proves the precision of the hydrodynamic model by using the data actually measured and the precision of the water quality model by using a MIKE11 model. Furthermore, the chapter explains the applicability of the 1-D hydrodynamic water quality coupling simulation model in the operation of sudden water pollution accidents via simulation of channel pool drainage and sudden water pollution accident cases.

However, the water quality variables are not enough, and the 1-D model is less accurate than 2-D or 3-D model. In the future, a hydrodynamic water quality coupling simulation 2-D or 3-D model for the main canal should be developed, and the model can consider other water quality variables such as oil and algae, which can analyze the hydrodynamic and water quality process more precisely.

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Chapter 3

Traceability Technology for Sudden Water Pollution Accidents in Rivers

Weihong Liao, Zhiguo Gan, Jiabiao Wang and Hezhen Zheng

Abstract

The traceability technology for sudden water pollution accidents can be used for fast, accurate identification of a pollution source in the river. A correlation optimization model with the pollution source position and release time as its parameters is established based on hydrodynamic calculation and on the coupling relationship between forward concentration probability density and backward position probability density; and the solution of the model is realized by using a differential evolution algorithm (DEA). A coupled probability density method is to convert the traceability of a sudden water pollution accident into the optimization of two minimum values. This method is simple in principle and easy in solution, realizing the decoupling of parameter of the pollution source. The concept of gradient is introduced to the differential evolution algorithm, improving the efficiency of searching process. The proposed method of traceability was applied to the emergency demonstration project of the SNWDMRP. The results indicate that the model has good efficiency of traceability and high simulation precision and that traceability results have a certain guiding significance to the emergent regulation and control of sudden water pollution events in a river.

Keywords: coupled probability density function, correlation optimization model, parameter decoupling, traceability, emergency, river

1. Introduction

Frequent occurrence of various kinds of water pollution events makes the research of pollutant traceability receive increasing attention, especially for long-distance water diversion projects including the SNWDMRP. The uncertainty of sudden water pollution events increases the difficulty in coping with pollution urgently. Finding out the source of water pollution is a prerequisite for realizing water quality prediction and water body pollution control. Therefore, the first task of urgently coping with sudden water pollution is to determine the source of pollution at the first time after such event, make a reasonable emergency handling plan on the basis of pollution source intensity and the occurrence place and time which have been determined, and meanwhile, provide preconditions for sudden water pollution prediction and early warning [1, 2]. The aforesaid traceability technology infers the position and time of occurrence as well as pollution source

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intensity through research on the transfer and conversion laws of pollutants in a river and on the basis of a monitored pollutant concentration process, realizing the reconstitution of the pollution event and playing an important role in the emergent regulation and control process of sudden water pollution events [3].

2. Basic traceability principle of sudden water pollution

Essentially, both pollutant concentration prediction and pollutant traceability fall into the process of pollutant traceability. To be more precise, the prediction of pollutant concentration distribution can be taken as the forward track process of pollutants, while the traceability of pollutants the process of backward track and traceability, as shown in **Figure 1**.

According to the information on a pollution event, the process of determining the concentration of the pollutant during the event by using the simulation technology in Chapter 2 is forward track of pollutant transfer and whereabouts, while the tack of the related information on the occurrence process and source of the event by using observed data including pollutant concentration distribution is the reversal of the forward track. Compared with the prediction of pollutant concentration distribution, the traceability of pollutant is obviously more complex, not only involving the process of track and backward reconstitution of the event but also having nonlinearity and ill-posedness as the reverse problem of prediction.

According to the connotation of sudden water pollution traceability, the definition of such traceability can be divided into five categories: The first category of traceability refers to reconstitution of unknown coefficients in a pollutant transport model according to some (observed) information of temporal and spatial distribution of a known pollutant, including longitudinal dispersion coefficient, lateral dispersion coefficient, and degradation coefficient, and the research is called a problem on parameter identification research; the second category refers to inference of the right pollution source (collection) item in the model by using observed values, including pollution source position and release intensity and time, and such traceability is also called an identification problem of the pollution source (collection) item (called a traceability problem for short); the third category is reverse inference of initial conditions on the basis of known information, and such traceability is also called a backward-time research problem; the fourth category means to infer the type or parameters of boundary conditions according to known information, namely, the reverse inference research of boundary conditions; and the fifth category is the combination of the four categories above. The

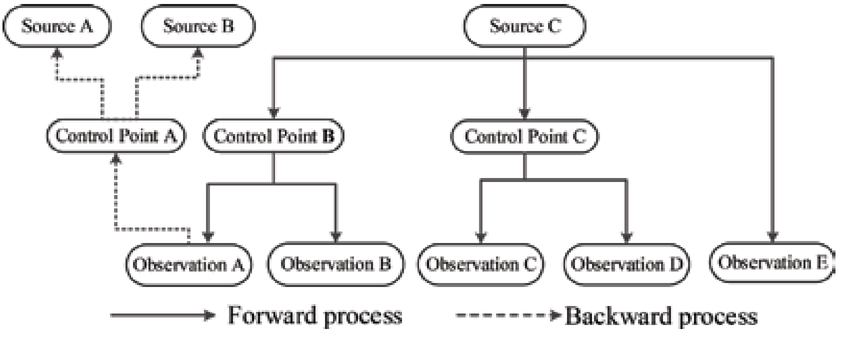

Figure 1.
Traceability process of sudden water pollution.

pollutant traceability technology studied in this chapter is mainly about the second category traceability problem, that is, the identification problem of source (collection) items.

Attention has being paid gradually to the research of the traceability of sudden water pollution in recent years. There are two kinds of common research methods: deterministic method and probabilistic method [4, 5]. The deterministic method mainly includes a regularization method [6, 7], a trial and error method [8], a least square method [9, 10], and other optimization methods. These methods have a clear physical meaning. A single solution can be obtained by using them, and the error is often great if information is not accurate. However, the probabilistic method is a kind of random method based on Bayes inference and Markov chain Monte Carlo (MCMC) sampling, providing multiple reliable alternative results depending on the collection of distribution information on random variables [4, 11, 12]. On the basis of common traceability methods, this chapter introduces a new fast traceability method for sudden pollution in a channel, which combines the deterministic and probabilistic methods [1] and provides multiple reliable alternative results for urgently coping with sudden pollution without depending on distribution information on random variables.

3. Common traceability methods

3.1 The deterministic method

The deterministic method is developed mainly by reference to the traceability research of underground water. Perform research on pollutant concentration distribution in a simulation event by using a pollutant transfer and diffusion model, set up an optimization model which takes the error square sum between the simulation result and the result actually measured as an objective function, solve the objective function of the optimization model by using a deterministic algorithm, and then seek for, via iteration, the calculation result which best matches the values actually observed. Typically, the deterministic method includes the regularization method, trial and error method, correlation regression analysis, and the like, as well as some heuristic algorithms including genetic algorithm and simulated annealing algorithm. Regarding the tedious problems of the trial and error method, Han et al. [8] performed the inversion by using "local and basic expansion of a forward problem" and the optimal control solution by converting traceability into a minimum value problem. Jin and Chen [13] obtained a response relation between the concentration of pollutants in a channel via variation of an objective function established by using the Laplace transform. On such basis, they determine the pollution source intensity of an upstream cross section under environmental capacity control via reasoning. Chen et al. [14] transformed the calculation expression of instantaneous discharge concentration at a 1D single-point source, obtaining a linear regression model. And via regression analysis, they determined the release position, time, and intensity of the pollution source as well as the longitudinal dispersion coefficient of a channel. Min et al. [15] carried out, by using genetic algorithm, the research, respectively, on the identification problem of multiple parameters including the flow velocity, diffusion coefficient, and attenuation coefficient of a 1D river, as well as on the identification problem of right items of a 1D convection-diffusion equation. This part presents a simple introduction of the deterministic method by taking the correlation regression analysis method as an example.

When pollutants are released instantaneously, the analytic solution of the 1D convection-diffusion equation can be further determined via Fourier transform or dimensional analysis. Assuming that the occurrence position and time of pollution are x_0 and t_0, respectively, then we obtain

$$C = \frac{M}{\sqrt{4\pi D(t - t_0)}} \exp\left(-\frac{((x - x_0) - u(t - t_0))^2}{4D(t - t_0)}\right) \tag{1}$$

where M is the average intensity of the initial surface source of the released pollutant along a cross-section, g/m^2.

After taking logarithmic transformation of both ends of Equation (1), we obtain

$$\ln C = aX + b \tag{2}$$

where $X = ((x - x_0) - u(t - t_0))^2$, $a = -\frac{1}{4D(t - t_0)}$, and $b = \ln \frac{M}{\sqrt{4\pi D(t - t_0)}}$.

For a fixed pollution event, the occurrence position x_0 and the time t_0 are constants. Therefore, if the concentration of multiple cross sections can be obtained at the same time and the control time parameter $(t - t_0)$ is a constant, then it is easy to judge that $\ln C$ and X (i.e., the observation position x) meet a linear relation; and a linearly dependent optimization model can be built by using a regression analysis method [14]:

$$R = \frac{\sum_{i=1}^{n}\left(\ln C_i - \overline{\ln C}\right)\left(X_i - \overline{X}\right)}{\sqrt{\sum_{i=1}^{n}\left(\ln C_i - \overline{\ln C}\right)^2}\sqrt{\sum_{i=1}^{n}\left(X_i - \overline{X}\right)^2}} \tag{3}$$

With the position of the monitored cross section as a variable, after selection of x_0 and t_0, the concentration measured C_i at different monitored cross sections x_i at the same time and corresponding to X_i, and at the monitored position corresponding to the same time are obtained via calculation. Theoretically, when x_0 and t_0 are selected as the true position and time of occurrence, R obtained via calculation should be 1.0.

From this, the problem of pollutant traceability is converted into determination of proper x_0 and t_0 via Formula (3), and the R meeting calculation is maximum, that is, 1.

The solution can be completed in five steps:

1. Firstly, calculate a proper $x' = x_0 + u(t - t_0)$, and it is selected to be 1.0 if meeting R. The calculation can be realized via derivation.

2. Calculate X_i corresponding to different observed cross sections at the same time according to known x', fit $\ln C$ and X in combination of Formula (2), calculate the corresponding slope a and intercept b, and therefrom, obtain t_0 via calculation.

3. Calculate the time parameter $t_0 = t + \frac{1}{4Da}$ according to calculated slope a, then $x_0 = x' - u(t - t_0)$ in combination of known x' and finally the intensity of the initial surface source $M = \sqrt{4\pi D(t - t_0)} \exp(b)$ in combination of known b.

4. The intensity of discharged pollutants calculated by using flow cross-section area A at time t_0 at estimated pollution position x_0 is MA.

5. Considering the existence of fitting and observation errors, calculate R via back substitution of obtained x_0 and t_0, then judge whether R is 1. If R is 1 (or the error is very small), then complete traceability calculation. Otherwise go back to Step (2), properly adjust the slope a and intercept b, and then recalculate x_0 and t_0.

From the aforesaid solution process, we can see that the observed data of multiple cross sections are needed at the same time for the model to realize the traceability of the sudden pollution due to a point source.

3.2 The probabilistic method

Because a traceability problem has ill-posedness, when the deterministic method is used for solution, the error in observation or model calculation might result in great deviation in results and distortion in the result of traceability. For this, random devices are introduced to the study of traceability. Among such devices, the one mostly used is the probabilistic method based on Bayes inference and MCMC sampling. Bayes inference is a kind of method based on the theoretical foundation of probability theory, which can reflect the uncertainty of sudden water pollution events in a channel. It solves the posterior probability distribution of such parameters on the basis of making full use of a likelihood function and the priori information of parameters to be determined and then obtains the estimated values of all parameters of a pollution source by respective sampling. This method can offer a random distribution function of the traceability result of a water pollution event. Therefore, the methods based on Bayes inference are mainly used to estimate happening probability of a sudden water pollution event. By using them, we can obtain the posterior probability distribution of the traceability result instead of a single solution, quantitate the uncertainty of the result, and gain more traceability information on the event. In order to acquire the estimated value of the result, Bayes inference should be combined with related sampling methods, such as Markov chain Monte Carlo (MCMC), randomized quasi-Monte Carlo (Monte Carlo, MC), and other sampling methods [11, 12]. Among them, the MC method is an estimation method in which an initial value easily converges to a suboptimal solution no matter whether the initial value deviates from its true value. Therefore, the accuracy rate of the traceability result obtained via this method is not high. The defect in the MC method can be compensated by iterating the mode of Bayes inference's combination with the MC or MCMC method into the obtained distribution function of the traceability result. The MCMC method is a sufficiently long Markov chain acquired via random walk, only by which can the proximity of the sampling result to the posterior distribution of the traceability result be ensured, that is, the limit distribution of Markov chain is used to express the posterior probability density function of the traceability result. For this, the MCMC method expands the application of Bayes inference in the traceability research of environmental pollution events.

Taking Bayesian MCMC method, for example, this part provides a simple introduction of the traceability method in probability statistics. The method takes all variables in a traceability research model as random ones and considers that the solution to the traceability problem of a sudden water pollution event is a probability distribution. Firstly, it converts the priori information of the solution into a priori probability distribution by using Bayes method, then combines with observed data, and finally obtains the posterior probability distribution of the solution to be determined from the random sampling process of Markov chain by using the likelihood function between observed values and the values calculated via simulation. The probabilistic method includes the following main processes:

1. Model a basic traceability problem, and quantitate the priori information quantity of related traceability solutions (the intensity, occurrence position, and time of a pollution source) by adopting a reasonable probability distribution function.

2. Select and establish a reasonable likelihood function based on a channel water quality coupling simulation model and in combination of the information related to the site of a sudden water pollution event and to the hydrologic data of water space of the event.

3. Obtain a posterior probability distribution of traceability solutions based on the prior probability distribution and the likelihood function.

4. Perform sampling of the posterior probability distribution and acquire the estimated values of traceability solutions to the event.

Firstly, express the probability distribution table of the priori information on the known parameter vector θ before collecting observed data into $p(\theta)$. After obtaining observed data, the posterior distribution of the known parameter acquired via Bayes inference is $p(\theta|d)$, meeting

$$p(\theta|d) = p(\theta)p(d|\theta)/p(d) \quad (4)$$

where θ stands for three parameters of the pollution source; d is the measured value of pollutant concentration.

$p(d)$ is the probability distribution of the measured value. $p(d)$ obviously has no relation with parameter θ. In the case with a known concentration actually measured, $p(d)$ can be understood to be 1. From this, we obtain

$$p(\theta|d) = p(\theta)p(d|\theta) \quad (5)$$

The prior distribution $p(\theta)$ of parameter θ in Formula (5) can be deemed a uniform distribution of a respective parameter within a prior range of values.

$$p(\theta) = \prod U(\theta_i) \quad (6)$$

Therefore, to determine the posterior distribution $p(\theta|d)$ of pollution source parameters, we firstly need to determine distribution $p(d|\theta)$, which can be defined as the likelihood function of a measured and a predicted value. Let d_i , $C_i(x,t|\theta)$, and $p(d_i|\theta)$ to be the measured value, predicted value, and likelihood function of no. i measuring point, respectively, and $\varepsilon_i = d_i - C_i(x,t|\theta)$ to be an error of measurement, $i = 1, 2, \ldots, N$. Assuming that obedience mean of error ε_i is 0, standard deviation is the normal distribution of ε_i, and all measuring points are independent from each other, then

$$p(d|\theta) = \prod_{i=1}^{N} p(d_i|\theta) = \frac{1}{\left(\sqrt{2\pi}\sigma\right)^N} \exp\left(-\sum_{i=1}^{N} \frac{\left(d_i - C_i(x,t|\theta)\right)^2}{2\sigma^2}\right) \quad (7)$$

Thereby, we obtain the posterior distribution of pollution source parameters $p(\theta|d)$:

$$p(\theta|d) = p(\theta)p(d|\theta) = \prod U(\theta_i) \cdot \frac{1}{\left(\sqrt{2\pi}\sigma\right)^N} \exp\left(-\sum_{i=1}^{N} \frac{\left(d_i - C_i(x,t|\theta)\right)^2}{2\sigma^2}\right) \quad (8)$$

However, complicated $C_i(x,t|\theta)$ or great model parameter space and number of dimensions make $p(\theta|d)$ very abstract and difficult to be expressed visually. Therefore, direct use of Bayes method almost cannot solve an actual problem directly, which, however, can be solved by means of the MCMC method. According to different transition probability matrixes making up the Markov chain, the MCMC method mainly has the following sampling algorithms: Gibbs sampling algorithm, Metropolis-Hastings algorithm, and self-adaptive Metropolis algorithm. The self-adaptive Metropolis algorithm can converge to a target distribution for any prior distribution of θ, so this algorithm is selected for sampling.

According to the aforesaid derivation process, the main steps for traceability of a sudden pollution event based on Bayes inference and the MCMC method are as follows:

i. Suppose $i = 0$, and then initialize different variables.

ii. Produce and accept random variables, and make up a Markov chain.

  1. The priori parameter θ producing a uniform distribution equals $\theta(m, x_0, t_0)$.

  2. From the produced pollution source parameters, calculate the concentration value at the observing point by using the methods in Chapter 3.

  3. Calculate the likelihood function $p(\theta|d)$ by using Formula (8).

  4. Calculate the acceptance probability of the Markov chain by using the following formula:

$$\alpha = \min\left\{1, \frac{p(\theta^*|d)}{p(\theta_i|d)}\right\}$$

  5. A random number R is produced which uniformly distributes within 0–1. If $R < \alpha$, then accept the parameters for this test and make $\theta_{i+1} = \theta^*$. Otherwise let original parameter $\theta_{i+1} = \theta_i$.

iii. Repeat Steps (1)–(5) until reaching the number of predefined iteration or acquiring the posterior sample number preset for pollution source parameters, and then perform statistics of the posterior distribution law of all parameters and complete the traceability calculation.

From the aforesaid process, we can see that the probability statistics method depends on the priori range of values of parameters and the information of calculation error distribution and that the convergence rate of producing parameter values randomly by using uniform distribution is very slow if the priori range of values is large. Moreover, the probabilistic method reaches solution based on Markov chain Monte Carlo random sampling, without directivity for making no optimal judgment of existing results, also resulting in a lower efficiency of traceability.

4. Coupled probability density method

4.1 Basic principles and method

According to the introduction and analysis of the common methods mentioned above, the deterministic method has a clear physical meaning but a single solution, resulting in a great error in the case with inaccurate information. Although the probability statistics method can offer multiple reliable alternative results, it depends on the acquisition of distribution information on random variables. For the point of view of research trend, a new generation of traceability method which integrates the deterministic method and the probability method will become the future trend of pollutant traceability research. The coupled probability density method proposed in this chapter combines the deterministic method with the probability method and sets up a sudden water pollution traceability model based on a coupled probability density function (C-PDF). Based on hydrodynamic calculation and considering the observation error of a system, an optimization model is built by taking pollution source position and release time as parameters and through correlation analysis of the forward concentration distribution probability density and backward position probability density of pollutants, and then the solution of such model is realized by using DEA. On such basis, a minimum value optimization model is established based on the forward concentration distribution probability density function of pollutants to determine the intensity of the pollution source.

In case of a sudden event, the majority of pollutants enters a channel in the form of a point source; move and convert with water flowing in the channel. Based on traceability research at home and abroad and the transfer and conversion law of pollutants in a channel, this paper sets up a 1D traceability model for sudden river water pollution depending on a coupled probability density function (C-PDF) and infers the pollution source in the light of observed concentration of pollutants, thus realizing the reconstitution of pollution event. Based on 1D hydrodynamic calculation and considering the observation error of a system, an optimization model is built by taking pollution source position and release time as parameters and through correlation analysis of the forward concentration distribution probability density and backward position probability density of pollutants, and then the solution of such model is realized by using DEA. Meanwhile, a minimum value optimization model is established based on the forward concentration distribution probability density function of pollutants to determine the intensity of the pollution source.

The magnitude of pollutant concentrations at different cross sections in a channel can be expressed as the statistic at such cross sections where tiny substance particles appear at a respective time. Statistically, the high or low probability that substance particles appear at some position can be expressed by a probability function, and the concentration distribution of pollutants in a channel can be also described by some probability density function. On the contrary, without knowing the pollution source, the pollutants observed at some cross section in a river might come from any upstream place, whose probability also can be described by a probability density function. Considering that pollutant traceability is the reverse problem of pollutant concentration distribution, a probability density function used to describe such distribution is called a forward concentration probability density function. However, a probability density function used to describe the probability that the pollution source is at different positions is called a backward position probability density function. Depending on the definition of the probability density function, a forward probability density function can be obtained after normalization of concentration.

According to $\int_x C(x,t)dx = m_0$, the normalization of $C(x,t)$ can be done, as shown below:

$$c(x,t) = C(x,t)/m_0 \tag{9}$$

where $c(x,t)$ is the forward concentration probability density value corresponding to $C(x,t)$, with a dimension of m^{-1}, indicating the probability that pollutants appear at cross section x at time t.

According to Neupauer and Wilson's derivations, forward concentration transport and backward position traceability are adjoint processes to each other [16, 17]. For a 1D channel, $P(x_s,t')$ is used to express the probability, judged from an observed cross-section x_d, that a pollution source is at x_s at time t' (i.e., the probability that pollutants are transported from cross-section x_s to cross-section $t_d - t'$ within time x_d, then $P(x_s,t')$ meets the adjoint state equation of a convective diffusion Eq. (10)) and normalization conditions ($P(x_s,t')$ also has a dimension of m^{-1}), as shown in the following formula:

$$-\frac{\partial P(x_s,t')}{\partial t} + \frac{\partial(uP(x_s,t'))}{\partial x} + D\frac{\partial^2 P(x_s,t')}{\partial^2 x} = 0 \tag{10}$$

$$P(x_d,t_d) = 1 \tag{11}$$

where t' is the time point for inverse calculation and t_d is the time point for observing pollutant concentration. Formula (11) indicates that the pollution source only can be at the observed cross section when pollutants are not transported ($t' = t_d$) but appear at such cross section.

Like concentration transport process, Formula (11) can be considered a position probability transport process. Similarly, when u is constant, an analytic solution can be obtained, as shown below:

$$P(x_s,t') = \frac{1}{\sqrt{4\pi D(t_d - t')}} \exp\left(-\frac{(x_d\text{-}x_s - u(t_d - t'))^2}{4D(t_d - t')}\right) \tag{12}$$

We can see that $P(x_s,t')$ and $C(x,t)$ have the same form. Actually, the relation between Formulas (1) and (12) decides that notwithstanding flow field, the relation between $P(x_s,t')$ and $C(x,t)$ can be determined by **Figure 2**, on which the arrows show the direction of transport (advection item).

Pollutant concentration distribution and traceability are inverse problems to each other, but both of the problems follow the same physical law basically. **Figure 2** shows that when $t - t_0 = t_d - t'$, the probability $c(x_d,t)$ that pollutants move from source x_0 to cross-section x_d after time $t - t_0$ equals to the probability, judged by an observer at cross-section x_d, that pollutants move from cross-section x_0 to cross-section x_d after time $t_d - t'$. Namely, if $t - t_0 = t_d - t'$, the following Formula (13) holds

$$P(x_0,t') = c(x_d,t) \tag{13}$$

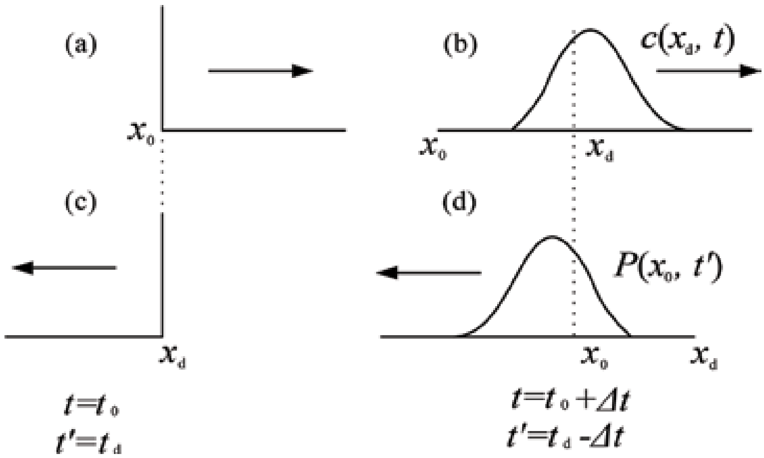

Figure 2.
Processes of forward concentration transport and backward position probability transport. a and b are the concentration probability distributions along the river at release time t_o and after Δt, respectively; c and d are the position probability density distributions at monitoring time t_d and of backward traceability for Δt, respectively.

From **Figure 2** and Formula (13), we can see that forward concentration transport process is highly coupled with backward position probability transport process. The two processes are exactly the same except for backward-time calculation direction. Because $P(x_s, t')$ has no relation with the intensity of the pollution source, $P(x_s, t')$ can be directly calculated by using Formula (10) or (12) with $x_s = x_0$ and $t' = t_d + t_0 - t$ given. Therefore, a linearly dependent model can be established by substituting $P(x_s, t')$ for $C(x_d, t)$ based on such coupling relation in Formula (13) to realize traceability calculation.

Assuming that observed concentration series is C_i and that the corresponding calculated position probability density series is P_i, $i = 1, 2, \ldots n$, according to the aforesaid derivation, we can obtain the expression of the correlation coefficient r of the two series is as follows:

$$r = \frac{\sum_{i=1}^{n}\left(C_i - \overline{C}\right)\left(P_i - \overline{P}\right)}{\sqrt{\sum_{i=1}^{n}\left(C_i - \overline{C}\right)^2}\sqrt{\sum_{i=1}^{n}\left(P_i - \overline{P}\right)^2}} \tag{14}$$

where $\overline{C}$ and $\overline{P}$ are the arithmetic averages of C_i and P_i, respectively.

According to Formulas (13) and (14), we can set up the following function:

$$\min(abs(1 - r)) \tag{15}$$

The constraint condition is the value range of x_0 and t_0, which is given by priori information and generally is obtained via field survey or existing information. See Formulas (16) and (17):

$$x_{0\min} \le x_0 \le x_{0\max} \tag{16}$$

$$t_{0\min} \le t_0 \le t_{0\max} \tag{17}$$

Through solution of the aforesaid optimization model, we can obtain the release position x_0 and time t_0 of the pollution source.

The optimization model realizes the decoupling of pollution source intensity and time parameter, so determine position and time parameters firstly and then the intensity of the nonpoint pollution source m_0. Considering that the occurrence position and time of the pollution source have been determined and that the range of pollution source intensity m_0 can be roughly determined by the previous model, the forward probability density function can be used to build an optimization model to calculate pollution source intensity. See Formulas (18) and (19).

$$\text{Target function } \min\left(\sum \omega_i (m_0 * c_i - C_i)^2\right) \tag{18}$$

$$\text{Constraint condition } m_{0\min} \le m_0 \le m_{0\max} \tag{19}$$

where coefficient $\omega_i = \frac{1}{(C_i+1.0)^2}$ is used to remove small concentration error loss arising from a large concentration difference.

After verification and analysis, the optimization models (18) and (19) can not only solve the intensity of pollution source m_0 rapidly but also have an advantage in observed error filtration.

4.2 Model solution

The model uses a smart differential evolution algorithm (DEA) method for solution, which is similar to the genetic algorithm [1, 18]. Solving a traceability optimization model by using the DEA includes population initialization, mutation, recombination, and selection. Take the first optimization model made up by Formulas (14) and (16), for example. The DEA defines x_0' and t_0' as property elements and produces basic evolutionary individuals, with $F = abs(1 - r)$ as a fitness target function. The detailed steps of solution are as follows:

(1) Population initialization: let the scale of population equal to *NP*, and then produce the first individual $X_i^{Gen}(x_0, t_0)$ from Formula (20) ($i = 1, 2, \ldots, Np, Gen$ are the individual and evolution generation numbers, respectively):

$$X_i^{Gen} = X_{\min} + rand(0,1)(X_{\max} - X_{\min}) \tag{20}$$

where $\text{rand}(0,1)$ stands for a random number within [0,1]; $X_{\min}$ and $X_{\max}$ are the minimum and maximum values of X_{iGen}, respectively. After initialization, calculate their fitness values $F(X_i^{Gen}) = abs(1 - r(X_i^{Gen}))$, respectively.

(2) Mutation: select three individuals X_{r1}^{Gen}, X_{r2}^{Gen}, and X_{r3}^{Gen} from population $Gen(1 < Gen < \max Gen)$ via uniform sampling, and then produce a mutation individual V_i^{Gen} from Formula (21):

$$V_i^{Gen} = X_{r1}^{Gen} + CF\left(X_{r3}^{Gen} - X_{r2}^{Gen}\right) \tag{21}$$

where *CF* is a scaling factor, generally $CF \in [0.5, 1]$, $r1, r2, r3 = 1, 2, \ldots, NP$, and $r1, r2, r3$ are all not i. If $V_i^{Gen} \notin [X_{\min}, X_{\max}]$, then regenerate a mutation individual V_i^{Gen} from Formula (20).

(3) Recombination: before and after mutation, each property element of an individual undergoes recombination and produces a new individual U_i^{Gen}. The rule of recombination is as follows:

$$U_i^{Gen}(x_j) = \begin{cases} V_i^{Gen}(x_j), rand(0,1) < CR \ or \ randn = j \\ X_i^{Gen}(x_j), others \end{cases}, j = 1, 2 \tag{22}$$

where CR is a recombination constant, generally $CR \in [0.8,1]$, $U_i^{Gen}(x_j)$ refers to the property value of x_j in U_i^{Gen}, and $randn$ is a random value of 1 or 2.

(4) Selection: superior individuals replace an original individual and go into the next generation. Select superior individuals according to Formula (23):

$$X_i^{Gen+1} = \begin{cases} U_i^{Gen}, F(U_i^{Gen}) < F(X_i^{Gen}) \\ X_i^{Gen}, F(U_i^{Gen}) > = F(X_i^{Gen}) \end{cases} \tag{23}$$

According to the given steps, perform cycle evolution and iterative computation until adaptability meets requirements (the value of target function meets limited conditions, for example, it can be controlled by $r \geq 0.95$) or ends when it evolves to a maximum algebra maxGen. After that, select the individual having the minimum value of fitness function (the parameter values that individual elements stand for are x_0 and t_0 to be determined), and the calculation is completed.

4.3 Improvement to coupled probability density method

The model uses the DEA for solution, the result of calculation has a great randomness, and some optimized results might appear repeatedly. Therefore, some optimized rules should be added to control the value of optimized fitness function to make it change unidirectionally. For this, this chapter puts forward the application of gradient concept in a differential evolution process to provide an optimization direction and improving optimization efficiency.

Firstly, add two new attributes signifying gradient into an individual, namely, gradient feature factor $gc(X)$ and gradient direction factor $gp(X)$. A population individual newly defined is $S_i^{Gen} = S_i^{Gen}(X_i^{Gen}, gc(X_i^{Gen}), gp(X_i^{Gen}))$, where the gradient feature factor $gc(X_i)$ and gradient direction factor $gp(X_i)$ are determined from Formulas (24) and (25).

$$gc(X_i^{Gen}) = \begin{cases} 1, & F(X_i^{Gen}) \leq F(X_i^{Gen-1}) \\ 0, & F(X_i^{Gen}) > F(X_i^{Gen-1}) \end{cases} \tag{24}$$

$$gp(X_i^{Gen}) = \begin{cases} [sgn(X_i^{Gen}(x_1) - X_i^{Gen-1}(x_1)), sgn(X_i^{Gen}(x_2) - X_i^{Gen-1}(x_2))], \ gc(X_i^{Gen}) = 1 \\ 0, \ gc(X_i^{Gen}) = 0 \end{cases} \tag{25}$$

where sgn is a symbolic function.

After setting gradient attributes, the trend information that facilitates population evolution can be kept during differential evolution selection. For example, supposing an individual X_i^{Gen} is superior to its previous generation of individuals X_i^{Gen-1} and $gp(X_i^{Gen}) = [1, -1]$, then this means that the transient superior evolution trend of this individual is "increasing position parameter x_0 and reducing time parameter t_0." The information on such trend can be used to guide the evolution of the next generation of individuals. When making improvement to DEA optimization in actual applications, we can utilize and obtain gradient information as an additional rule to control evolution direction and guide the evolution of each generation of individuals.

5. Application examples

As shown in **Figure 3**, a emergent treatment demonstration project of sudden water pollution events was carried out from the check gate in the Fangshui River at Jingshi Section (Pile 100 + 294.750) to the check gate in the Puyang River (Pile No. 113 + 492.750) of the SNWDMRP on March 22, 2014. Cane sugar was used as tracer, and the concentration of cane sugar solution, as a water quality detection index. The experimental channel has a trapezium cross section, whose parameters are bottom width = 18.5 m, side slope = 2.5, average water depth = 4.0 m, channel bottom slope = 0.00005, manning roughness = 0.015 (design value), the experimental estimated value of dispersion coefficient = 3.43 m^2/s, and constant flow ≈ 8.0m^3/s. About 800 kg of cane sugar was put instantaneously into the channel at 9:00 AM on March 22, and four monitoring cross sections were set downstream. In order to ensure successful instantaneous putting of 800 kg cane sugar, the cane sugar was dissolved in heated clear water of 1.0 m^3 (about 80.0°C) timely and in advance. When the experiment started, a floating bridge already built was used to directly pour dissolved cane sugar solution into the middle of the channel. The concentration monitoring process at 1508 m downstream of the cross section where the solution was poured is shown in **Figure 4**, and the start time of monitoring was 9:30.

Supposing the pile number of the pouring cross section is 0 m, then the pile number of the monitored cross section is 1508 m. The width and water depth at the experimental section of the channel were very small with regard to the length. After checking calculation, the pollutants 1508 m from the monitored cross section were uniformly mixed roughly in the lateral and vertical directions. The time for urgently coping with sudden pollution is short. Without considering degradation, the transport of pollutants can be described by using a convective diffusion equation at a 1D constant flow. A traceability model is applied for traceability calculation,

Figure 3.
Schematic diagram of experiment channel selected for demonstration project.

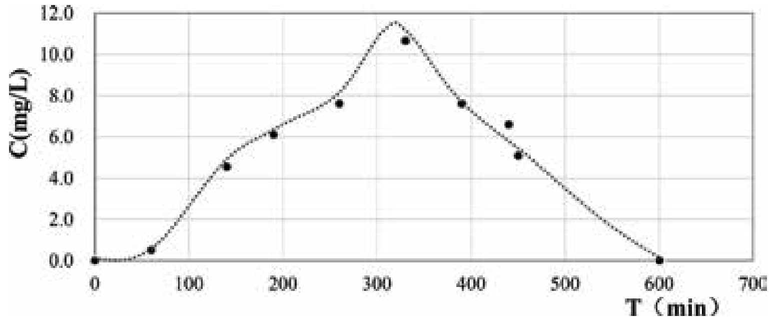

Figure 4.
Actual concentration monitoring process at monitored cross section.

	M (kg)	x_0 (m)	t_0
Actual value	800.00	0.00	9:00
The value calculated by the probabilistic method	781	−570	9:14 AM
The value calculated by the coupled density method	731.13	−28.42	8:48 AM

Table 1.
Table of results of traceability by all methods.

	M (kg)	x_0 (m)	t_0
The error calculated by the probabilistic method	2.38%	570	14 min
The error calculated by the coupled probability density method	8.59%	28.42	12 min

Table 2.
Analysis of errors of traceability by all methods.

which was established by using the probability statistics method and the proposed coupled probability density method among conventional traceability methods. The results are shown in **Tables 1** and **2**.

From **Tables 1** and **2**, we can see that the error in the position of a pollution source calculated by using the coupled probability density method is not up to 30 m, time error about 12 min, and the intensity of pollutants not larger than 10%. However, considering the calculation done by the probability statistics method, although the precision of its pollutant intensity is higher, the position and time are not calculated more accurately, especially the error of the position calculated is above 500 m. Furthermore, to solve the same problem, it takes much more time for calculation by the probability statistics method than by the probability density method. As a whole, when a sudden water pollution accident happens during water diversion, the adoption of the traceability technology based on the coupled probability density method is recommended more.

6. Summary

Due to uncertainty of sudden water pollution events, after occurrence of an event, it is very difficult to determine the location and release time by experience. Therefore, the first task for emergent treatment is to determine the pollution source. The main content of fast traceability of sudden pollution is to determine the location, release time, and the intensity of the pollution source. Regarding fast determination of the conditions of a pollution source in case of sudden water pollution under SNWDMRP, this chapter presents detailed introduction and analysis of the fast channel traceability technology including conventional traceability, provides practical effect of the traceability technology via analysis of example applications, and recommends the application of the coupled probability density method to traceability calculation according to traceability results, in order to provide support for fast traceability and emergent regulation and control of sudden water pollution under long-distance water transfer projects.

However, the traceability model can have good application results for single-point source of sudden water pollution accident in a river. The traceability model for multipoint sources of sudden water pollution accidents needs deeper research and can refer to the ongoing research by Wang [19].

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Chapter 4

Emergency Operations of Sudden Water Pollution Accidents

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Abstract

Emergency operation technologies can help to make reasonable operation measures of hydraulic structures, which are important to control the scope of the effect arising from an event and reduce the harm caused thereby. The main canal of MRP is divided into three parts in case of sudden water pollution accidents: the accident pool, the upstream section of the accident pool, and the downstream section of the accident pool. For each part, the target and strategy for emergency operation technologies are discussed. With regard to an accident pool, multiple kinds of check gate closing methods, synchronous, asynchronous, identical speed, and different speed are put forward; for the upstream section, a new method of equal-volume operation is introduced; and for the downstream section, three emergency operation methods are proposed. The simulation result of case study shows that the methods raised in this chapter can be used to determine suitable emergency operation measures.

Keywords: emergency operation technologies, gate closing methods, equal-volume operation, accident pool, upstream section, downstream section

1. Introduction

When a sudden water pollution accident happens, control structures including check gate, pumping station, and dam can be utilized for emergency operation to reduce the harm caused thereby [1]. Regarding sudden water pollution accidents that are unforeseeable in the MRP, safety and stability are the ones of the key problems to which most attention are paid in an emergency regulation process. The main response characteristic parameters which reflect the safety and stability of a water transfer system are the response duration time from the transition of the steady water state to an emergency state of an open channel, as well as the change range and change speed of the water level in the channel during such response duration. The factors that affect these response characteristic parameters mainly include flow, check gate closing mode and time, and the use of drainage gate. Therefore, it is necessary to develop a complete set of operation rules to achieve safe and stable operation of a channel under emergency conditions. In addition, in order to make the operation process of the channel go with the aforesaid rules, emergency conditions often lead to great changes in the operating conditions of the channel. At present, the mainly conventional control algorithm of a channel is PI control algorithm. The control condition generally is a small change in water diversion and restores to the original water diversion process after lasting for a not very long period of time; or water diversion experiences a small change, but fails to restore to the original water diversion process [2]. In case of an emergency, the operating conditions of a channel pool undergo great changes.

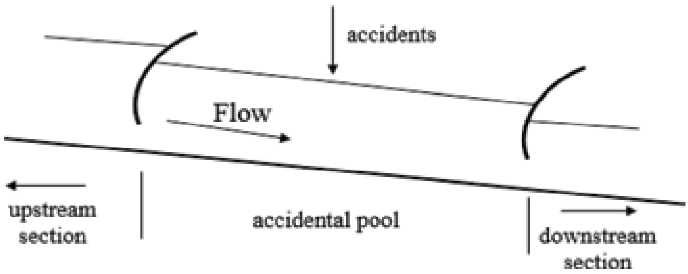

Figure 1.
Schematic diagram of subsections of main canal of MRP.

For this, the main characteristics of the channel, such as the lagged effect of the upstream of the channel on the downstream, as well as the cumulative impact of flow on the water level, will change [3]. At this time, it is difficult to achieve emergent adjustment by using a conventional automatic control system. Therefore, an automatic check gate control algorithm should be used for a change in an emergency to meet the fast smooth transition from a conventional process to an emergent process and then to a conventional process.

2. Target of emergency operations

In case of a sudden water pollution accident in the main canal in the MRP, channels are divided into three parts: accident pool, the upstream section of the accident pool, and the downstream section of the same, for which, joint emergency operations will be carried out (**Figure 1**).

2.1 Overall operation strategies

In case of an emergency, related operation strategies should be developed at the first time. All check gate operation rules should fulfill the strategies. The following control strategies are put forward by referring to the control strategy of some large water transfer project and the experience of water diversion:

2.1.1 Reduce excess flow and storage volume as soon as possible to control the development trend of an event

In the event of an emergency, cut off the accident section or reduce the flow of water that transfers downstream. If excess flow and storage volume occur in all channel pools at the same time, and if their reduction is not done as soon as possible, the channel pools will suffer overflow and water loss. Therefore, all upstream check gates of the accident section should have their flow reduced or even cut off. Not only should the "excess flow" be reversed but also the "excess storage volume" should be reduced.

2.1.2 Do not affect water supply at upstream turnouts as far as possible

Because the water volume in upstream pools tends to increase rather than decrease after an emergency, there is adequate water supply to the turnouts. Therefore, after the emergency, operation measures should not interfere with the water supply at upstream turnouts.

Use the regulation and storage capacity of channels and try not to waste and abandon water so as to reduce economic losses.

2.1.3 Maintain storage volume in channels when in normal water diversion

Maintain the volume of the canal pools as much as possible at the volume storage which the pools have in normal condition, so as to reduce the repeated adjustment of the storage volume and the operation and management costs.

A general water supply channel adopts a constant downstream depth mode. The response and the recovery characteristics of this mode lead to repeated adjustment of storage volume before and after channel pool flow switching, not only increasing the costs of operation and management but also taking a lot of time in storage volume regulation. Comparatively speaking, equal-volume operation has no defects. Therefore, equal-volume operation should be applied if a channel satisfies the condition for using an equal-volume operation mode.

2.1.4 Regulate the flow and storage volume in the channel so as to reduce the time of contingency transition state

Some research shows that the channel response and recovery time of synchronous gate operation is shorter than that of asynchronous gate operation, and synchronous operation should be preferred in emergency phase.

2.2 Gate operation rules

According to the channel pool where a water pollution event occurs and the control strategy thereof, a whole channel can be divided into accident pool, the upstream section of the accident pool, and the downstream section of the same. Use different operation rules for different control strategies.

2.2.1 Control rules for drainage gate

The first main purpose of using a drainage gate is for clipping peak and reducing the maximum backwater height of a check gate; the second is for regulating the upstream water level of the check gate during fluctuation of the water level in the channel so as to make them come near to the target water level. The use of drainage gate, however, also results in water waste. Therefore, the drainage gate is not opened generally. It is only opened when the water level endangers the safety of a project. A warning level indicator is used here to judge whether to adopt drainage gate, and it is used when the warning level is above the warning level.

Drainage gate control rules for an accident pool: when the upstream water level of drainage gate is higher than the warning level, the drainage gate is used; when the volume stored in the channel pool comes near to the target volume, the drainage gate should be closed rapidly. After the water surface of the accident pool becomes steady, then decide whether to reuse the drainage gate to abandon water, according to the level of the accident. If water body in the channel pool fails to realize self-purification or too long duration of self-purification which has an effect on downstream water supply, the drainage gate should be reused to discharge polluted water body into a temporary water pool or an abandoned lake by the channel section for treatment. Close the drainage gate and restore water supply after the channel pool is drained.

Drainage gate control rules for the upstream section of an accident pool: when the upstream water level of all upstream check gates is above the warning level, use the drainage gate; for a channel section without drainage gate, when the upstream water level is above the warning level, do not close the check gate temporarily for current calculation. The purpose is for clipping peak and reducing the maximum backwater height of a check gate; the second is for regulating the upstream water level of the check gate during fluctuation of the water level in the channel so as to make them come near to the target water level.

The drainage gate is not used for the downstream section of the accident pool, because the downstream section is where inflow water is smaller than outflow water. When a reasonable check gate operation process is used, the situation that the upstream water level is above the warning level will not happen basically. Moreover, the drainage gate will not be used in order to keep water supply at the downstream section as much as possible.

2.2.2 Control rules for turnout

Turnout control rules for an accident pool: the turnout closes rapidly as the check gate closes rapidly to reduce the impact of polluted water diversion.

The turnout at the upstream section of the accident pool does not involve emergency scheduling, keeping original normal scheduling opening, and ensuring normal water supply at the upstream section.

Turnout control rules for the downstream section of an accident pool: the flow of all turnouts at the downstream section is controlled according to the reduction ratio of the flow of the downstream check gate of the channel.

2.2.3 Control rules for check gate

Check gate control rules for an accident pool: the two check gates should be closed rapidly to prevent clean water of the upstream channel pool from entering the accident pool and to avoid polluted water body from flowing into the downstream channel pool.

Check gate control rules for the upstream section of an accident pool: a relevant operation plan should be developed according to a water level control target in order to ensure normal water supply at the upstream section.

Check gate control rules for the downstream section of an accident pool: the check gate should be closed gradually depending on a water level control target.

3. Emergency regulation algorithm

3.1 Accident pool

The objective of gate emergency operations in the accident pool (**Figure 2**) is to confine the pollutants within the accident pool, and then the polluted water can be

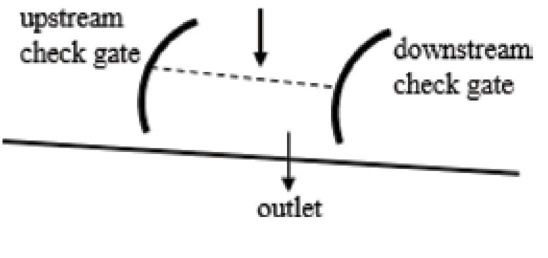

Figure 2.
A schematic of the accident pool.

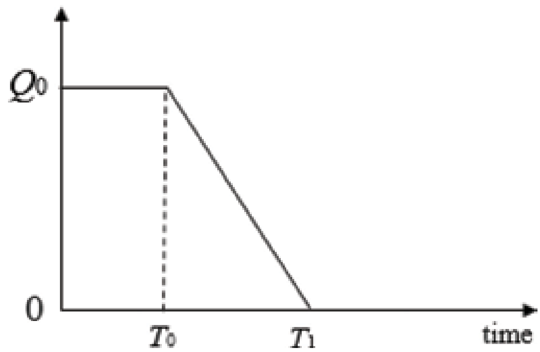

Figure 3.
The emergency operation of a gate.

Number	Mode	When the gate starts closing (T_0)	Closing time ($T_1 - T_0$)
1	Synchronous with the same speed	$T_{0up} = T_{0down}$	$(T_1 - T_0)_{up} = (T_1 - T_0)_{down}$
2	Asynchronous with the same speed	$T_{0up} \neq T_{0down}$	$(T_1 - T_0)_{up} = (T_1 - T_0)_{down}$
3	Synchronous with different speeds	$T_{0up} = T_{0down}$	$(T_1 - T_0)_{up} \neq (T_1 - T_0)_{down}$
4	Asynchronous with different speeds	$T_{0up} \neq T_{0down}$	$(T_1 - T_0)_{up} \neq (T_1 - T_0)_{down}$

Note: Up, upstream check gate; down, downstream check gate.

Table 1.
Closing modes of the two check gates and their features.

treated with other measures, for example, adsorption. In this situation, the discharge of the two gates in the initial and final state is known, but it remains unknown when the gate starts closing and how long it takes from the initial discharge to the final discharge (**Figure 3**). Therefore, there are four closing modes for the two check gates, as shown in **Table 1**.

3.2 Upstream section of accident pool

3.2.1 Adjustment of operation mode

3.2.1.1 Existing operation mode and its defects

Emergency Scheduling Plan for Sudden Events under Main Route of South to North Water Diversion Project (Q/NSBDZX G014-2014) points out that the regulation method of the upstream section of an accident pool should be performed according to the control principle of constant downstream depth; and that the opening of check gates should be adjusted real time depending on the actual water level.

Constant downstream depth operation is an operation mode to keep relative steady the downstream water depth of each channel section. When the flow in the channel section changes, the water surface profile will rotate around the pivot point at the downstream end of the channel section, as shown in **Figure 4**. Wedge volume between different steady flowing water surface profiles is formed therefrom. When

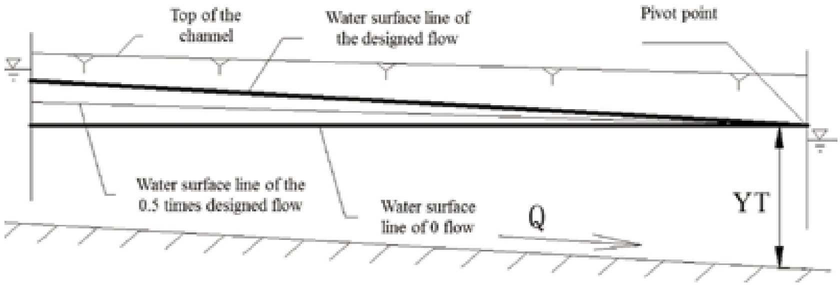

Figure 4.
Operation mode of constant downstream depth.

the flow increases, so do the water surface slope and wedge capacity volume; on the contrary, when it decreases, so do the water surface slope and wedge volume [4].

At present, constant downstream depth is set as the operation mode used for emergency regulation. This mode features high capacity of water transfer but poor response and recovery characteristics, so it needs to be improved further.

3.2.1.2 Feasibility analysis of using equal-volume operation mode under emergency regulation

On the basis of analysis of equal-volume operation mode features, this section demonstrates the feasibility of using equal-volume operation mode under emergency regulation in three steps: Step (1) determines the most unfavorable condition of equal-volume operation; Step (2) analyzes the free board margin; and Step (3) determines the flow interval suitable for equal-volume operation under the main route project.

3.2.1.2.1 Equal-volume operation mode and its features

Equal-volume control features "water retention capacity." When in equal-volume operation mode, the volume in a channel section keeps unchanged at any time. When flow transits from one steady state to another one, water surface rotates around the pivot point by the middle point of the channel section. Sometimes, the equal-volume operation mode is also referred to as "a synchronous operation mode," because this mode keeps a steady volume by synchronously controlling upstream and downstream check gates. The change of wedge volume in the channel section appears at both sides of the middle pivot point of the channel section, as shown in **Figure 5**. Providing a given flow change, the changes of wedge volume at

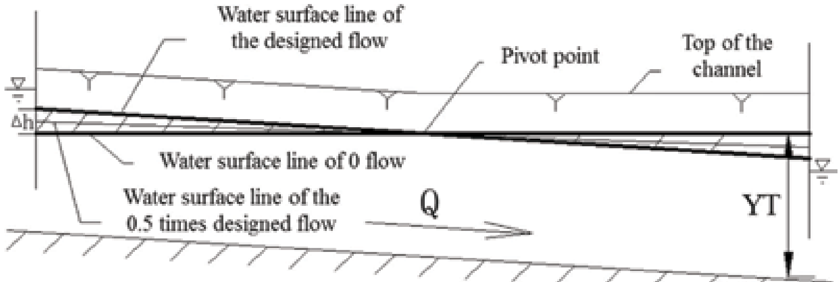

Figure 5.
Equal-volume operation mode.

the both sides of the pivot point equals and are different in direction. When the flow goes down, so does the upstream wedge volume, and the downstream wedge volume goes up; when the flow goes up, the upstream wedge volume goes up, and the downstream wedge volume goes down.

Advantages of equal-volume operation: the volume in the channel pool keeps unchanged during operating condition switching. Therefore, the equal-volume operation can change the state of water flow in the whole channel system rapidly. This is particularly suitable for operating conditions.

3.2.1.2.2 Judgment of the most unfavorable operating condition of equal-volume operation

When analyzing the feasibility of the equal-volume operation, identify and test the most unfavorable operating condition.

During normal water transfer in the MRP, constant downstream depth mode is used and changed to the equal-volume operation mode during an emergency. In order to ensure fast, smooth switching, water makeup from adjacent channel sections, or to ensure water discharge should not be performed before such switching. For this, the volume in a channel during equal-volume operation should be equal to that of constant downstream depth operation.

In the constant downstream depth mode, the water volume stored in the channel changes with the flow: the greater the flow is, the volume is more and the operation safety margin of the corresponding channel is smaller. Similarly, the safety margin at the time when this operating condition is switched to equal-volume operation is the smallest. Therefore, when equal-volume operation is under test, the volume should be the one with the design flow of constant downstream depth.

If volume is the same, the water surface profile of equal-volume operation is not unique, but rotates with flow change, among which the most unfavorable situation is the horizontal water surface profile corresponding to zero flow. This is because that the water surface profile at the downstream position of the channel section (i.e., the upstream water surface profile of downstream check gate) comes nearest to the top of a dike. For this reason, the test flow for equal-volume operation mode is zero.

Based on the above analyses, the most unfavorable operating condition of the equal-volume operation is that the flow is zero, with a volume equal to the one under the design flow of constant downstream depth. Because of subcritical flow in the open water conveyance canal, the average of the water surface profile is a backwater-type water surface profile. The corresponding upstream water level of check gate can be based on the corresponding relation among channel section flow, upstream water level of check gate, and volume and is determined by using a trial method.

3.2.1.2.3 Calculation of engineering safety margin for equal-volume operation

Under the most unfavorable operating condition of equal-volume operation, the rising maximum of the water level in all channel pools occurs upstream of check gate. First, by using a constant uniform flow program for calculating water surface profile, and via section integral, the volume V_0 is obtained according to the upstream water depth H_d of check gate corresponding to the design flow and the constant downstream depth of all channel pools. Second, take the volume in a channel pool, V_0 as a target and zero flow as a known condition, set different upstream water depths of check gate, and perform trial calculation by using a dichotomy until the respective H_{d1} is determined, that is, the upstream water depth

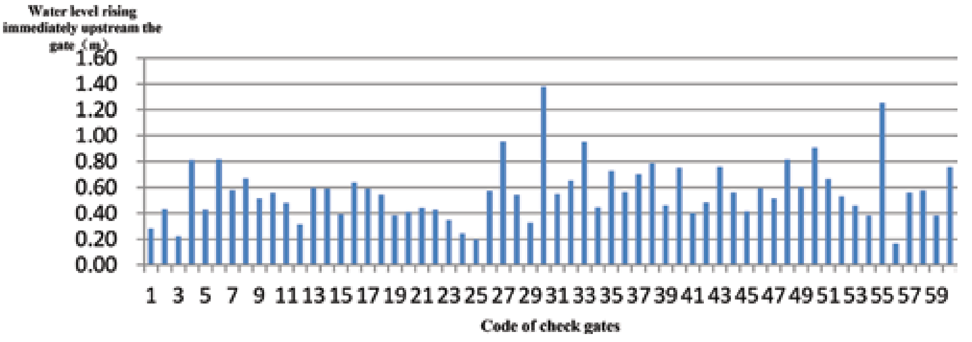

Figure 6.
Rising value of upstream water depth of check gate under equal-volume operation compared with constant downstream depth (the most unfavorable operating condition).

of check gate in each channel pool of the middle route under the most unfavorable operating condition of equal-volume operation. Calculate the water level rising value (**Figure 6**) and engineering safety margin under the most unfavorable operating condition of equal-volume operation (**Figure 7**).

From these figures, we can see that the first 25 channel pools have sufficient margin to implement equal-volume operation, while more channel pools at the right section have no sufficient margin.

3.2.2 Emergency regulation algorithm

Emergency regulation algorithm module can be divided into a feedforward control module and a feedback control module as a whole, as shown in **Figure 8**.

The feedforward control module is based on the water diversion quantity and volume of a channel before and after a channel accident; determine the operation plan for upstream check gates from the channel head to the accident section in combination of the plan for water diversion at dividing gate, in a bid to ensure safe, smooth transition between normal operation and emergency operation, reduce water abandoned by drainage gate as soon as possible, and cut the operation cost. The main decision-making basis for feedforward control module is overall water amount balance relation of a channel, without detailed calculation of factors such as

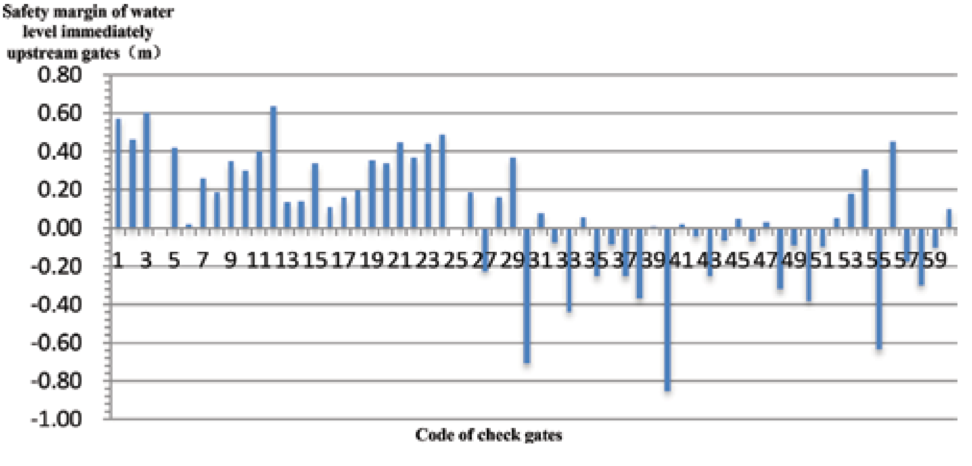

Figure 7.
Safety margin of upstream water depth of check gate under equal-volume operation (the most unfavorable operating condition).

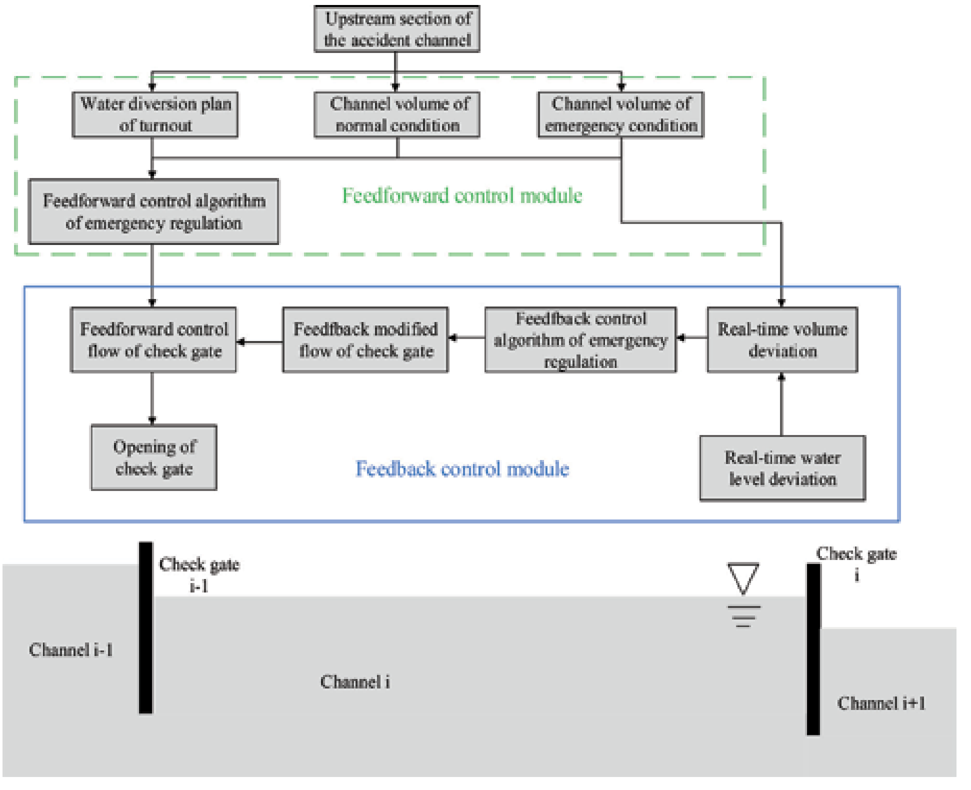

Figure 8.
Construction of emergency integrated control module at upstream of accident section of the middle route project.

check gate mechanical dead band, channel hydraulic model, sensor measuring error, etc. The purpose is "rough regulation."

The purpose of the feedback control module is to eliminate the water level control deviation arising from various types of perturb factors during a whole transition. By using a water level sensor, the module performs real-time measurement of the deviation in the upstream water level of check gate and converts it to a volume deviation; after that, it realizes the correction of feedforward control flow of check gate by changing the flow of water entering the channel, maintaining a target water level within a preset reasonable range.

3.2.2.1 Feedforward control

The channel adopts conventional constant downstream depth operation mode, constant downstream depth, and equal-volume operation mode. The emergency integrated feedforward control module differs, so do the respective tradition process and control effect. This research carried out calculation, analysis, and comparison depending on different operation modes of the middle route project.

3.2.2.1.1 Constant downstream depth

For easy explanation, assume that there is a generalized canal with three channel pools in series as shown in **Figure 9**, and that the accident section lies at the most downstream. In the figure, DV1–DV3 stand for the decrease of the volume in each channel pool before and after emergency control (i.e., normal

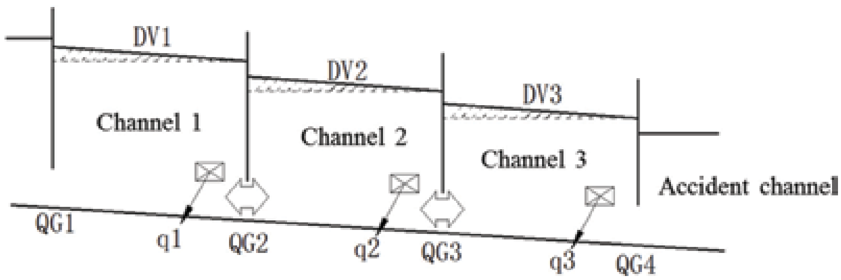

Figure 9.
Channels emergency regulated and controlled by using constant downstream depth operation.

operation and emergency operation); QG1–QG4, the flow of inflow or outflow check gate of each channel pool; q1–q3, the flow at the dividing gate of each channel pool.

On the basis of overall operation strategies and measures of the MRP, an integrated feedforward control module is designed in the constant downstream depth operation mode of the whole route. The detailed process description is shown in **Figure 10**.

1. After an accident occurs, rapidly close all check gates at the upstream accident section at the first time; if the water level is above the warning level, open the drainage gate until the water level falls back to the design level.

2. Determine a new target flow after emergency operations of inflow of each channel pool. Depending on the balance relation between inflow and outflow of channel pool, from the adjacent channel pools at the upstream accident section, accumulate the flow of all downstream turnouts to the upstream direction, taking the total flow as a new target flow after emergency operations of inflow check gate of this channel pool.

3. Calculate the regulation target value of volume in each channel pool.
   (1) Calculate the regulation target value of volume before and after emergency response of single channel pool. According to a new target flow after emergency operations of inflow check gate of each channel pool, calculate the volumes before and after emergency operations of each channel pool, respectively, by using a calculation program for constant nonuniform water surface profile of conventional open channel; the regulation target value equals to the difference of the two volumes; (2) calculate the regulation target value of accumulated volume of channel pools in series. Starting from adjacent channel pools at the upstream accident section to channel head, accumulate the regulation target value of single channel pool volume, channel pool by channel pool, and then obtain the regulation target value of accumulated volume of channel pools in series, and take it as the target volume for emergency operations of each channel pool.

4. Calculate the time when inflow of each channel pool is reduced. (1) Starting from the upstream channel section adjacent to accident section to upstream direction, accumulate the flow of dividing gates channel by channel, and then obtain the accumulated flow for emergency operations of each channel pool. (2) Divide the target volume for emergency operations of each channel pool by the accumulated flow for emergency operations of each channel pool, and then we obtain the change transient time of emergency operation volume in each channel pool.

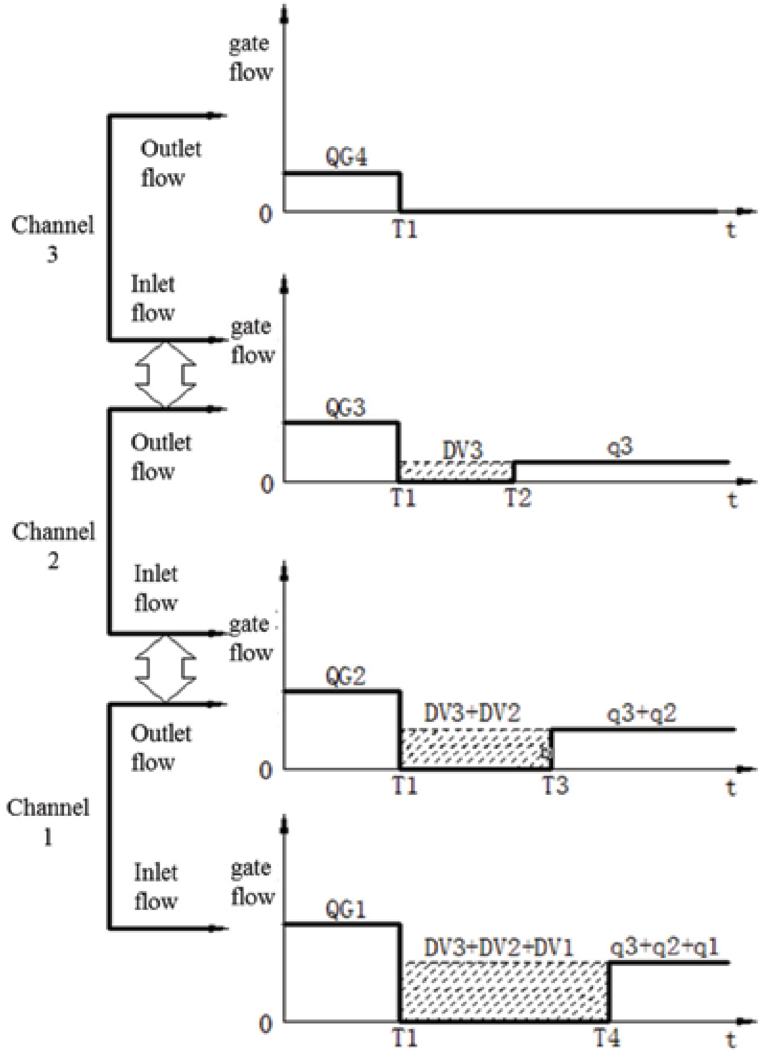

Figure 10.
Schematic diagram of emergency control mode by using constant downstream depth operation mode.

5. Carry forward with time and make implementation, respectively, according to the new target flow after emergency operations of inflow check gate of channel pool determined in Step (3), as well as the time when inflow of each channel pool is reduced (determined in Step (4)).

3.2.2.1.2 Constant downstream depth + equal-volume operation

For easy derivation and explanation, assume that there is a generalized canal with three channel pools in series as shown in **Figure 11**, and that channel pools 1 and 2 use the equal-volume operation mode. In the figure, DV1–DV3 stand for the decrease of the volume in each channel pool before and after emergency control; QG1–QG4, the flow of inflow or outflow check gate of each channel pool; q1–q3, the flow at the dividing gate of each channel pool.

On the basis of overall operation strategies and measures of the MRP, integrated feedforward control module is proposed by combining constant downstream depth

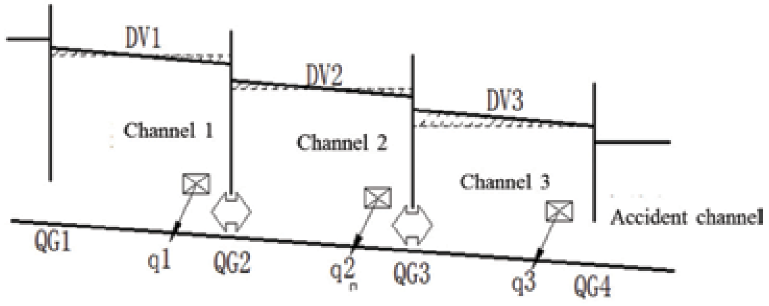

Figure 11.
Channels emergency regulated and controlled by using equal-volume operation.

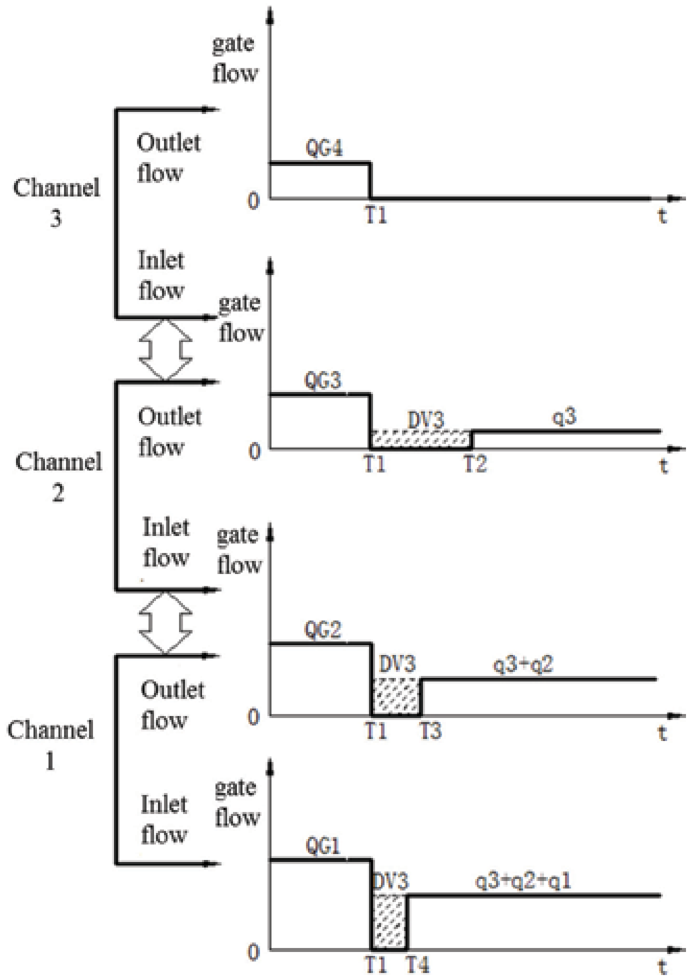

Figure 12.
Schematic diagram of emergency channel integrated control mode by using constant downstream depth operation mode.

and equal-volume operation mode. For the description of the detailed process, refer to **Figure 12**:

1. After an accident occurs, rapidly close all check gates at the upstream accident section at the first time. If the water level is above the warning level, open the drainage gate until the water level falls back to the design level.

2. Determine a new target flow after emergency operations of inflow check gate of each channel pool. Depending on the balance relation between inflow and outflow of channel pool, accumulate the flow of all downstream dividing gates channel section by channel section from the adjacent channel pools at the upstream accident section to the upstream direction, taking the total flow as a new target flow after emergency operations of inflow check gate of this channel pool.

3. Calculate the regulation target value of volume in each channel pool. (1) Calculate the regulation target value of volume before and after emergency response of single channel pool. For channel pools using equal-volume operation, let the regulation target value of volume in a single channel pool equal to 0. By adopting the channel pool using constant downstream depth operation and according to a new target flow after emergency operations of inflow check gate of each channel pool, calculate the volumes before and after emergency operations of each channel pool, respectively, by using a calculation program for constant nonuniform water surface profile of conventional open channel; the regulation target value equals to the difference of the two volumes; (2) calculate the regulation target value of accumulated volume of channel pools in series. Starting from adjacent channel pools at the upstream accident section to channel head, accumulate the regulation target value of single channel pool volume, channel pool by channel pool, and then obtain the regulation target value of accumulated volume of channel pools in series, and take it as the target volume for emergency operations of each channel pool.

4. Calculate the time when inflow of each channel pool is reduced. (1) Starting from the upstream channel section adjacent to accident section to upstream direction, accumulate the flow of dividing gates channel by channel, and then obtain the accumulated flow for emergency operations of each channel pool. (2) Divide the target volume for emergency operations of each channel pool by the accumulated flow for emergency operations of each channel pool, and then we obtain the change transient time of emergency operation volume in each channel pool.

5. Carry forward with time and make implementation, respectively, according to the new target flow after emergency operations of inflow check gate of channel pool determined in Step (3), as well as the time when inflow of each channel pool is reduced (determined in Step (4)).

3.2.2.2 Feedback control

Feedback control is also called closed-loop control. It adjusts the input quantity of a controlled object by measuring the deviation between the state of the controlled object and the target state and makes the state of the controlled object meet actual demands. During the design of feedback control algorithm, perform in-depth analysis of the dynamic response characteristics of the controlled object, and then on such basis, design the control algorithm and calibration control parameters in order to improve the performance of the controller as much as possible. For example, reduce the response process of the controlled object and the fluctuation range of the controlled parameters, improve the stability of the controlled object, and so on.

In the field of channel control, PID feedback control algorithm is the algorithm that is applied most. This algorithm is characterized by simple structure, good stability, reliable operation, convenient adjustment, etc. [5]. The dynamic response characteristics of PID control algorithm are closely related with control parameters of the control algorithm. These parameters will change with time because of uncertainty of channel roughness, cross-section size, check gate overflow coefficient, etc., and for sediment deposition, aquatic weed growth, and other reasons. Therefore, the setting of controller parameters is very critical to control performance and safe operation of a water conveyance system. There is a lot of work to do for such setting. At present, there are three kinds of commonly used methods: theoretical analysis, empirical trial method, and on-line setting method.

In order to reduce controller setting workload, this research proposes a feedback control algorithm based on dynamic volume correction thinking. The core of the algorithm is real-time dynamic correction of volume in each channel pool to make the volume of the channel pool the same with that at the target water level. For the principle, see **Figure 13**. The specific process of implementation of dynamic volume feedback correction is as follows:

1. Perform real-time monitoring of upstream target water level of check gate $Z_{\text{up}}(i,t)$. If the deviation of the water level from the target value is more than water level dead band, then the feedback control algorithm for correcting channel pool volume will be triggered.

2. Calculate the real-time volume deviation ΔV by using the following method: calculate constant water surface profile depending on the current upstream water level and target value of check gate, respectively, determine the current volume of the channel pool and its target volume via integral, and then we obtain the real-time volume deviation ΔV via subtraction between the two volumes.

3. Calculate volume correction process parameters including volume correction duration $\Delta\tau$ and respective correction flow ΔQ_{volume}. See Formula (1). For the determination of ΔQ_{volume}, we should consider multiple constraint conditions. For example, respective flow passing through check gate should be more than 0, but less than the design flow of channel, etc.

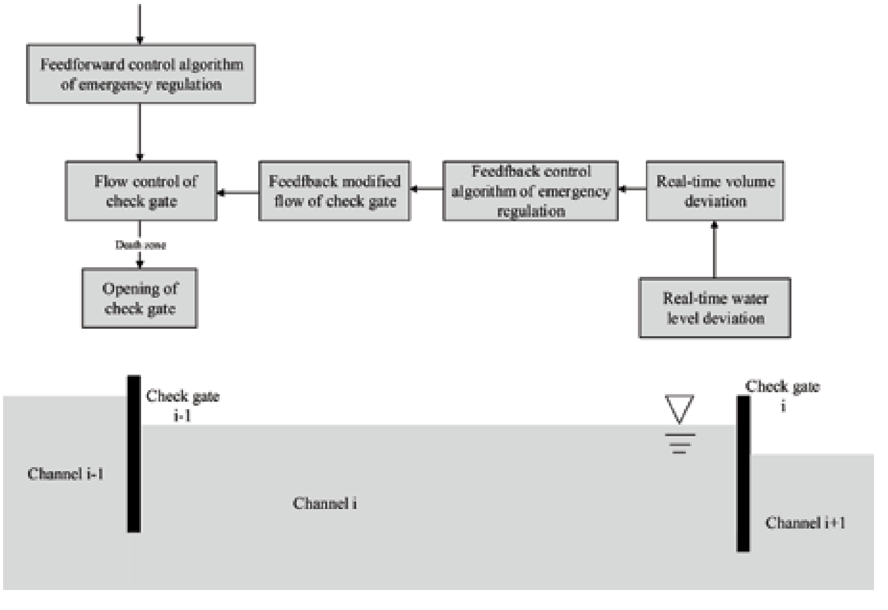

Figure 13.
Schematic diagram of emergency feedback control of upstream accident section.

$$\Delta Q_{\text{volume}}(i,t) = \Delta V(i)/\Delta\tau(i) \tag{1}$$

where $i = 1 \sim N$, and N is the number of channel pools; ΔQ_{volume} is the volume compensation flow; ΔV is the volume variation during volume compensation; $\Delta\tau(i)$ is the duration used by the volume compensation process of channel pool i.

4. The action that check gate implements volume correction for the corrected control flow of check gate $Q_{\text{gate}}(i,t)$, see Formula (2). For reverse calculation of check gate opening $Gate(i,t)$ by using check gate overflow formula, see Formula (3) and (4), where $Z_{\text{up}}(i,t)$ and $Z_{\text{down}}(i,t)$ are the upstream and downstream water levels of check gate, respectively. Compare them with check gate motion dead band (DB) and then determine whether to output check gate action (see Formulas (4) and (5)).

$$Q_{\text{gate}}(i,t) = \Delta Q_{volume}(i,t) + Q_0(i,t) \tag{2}$$

$$Gate(i,t) = \text{f}^{-1}\left(Q_{\text{gate}}(i,t),\ Z_{\text{up}}(i,t),\ Z_{\text{down}}(i,t)\right) \tag{3}$$

$$DG(i,t) = Gate(i,t) - Gate(i,t-1) \tag{4}$$

$$Gate(i,t) = \begin{cases} Gate(i,t) + DG(i,t) & if \;\; DG(i,t) \geq DB \\ 0 & if \;\; DG(i,t) < DB \end{cases} \tag{5}$$

5. Following the completion of this volume correction, go back to Step (1) and realize dynamic rolling correction.

It is important to note that in order to speed up the transition process of volume feedback regulation, the action of volume correction of each check gate (refers to ΔQ_{volume}) should be passed to all upstream check gates so as to make all upstream check gates operate synchronously, jointly completing volume feedback regulation.

For constant downstream depth and equal-volume operation modes, the thinking for implementing the aforesaid feedback control algorithms is basically the same, and the main difference lies in presetting of the upstream target water level of check gate. The upstream target water level of check gate in constant downstream depth operation mode is its design value and keeps unchanged all the time, while the upstream target water level of check gate in equal-volume operation mode changes with flow dynamically. In this research, the upstream target water level of check gate changes linearly with the whole transition process in a preset equal-volume operation mode. That is to say that the upstream target water level of check gate changes linearly from the initial value to the final value during the whole time interval from the occurrence of a sudden event to the completion of feedforward control and regulation.

3.3 Downstream section of accident pool

3.3.1 Demands for operation

During the emergency operations of downstream section of an accident pool, it is considered that the flows passing through upstream and downstream check gates of all channel pools reduce linearly to 0 from normal operation condition of the project (**Figure 14**).

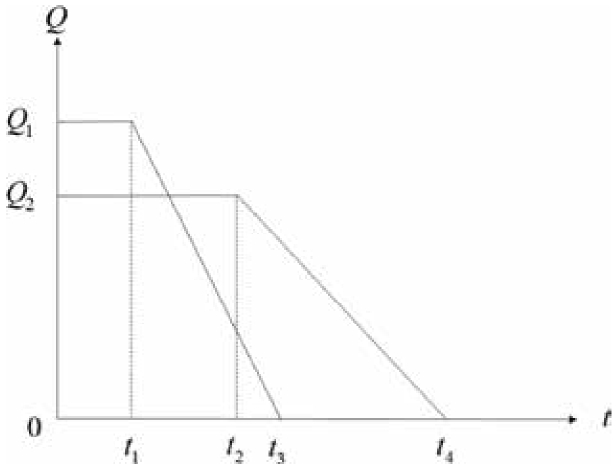

Figure 14.
Upstream and downstream flow change processes of any channel pool at downstream accident pool.

In the figure, Q_1 is the water discharge through check gate corresponding to upstream check gate under normal operation condition of the current channel pool; Q_2 is the water discharge through check gate corresponding to downstream check gate under normal operation condition of the current channel pool; t_1 is the time when the regulation of upstream check gate starts; t_2 is the time when the regulation of downstream check gate starts; t_3 is the time when the regulation of upstream check gate ends; and t_4 is the time when the regulation of downstream check gate ends.

All check gates of downstream accident pool take the downstream check gate as a boundary condition, and emergency operations is carried out. Therefore, during this process, the aforesaid t_1 and t_3 can be taken as known parameters, t_2 and t_4 as unknown parameters, and then they can be converted to the determination of the start time t_2 and end time t_4 of regulation of downstream check gate.

Therefore, the operation of downstream accident section generally is similar to volume control algorithm. The purpose of water level control is realized by regulating channel pool volume [6]. At present, we have obtained the operation strategy analysis of downstream check gate after an accident occurs upstream. The operation process is that the opening of check gate is performed according to conventional regulation opening at each step [7]. Actually, the operation of check gate under an emergency may not follow conventional opening change constraint, but be the maximum opening. Therefore, for the operation strategy of downstream accident section, an equation can be established according to water budget with a known change ΔV of water body volume in channel pool. With another condition added, t_2 and t_4 can be determined by combining the equation and the condition. This chapter puts forward three methods for reference.

3.3.2 Methods for operation

3.3.2.1 Equal proportion reduction of water discharge through check gate

The water discharge through all check gates at downstream accident pool reduces in an equal proportion, and the reduction of discharge is the same per unit time, as shown in **Figure 15**.

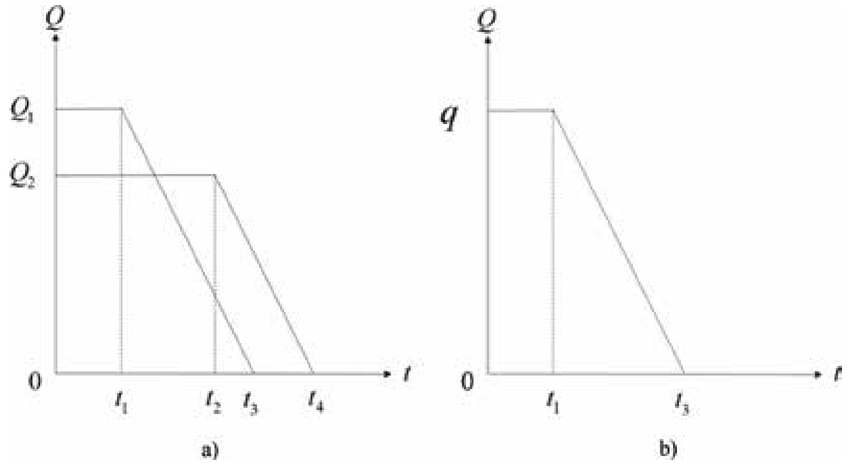

Figure 15.
Schematic diagram of equal proportion reduction of water discharge through check gate. (a) Upstream and downstream flow change processes of channel pool. (b) Flow change process at the turnout of such channel pool.

By using this method, we obtain

$$\frac{Q_2}{t_4 - t_2} = \frac{Q_1}{t_3 - t_1} \tag{6}$$

During emergency operations, the volume of water body flowing into such channel pool through upstream check gate is

$$\Delta V_u = Q_1 t_1 + \frac{1}{2}(t_3 - t_1)Q_1 \tag{7}$$

During such process, the volume of water body flowing out of such channel pool is

$$\Delta V_d = Q_2 t_2 + \frac{1}{2}(t_4 - t_2)Q_2 + q t_1 + \frac{1}{2}(t_3 - t_1)q \tag{8}$$

then, the volume change ΔV of water body in the channel pool is

$$\Delta V = \Delta V_d - \Delta V_u \tag{9}$$

Put Formulas (7) and (8) into Formulas (9), and then we obtain

$$2\Delta V = (t_2 + t_4)Q_2 + (t_1 + t_3)q - (t_1 + t_3)Q_1 \tag{10}$$

From water budget, we know that

$$Q_1 = Q_2 + q \tag{11}$$

Put Formula (11) into Formula (10), and then we obtain

$$\frac{2\Delta \mathrm{V}}{Q_2} = t_2 + t_4 - t_1 - t_3 \tag{12}$$

By combining Formulas (4)–(6) and (4)–(12), we can obtain

$$t_2 = \frac{\Delta}{VQ_2} + \frac{t_1 + t_3}{2} - \frac{Q_2}{2Q_1}(t_3 - t_1) \tag{13}$$

$$t_4 = \frac{Q_2}{2Q_1}(t_3 - t_1) + \frac{\Delta V}{Q_2} + \frac{t_1 + t_3}{2} \tag{14}$$

Because $Q_1 \geq Q_2$, we can obtain from Formula (15)

$$t_2 \geq \frac{\Delta V}{Q_2} + t_1 \tag{15}$$

Namely, the regulation start time of downstream check gate is later than that of upstream check gate.

3.3.2.2 Starting regulation of all check gates at the same time

The regulation of check gate of all channel pools of downstream accident pool and the check gate of downstream accident pool starts at the same time, but ends at different times, as shown in **Figure 16**.

By using this method, we know that the regulation start time of each check gate t_2 is the same, equaling to that of downstream check gate of accident pool. Then, there is only one unknown parameter t_4 under such condition.

During emergency operations, the volume of water body flowing into channel pool through upstream check gate is

$$\Delta V_u = \frac{1}{2} t_3 Q_1 \tag{16}$$

During such process, the volume of water body flowing out of such channel pool is

$$\Delta V_d = \frac{1}{2} t_4 Q_2 + \frac{1}{2} t_3 q \tag{17}$$

then, the volume change ΔV of water body in the channel pool is

$$\Delta V = \Delta V_d - \Delta V_u \tag{18}$$

Put Formulas (16) and (17) into (18), and then we obtain

$$2\Delta V = t_4 Q_2 + t_3 q - t_3 Q_1 \tag{19}$$

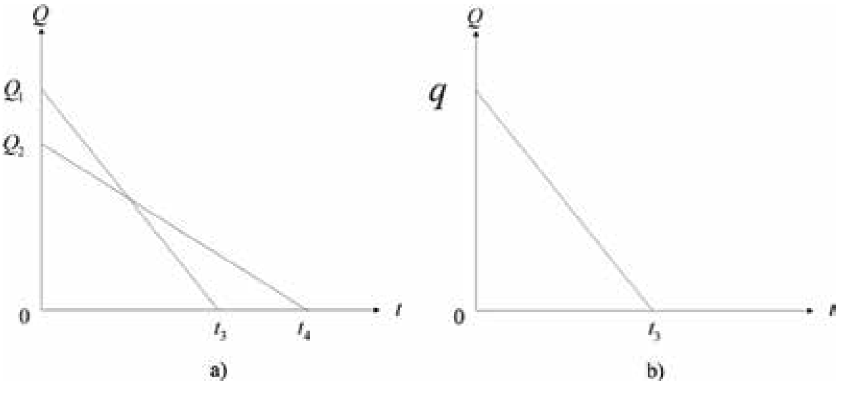

Figure 16.
Schematic diagram of equal proportion reduction of water discharge through check gate. (a) Upstream and downstream flow change processes of channel pool. (b) Flow change process at the dividing gate of such channel pool.

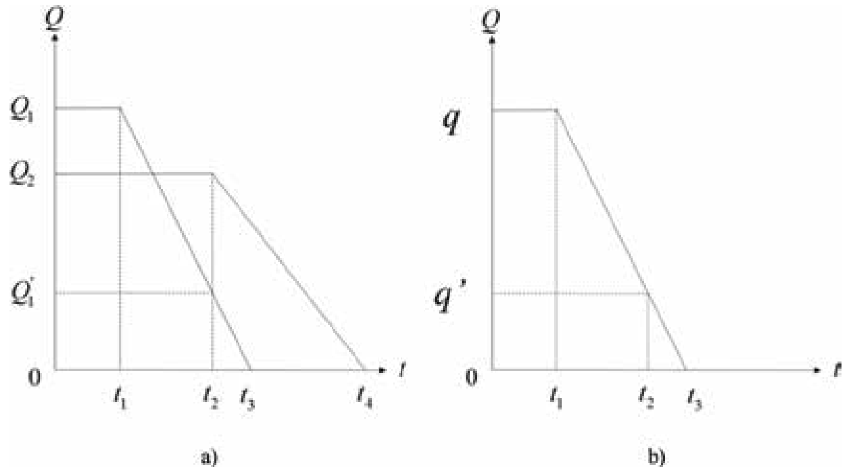

Figure 17.
Schematic diagram of equal proportion reduction of water discharge through check gate. (a) Upstream and downstream flow change processes of channel pool. (b) Flow change process at the dividing gate of such channel pool.

From water budget, we know that

$$Q_1 = Q_2 + q \tag{20}$$

Put Formula (20) into Formula (19), and then we obtain

$$t_4 = \frac{2\Delta V}{Q_2} + t_3 \tag{21}$$

From this Formula, we can see that the regulation end time of downstream check gate is always later than that of upstream check gate.

3.3.2.3 Performing control depending on change in water body volume in channel pool

The regulation start time of downstream check gate of the channel pool is controlled via the change of water body volume in the channel pool. In this chapter, assuming when the water body volume reduces $\frac{\Delta V}{n}$ (n is an integer), the regulation of downstream check gate starts. The operation process of this method is as shown in **Figure 17**.

In the figure, the water discharge through upstream check gate of channel pool corresponding to time t_2 is $Q_1^{'}$, and the water diversion at dividing gate is q, then we obtain

$$\frac{Q_1^{'}}{t_3 - t_2} = \frac{Q_1}{t_3 - t_1} \tag{22}$$

$$\frac{q^{'}}{t_3 - t_2} = \frac{q}{t_3 - t_1} \tag{23}$$

By time t_2, the volume of water entering the channel pool is

$$\Delta V_u = Q_1 t_1 + \frac{Q_1^{'} + Q_1}{2}(t_2 - t_1) \tag{24}$$

The volume of water body flowing out of channel pool is

$$\Delta V_d = Q_2 t_2 + q t_1 + \frac{q' + q}{2}(t_2 - t_1) \tag{25}$$

As described above, by time t_2, the change of water body volume in channel pool is

$$\frac{\Delta V}{n} = \Delta V_d - \Delta V_u \tag{26}$$

From water budget, we know that

$$Q_1 = Q_2 + q \tag{27}$$

Put Formulas (22)–(25), and (27) into (26), and then we obtain

$$\frac{2\Delta V}{nQ_2} = \frac{(t_2 - t_1)^2}{t_3 - t_1} \tag{28}$$

We obtain the following solution:

$$t_2 = t_1 + \sqrt{\frac{2\Delta V}{nQ_2}(t_3 - t_1)} \tag{29}$$

By time t_2, the volume of water entering the channel pool is

$$\Delta V_u' = \frac{Q_1'}{2}(t_3 - t_2) \tag{30}$$

The volume of water body flowing out of channel pool is

$$\Delta V_d' = \frac{Q_2}{2}(t_4 - t_2) + \frac{q'}{2}(t_3 - t_2) \tag{31}$$

As described above, after time t_2 and until completion of emergency operations, the change of water body volume in channel pool is

$$\frac{(n-1)\Delta V}{n} = \Delta V_d' - \Delta V_u' \tag{32}$$

Put Formulas (22), (23), (27), (30), and (31) into (32), and then we obtain

$$t_4 = \frac{2(n-1)\Delta V}{nQ_2} + \frac{(t_3 - t_2)^2}{t_3 - t_1} + t_2 \tag{33}$$

From Formula (29), we know that the regulation start time of downstream check gate t_2 is certainly later than that of upstream check gate t_1. Since the above derivation processes is obtained under the condition of $t_2 \le t_3$, then by combining Formula (29), we obtain

$$t_3 - t_1 - \sqrt{\frac{2\Delta V}{nQ_2}(t_3 - t_1)} \ge 0 \tag{34}$$

We obtain the following via solution

$$\frac{\Delta V}{nQ_2} \le \frac{t_3 - t_1}{2} \tag{35}$$

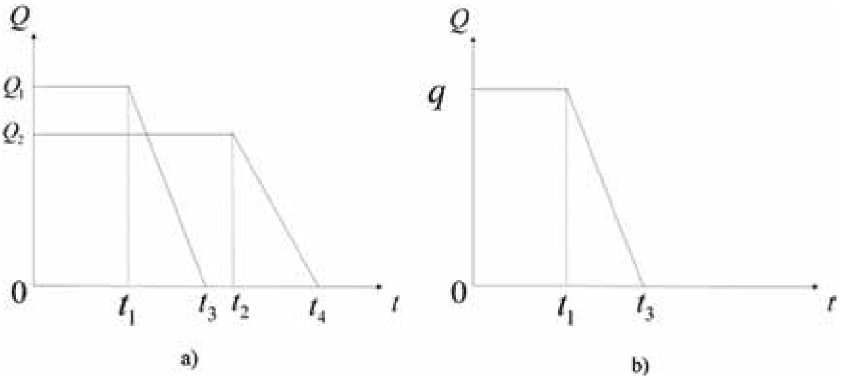

Figure 18.
Schematic diagram of equal proportion reduction of water discharge through check gate. (a) Upstream and downstream flow change processes of channel pool. (b) Flow change process at the dividing gate of such channel pool.

However, at $t_2 \geq t_3$, the operation method is shown in **Figure 18**.
Then by time t_2, the volume of water entering the channel pool is

$$\Delta V_u = \frac{Q_1}{2}(t_1 + t_3) \tag{36}$$

The volume of water body flowing out of channel pool is

$$\Delta V_d = Q_2 t_2 + q\frac{t_1 + t_3}{2} \tag{37}$$

By time t_2, the change of water body volume in channel pool is

$$\frac{\Delta V}{n} = \Delta V_d - \Delta V_u \tag{38}$$

From water budget, we know that

$$Q_1 = Q_2 + q \tag{39}$$

Put Formulas (36), (37), and (39) into (38), and then we obtain

$$t_2 = \frac{\Delta V}{nQ_2} + \frac{t_1 + t_3}{2} \tag{40}$$

After time t_2, no water flows into the channel, and then the volume of water flowing out of channel is

$$\Delta V_d' = Q_2 \frac{t_4 - t_2}{2} \tag{41}$$

Then we obtain

$$\frac{(n-1)\Delta V}{n} = \Delta V_d' \tag{42}$$

Put Formulas (40) and (41) into (42), and then we obtain

$$t_4 = \frac{(2n-1)\Delta V}{nQ_2} + \frac{t_1 + t_3}{2} \tag{43}$$

For this method, because the regulation start time of downstream check gate t_2 is later than the regulation end time of upstream check gate t_3, namely $t_2 \geq t_3$, then by combining Formula (40), we obtain

$$\frac{\Delta V}{nQ_2} + \frac{t_1 + t_3}{2} - t_3 \geq 0 \tag{44}$$

We obtain the following by solution

$$\frac{\Delta V}{nQ_2} \geq \frac{t_3 - t_1}{2} \tag{45}$$

From Formulas (44) and (45), we can see that for this method, the specific form of the operation method which takes volume as a control condition should be determined by comparing the magnitude relation between $\frac{\Delta V}{nQ_2}$ and $\frac{t_3 - t_1}{2}$.

4. Case study

Take the MRP for example. Assuming that 6th channel pool (from check gate for Shierli River to the check gate at the outlet of the inverted siphon for Baihe River) undergoes a sudden water pollution event, the control rules and algorithms proposed herein should be used immediately to control the check gates at accident section, and upstream and downstream sections of accident pool, to make emergency operations process realize rapidly and to ensure that water level thereafter reaches the target water level. In case of an accident under a simulated operating condition, the water diversion flow at the accident section is 126 m^3/s.

Firstly, in order to test the effectiveness of the algorithm herein, assuming that current control is constant downstream depth operation mode, analyze whether the steady water level after control completion satisfies requirements by using emergency control algorithm adopted herein. Secondly, in order to compare the operation results of constant downstream depth and constant downstream depth + equal-volume operation modes, further assuming that the constant downstream depth operation mode is used for downstream sections of accident pool, and that equal-volume control mode is used for some pools of the upstream section of polluted channel section, here the upstream 2-5 pools were assumed to be under equal-volume control mode while upstream 1 pool was assumed to be under constant water level control mode (the farther away the pool is from the accident section, the larger the pool is numbered), then compare the results of upstream some sections of accident pool obtained by using constant downstream depth and constant downstream depth + equal-volume operation modes respectively.

In constant downstream depth operation mode, see **Figures 19–25** for the operation process of accident pool, and upstream and downstream sections of accident pool.

4.1 Accident pool

Two check gates in accident pool were closed rapidly within 60 min (**Figure 19**), leading to violent water oscillation therein. Below we take the fluctuation process and initial water level deviation of upstream water level of check gate for example for display (**Figure 20**).

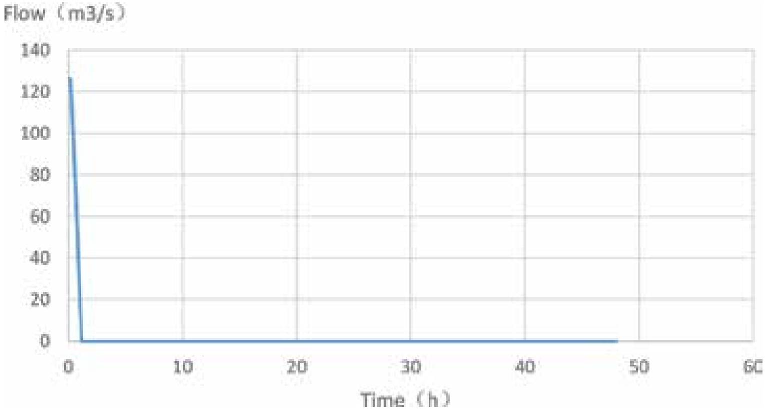

Figure 19.
Change process diagram of water discharge through check gate in accident pool.

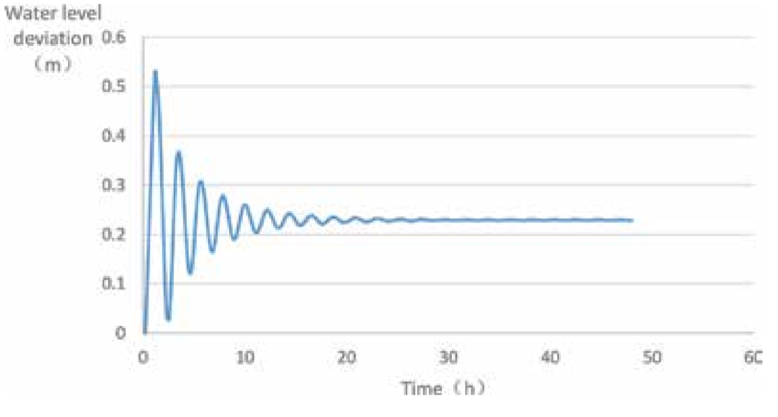

Figure 20.
Deviation change process diagram of upstream water level of downstream check gate in accident pool.

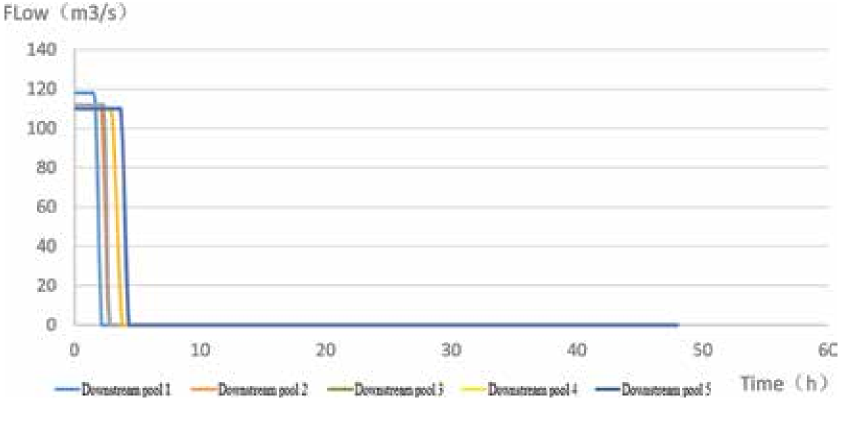

Figure 21.
Change process diagram of water discharge through check gate at downstream section of accident pool.

4.2 Downstream section of accident pool

For downstream section of accident pool, select the operation process of water discharge through five downstream check gates near to polluted channel section

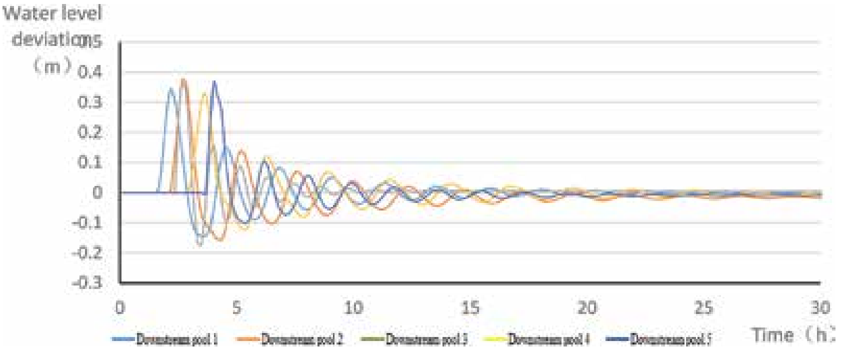

Figure 22.
Deviation change process diagram of upstream water level of check gate at downstream section in accident pool.

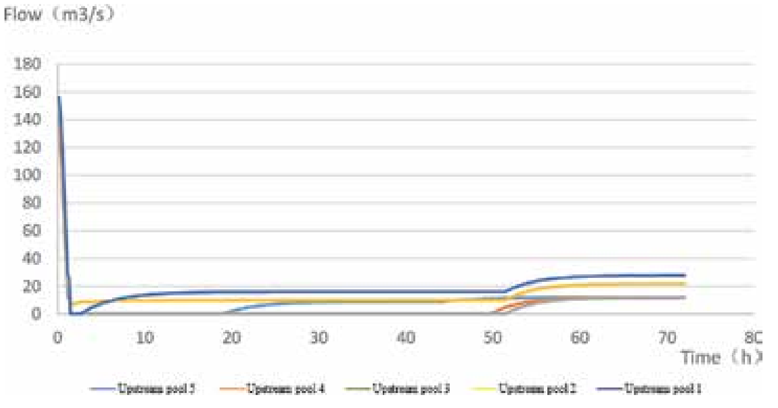

Figure 23.
Change process diagram of water discharge through check gate at upstream section of accident pool.

and the change process of upstream water level of check gate, as shown in **Figures 21** and **22**.

From **Figure 22**, we can see that the water level in accident pool meets constant downstream depth after stability. This is because that the water diversion flow change process at the downstream section was artificially determined and that the most upstream check gate at the downstream section was closed first. The feedforward analysis can be calculated by using the principle of volume compensation algorithm. And also, we can see that the fluctuation of the upstream water level of check gate is small, not more than 0.4 m. This is because that asynchronous closing operation was adopted at downstream section. This is reflected by the time when the change of water discharge starts in **Figure 21**.

4.3 Upstream section of accident pool

4.3.1 Constant downstream depth

For upstream section of accident pool, select the operation process of water discharge through five downstream check gates near to accident pool and the change process of the upstream water level of check gate, as shown in **Figures 23** and **24**.

From **Figure 24**, we can see that the water level at upstream section in accident pool meets constant downstream depth after stability. This is because feedback

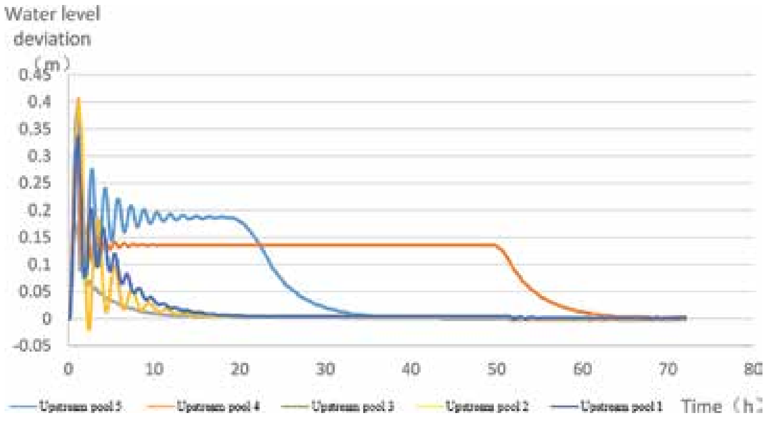

Figure 24.
Deviation change process diagram of upstream water level of check gate at upstream section in accident pool.

algorithm was used for the upstream section. Gradually regulate the flow of check gate to make channel pool volume come near to the target volume, making the upstream water level of check gate come near to the target water level. And from **Figure 23**, we can see that the synchronous closing method was used for check gate at upstream section. After check gate was closed completely for a period of time and then opened, the opening time was different. After stability, keep a certain flow unchanged to meet normal water supply at the upstream section. Because of nonuniform water discharge through check gate, discharge difference was used to regulate channel pool volume and to realize the control of the upstream water level of check gate.

4.3.2 Constant downstream depth + equal-volume

In constant downstream depth and equal-volume check gate adding constant downstream depth operation modes, the operation result of channel section is shown in **Figures 25–34**.

Figures 25 and **26** are the results of upstream check gate 1. From these figures, we can see that the results under two operating conditions are the same. This is because that the channel section corresponding to upstream check gate 1 was near

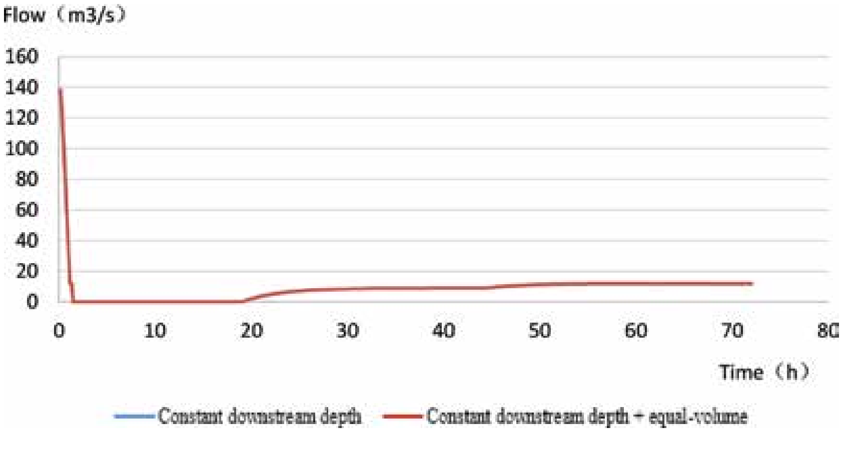

Figure 25.
Flow change of upstream check gate 1 under different operation conditions.

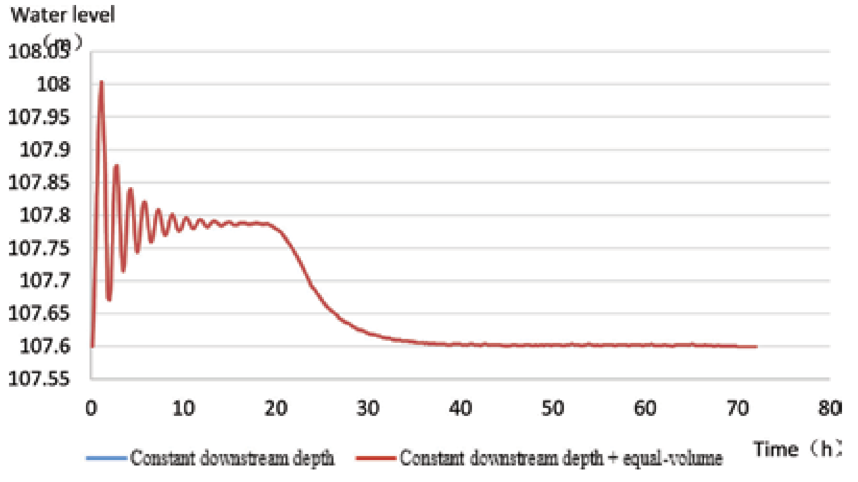

Figure 26.
Flow change of upstream water level of upstream check gate 1 under different operation conditions.

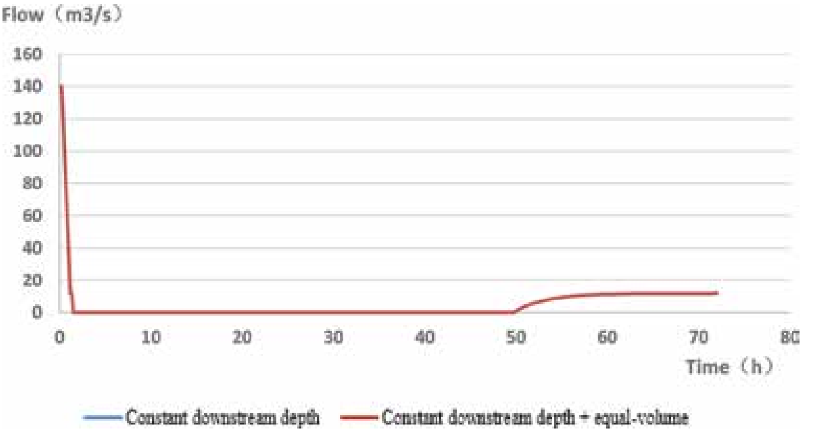

Figure 27.
Flow change of upstream check gate 2 under different operation conditions.

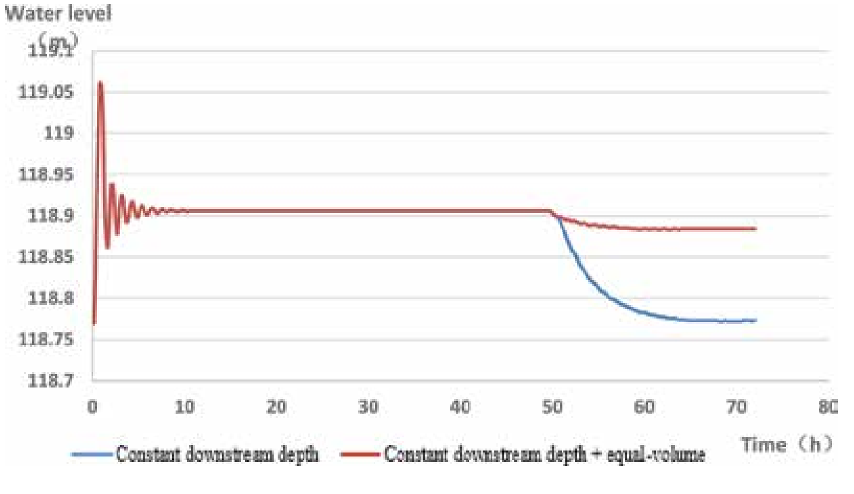

Figure 28.
Flow change of upstream water level of upstream check gate 2 under different operation conditions.

to downstream and constant downstream depth was adopted under such two operating conditions.

Figures 27 and **28** are the results of upstream check gate 2. From **Figure 28**, we can see the difference in the upstream water level of check gate under two operating conditions: steady upstream water level of check gate under equal-volume control is higher than the upstream water level in constant downstream depth operation mode. But from **Figure 27**, we cannot see the difference in opening and flow of check gate. Therefore, the reason that leads to nonuniform water level lies in nonuniform opening change of upstream check gate which is not reflected therein.

Figures 29–34 are the results of upstream check gates 3, 4, and 5 under two operating conditions. After comparing the upstream water level, water discharge and opening of check gate, respectively, we can find out the difference between the two operating conditions. In equal-volume operation mode, all upstream water levels of check gate after stability are higher than that in constant downstream depth operation mode. Under such condition, the water level drop rate of check gate will be smaller, and it is more favorable to the safety of the project. In equal-volume operation mode, check gate is opened at an earlier time, so the emergency regulation time corresponding thereto becomes shorter. Moreover, this situation is

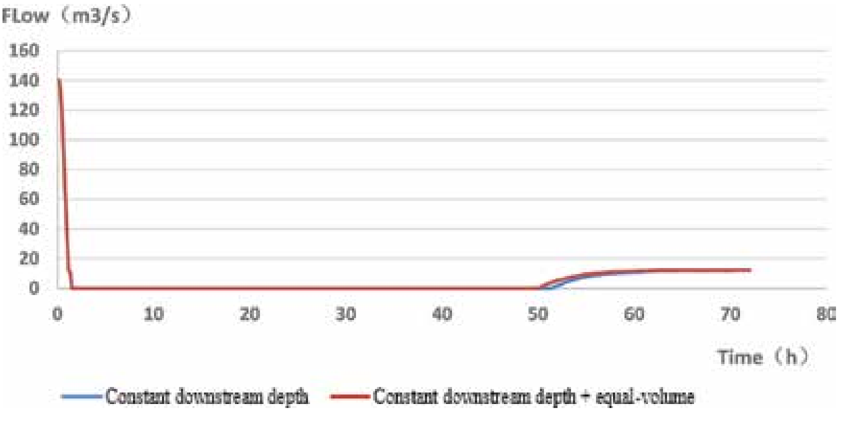

Figure 29.
Flow change of upstream check gate 3 under different operation conditions.

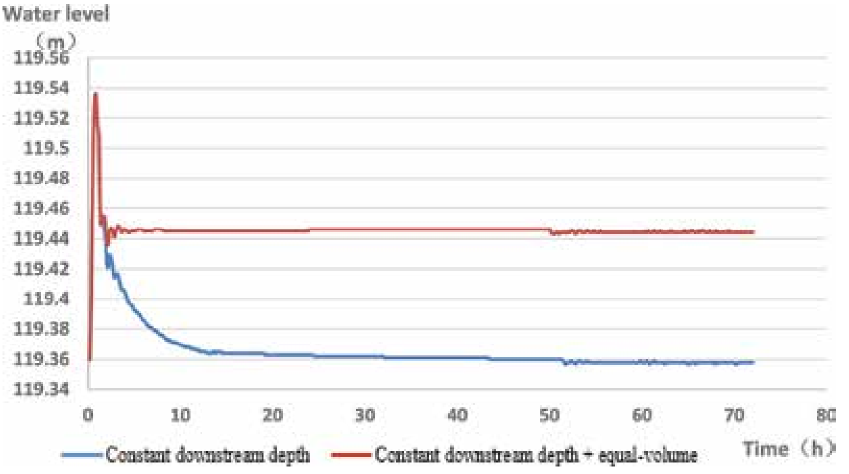

Figure 30.
Flow change of upstream water level of upstream check gate 3 under different operation conditions.

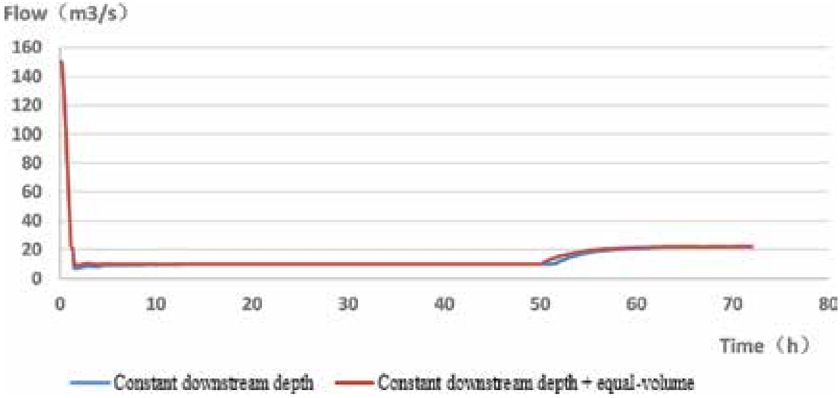

Figure 31.
Flow change of upstream check gate 4 under different operation conditions.

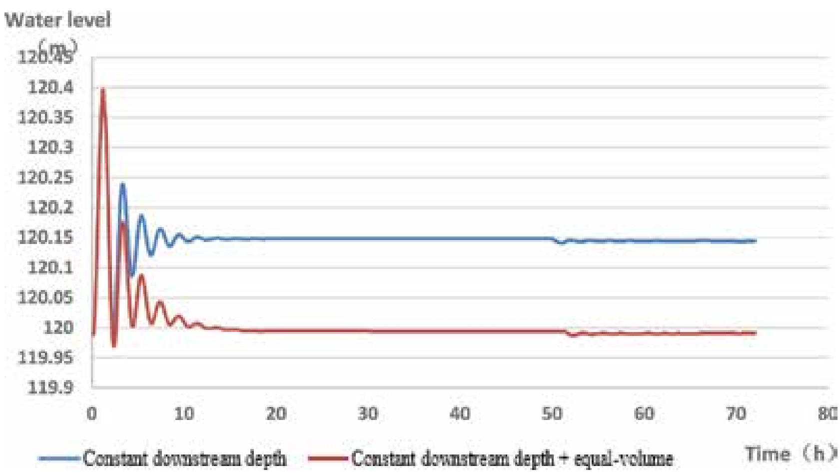

Figure 32.
Flow change of upstream water level of upstream check gate 4 under different operation conditions.

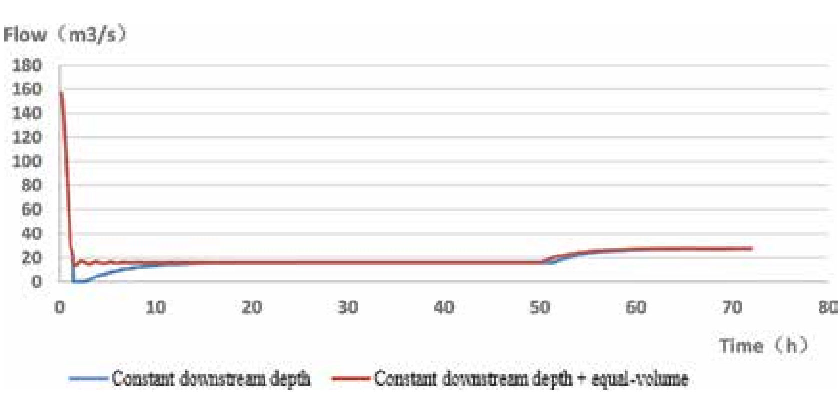

Figure 33.
Flow change of upstream check gate 5 under different operation conditions.

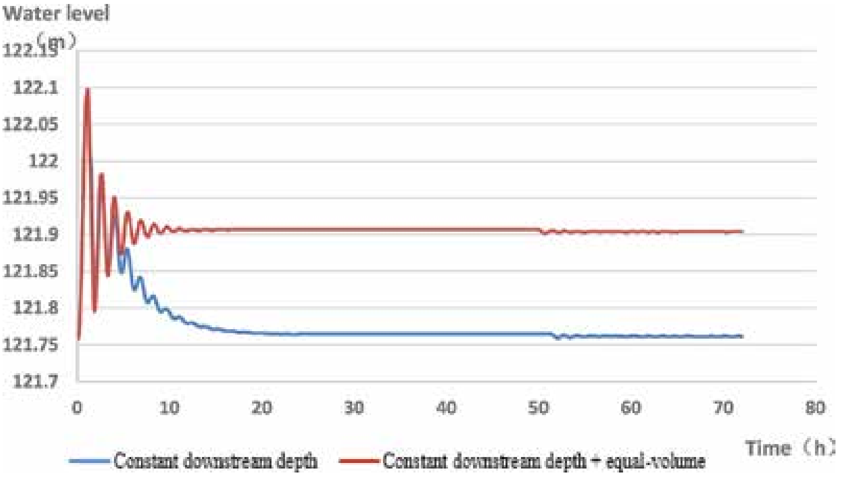

Figure 34.
Flow change of upstream water level of upstream check gate 5 under different operation conditions.

more obvious when nearer to upstream. This is because that the nearer it is to upstream, the greater the cumulative difference between equal-volume operation and constant downstream depth operation modes becomes, and the greater the check gate operating difference becomes.

5. Conclusions

This chapter develops a complete set of check gate operation rules and automatic control algorithm for sudden water pollution events in the MRP. For great change in channel pool operating conditions due to emergency operating conditions, a set of operational thoughts different from conventional ones is put forward. Emergency operations are divided into operation rules development and automatic control algorithm research. Sudden water pollution accidents under emergency operations' operating conditions of the main canal are divided into accident pool, joint emergency operations of upstream, and downstream sections of the accident pool. Different automatic control algorithms come up with different channel sections. The results of their applications indicate that the methods proposed can be used to guide safe, steady emergency operations of channel section, ensure steady water level in the final channel section, and keep it to the target water level.

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Chapter 5

Emergency Management System for Sudden Water Pollution Accidents

Haichen Li, Weihong Liao, Jiabiao Wang and Zhiguo Gan

Abstract

The emergency management system for sudden water pollution accidents of the main canal is the integrated application of the aforesaid three key technologies and is the key to verify the effect of practical application of these technologies. The emergency management system is formed by integrating basic information, measured data, and professional models through the communication mode of network transmission. The system can provide support for emergency response in case of emergency conditions including sudden water pollution accidents and technical support for security operations of the MRP.

Keywords: emergency management, system, integrated application

1. Introduction

The system integrates and visualizes accident-related information and model by mainly using computer and information technologies to provide decision making support for emergency response [1]. For a large-scale complex, water diversion project like the MRP, the scale, and complexity of the project requires more scientific decision-making methods and tools under emergency conditions including sudden water pollution accidents to ensure its safe and reliable operation and reduce the loss and influence scope as far as possible. In order to achieve consistency of information data of the project and improve model analysis efficiency, as well as the water diversion stability and the recovery performance for coping with emergencies, it is very important to establish an emergency management platform, which integrates project parameters, spatial analysis, mathematical model, decision-making consultation, and other functions.

2. Service objects and construction goals

2.1 Service objects

Three levels of management institutions are set up for the MRP: 1 head company, 5 sub-companies, and 47 management offices. The emergency management system mainly serves these management institutions above. Through operation of the system, the management staff can make scientific and reasonable responses to sudden water pollution accidents in the main canal.

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2.2 Construction goals

The system focuses on the emergency management of sudden water pollution accidents in the MRP by integrating multisource data (i.e., project information and real-time measured data), professional models (i.e., hydrodynamic and water quality model, sudden water pollution accidents source identification model, and emergency operation model), and developing emergency response and decision-making consultation modules to formulate the emergency management system, in order to provide support for emergency response in case of emergency conditions including sudden water pollution accidents, and provide technical support for security operations of the MRP.

3. System framework

3.1 System hierarchy

The emergency management system takes GIS as a platform, uses Client/Server architecture and server to arrange data server and store space data, project parameters, model parameters, result data, and the like, and provides data management and sharing, system maintenance, concurrency control, and other services. An application server is arranged at the client to encapsulate user's application business logic and provides friendly, simple operating interfaces. Through inputting a request or command, the user calls the service related to the application server. The application server interacts with a data server according to customer's demands. After receiving an application service request, the client implements corresponding data processing according to a received application service request and returns the processing result to the application server, and then, the application server performs the business logic process corresponding thereto and finally returns the processing result to the user.

The emergency management system includes four layers: monitoring, database, decision control, and user interface. The monitoring layer is responsible for water quantity and quality monitoring and the local control and implementation of check gate pump group. The database layer includes real-time water quantity and quality information, real-time operating condition information and parameters, model parameter information, etc. The application layer includes four modules: information management, traceability simulation, emergency control, and decision consultation. The user interface layer is used for completing interactive operations with the user's diagrams, tables, GIS, and so on. The overall system framework design is shown in **Figure 1**.

3.2 Database design

The database for the emergency management system is designed and managed by using Microsoft SQL Server 2008 R2. The design is introduced below by taking two types of data—attribute data of inverted siphon and the data real-time monitored at dividing gate—as examples. Refer **Tables 1** and **2**.

3.3 Model encapsulation and integration

3.3.1 C# language call convention

The model kernel is developed on the basis of the C++ source code of the control simulation model for the middle route. The interface is packaged in the style of standard C++, and the call is made by the C# language.

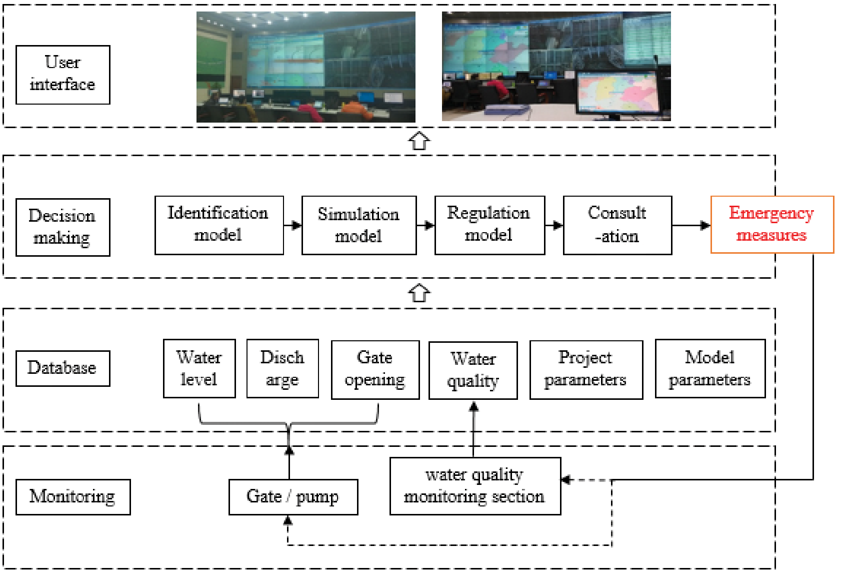

Figure 1.
System framework diagram.

Item	Code	Type of data
Inverted siphon node no.	DXHJD_ID	numeric
Type of node	JCTTYPE	int
Type of cross-section	PORTTYPE	int
Name of node	JCTNAME	varchar (100)
Node inlet pile no.	JCTINCOOR	float
Node outlet pile no.	JCTOUTCOOR	float
Number of parallel inverted siphon	PARALLELNUM	int
Actual pipeline length	LENGTH	float
Elevation of inlet bottom	IN_TOPELEV	float
Elevation of outlet bottom	OUT_TOPELEV	float
Roughness coefficient	ROUGHNESS	float
Loss coefficient at inlet transition section	KSI_0	float
Loss coefficient of inlet sluice chamber	KSI_1	float
Loss coefficient of inlet pipe orifice	KSI_2	float
Loss coefficient of check gate	KSI_3	float
Loss coefficient of outlet pipe orifice	KSI_4	float
Loss coefficient of outlet sluice chamber	KSI_5	float
Loss coefficient at outlet transition section	KSI_6	float

Table 1.
Table of basic data of inverted siphon node.

The program source code for developing C# is not compiled into a local binary code that can be directly executed on an operating system but into an intermediate code. And then, the intermediate code is executed via a virtual machine for .Net framework, which is called common language runtime (CLR). All the .Net

Item	Code	Type of data
Real-time data no.	FSKSSSJ_ID	numeric
Node no.	JCTID	int
Monitoring time	JCTIME	date time
Real-time flow	INSTANTFLOW	float
Total flow	TOTAL_FLOW	float

Table 2.
Data real-time monitored at dividing gate.

programming languages are compiled into an intermediate code, which is called Microsoft intermediate language (MSIL). Therefore, although both final programs and traditional executable files have the suffix ".exe" on the surface, in fact, if the .Net framework is not installed on a computer, these programs will not be able to be executed. When executing the programs, .Net framework translates the intermediate code into a binary machine code so that it can run correctly. The final binary code is stored in a buffer. Once the programs use the same code, the version in the buffer will be invoked.

3.3.2 Fault-tolerant processing

When data or a file is damaged or lost due to various reasons in the model system, the system can record the related error code by using the running log mode and basic reasons. At this time, the system will enter a self-checking data process and correct data in such a manner that has no effect on the next computing. On the contrary, the model releases resources, stops working, and pops up an outer notification frame.

3.3.3 Management of input and output of text and database formats

Model basic engineering parameter files support text read-in, other parameters are inputted by calling an interface, and input data are checked by the model kernel. When finding unqualified data, the model kernel notifies an external call framework via error log and returned value. The returned value is judged each time before entering the next step. After basic engineering at the initial phase of model simulation is established, the user can intervene data preparation by a computing unit by calling an intermediate interface. When each unit completes computing, the user can also make judgment through an interface to obtain computing data at each step. The acquisition of such data is generally done by calling an interface and finally reflected in final text output.

The addition of database elements supports SQL Server database data import and export, and previous preparation data and intermediate computing data of the model can be easily imported into the database.

When data is being outputted, the external framework is able to obtain intermediate data by calling an interface at each step during computing, realizing real-time display demand, and importing all computing data via analysis of a final computed result file for animation play browsing.

3.3.4 Algorithm and model encapsulation

In this part, the kernel model is encapsulated; mainly, the model code is encapsulated as a complete functional code. The model is divided into three parts: model preparation, model computing, and model resource release.

3.3.4.1 Model preparation phase

Model preparation phase needs basic engineering parameters and computing simulation parameters, which are imported into database by reading all element parameters of channel cross-section of the main canal in a text mode. The user manually inputs computing simulation parameters required for control model in the system interface and then imports them into the database.

3.3.4.2 Model computing

First, perform initialization of the parameters of the channel, check gate, release sluice, dividing gate, aqueduct, inverted siphon, then read the parameters needed for control model, entered by user and finally compute with with control algorithm [2]. This computing unit is called by external logic and stores intermediate data computed so as to meet the demand for the next computing. When the computing is finished, the resources of various types of computing units can be released according to user demands.

3.3.4.3 Model resource release

The memory resource that is occupied by the model is released according to user demands.

3.3.5 Encapsulation of algorithm Web service

XML is used as the standard format for data exchange between systems; Web services are used to publish the service to the Internet; dynamic combination and integration of Web components are performed according to business and processes; data transmission is ensured by using a message queue mechanism, in order to achieve data exchange and sharing purposes. An integrated system with good expansibility, less resources occupation, loose coupling, strong reusability, and convenient maintenance can be built by using the general exchange platform.

3.4 Technical structure for system implementation

The design of the emergency management system will be developed by using an MVC three-layer structure. That is to say, the whole business application will be divided into a presentation layer, business logic layer, data access layer, and so on. Among them, the data access layer (DAL) is used to achieve the interaction with and access to the database and to obtain data from the database or save data to the database. The business logic layer (BLL) connects the preceding and the next and used for logical data processing of upper and lower interactive data to achieve business goals. The presentation layer (UI) mainly used to realize the interaction with the user, receive the user request, or return display of data result of the user request, while concrete data processing is handed over to the service logic layer and the data access layer to process. The business entity model used to encapsulate entity data structures, generally used to map the data tables or views of a database and to describe objects that exist objectively in the business. Model is separated for better decoupling, giving a better play to layering, better reuse and expansion, and enhancing flexibility. Common class library (Common): common utility helpers.

Visual Studio is the development environment launched by Microsoft Corp. At present, it is the most popular Windows platform application development environment. The system will use Visual Studio 2012 version for development and C# language.

4. System functions

The emergency management system includes four core modules: a data management module, a traceability simulation module, an emergency operation module, and an emergency consultation module.

4.1 Data management

4.1.1 Attribute data management

A node data management and maintenance module mainly include the data management and maintenance in the following aspects: channel node data, inverted siphon node data, transition section node data, connecting element node data, lateral bypass flow node data, check gate node data, aqueduct node data, tainter gate data, rectangular cross-section node data, etc [3]. For details, see **Table 3** below.

The module function interface is displayed by taking tainter gate data management interface (**Figure 2**) for example.

Node data	Description
Channel node data	Mainly includes basic node data, such as node name, node inlet/outlet pile no., channel bottom width, channel side slope, inlet bottom elevation, outlet bottom elevation, etc.
Inverted siphon node data	Mainly includes basic node data, such as node name, node inlet/outlet pile no., the number of parallel inverted siphons, channel bottom width, channel side slope, inlet bottom elevation, outlet bottom elevation, inlet sluice chamber loss coefficient, outlet sluice chamber loss coefficient, inlet pipe orifice loss coefficient, outlet pipe orifice loss coefficient, etc.
Transition section node data	Mainly includes basic node data, such as node name, node inlet/outlet pile no., area change loss coefficient, other loss coefficients, etc.
Connecting element node data	Mainly includes basic node data, such as node name, the number of inlet cross-sections, inlet cross-section bottom elevation, the number of outlet cross-sections, outlet cross-section bottom elevation, etc.
Lateral bypass flow node data	Mainly includes basic node data, such as element type, element no. node name, province/city where they are, etc.
Check gate node data	Mainly includes basic node data, such as node name, node inlet/outlet pile no., control water stage, sluice type, sluice parameters, initial opening, etc.
Aqueduct node data	Mainly includes basic node data, such as node name, node inlet/outlet pile no., the number of parallel aqueducts, inlet bottom elevation, outlet bottom elevation, roughness coefficient, etc.
Tainter gate data	Mainly includes basic node data, such as node name, the number of gate holes, gate bottom sill elevation, gate hole bottom width, height difference between tainter gate shaft and seating point when the tainter gate is closed, tainter gate radius, etc.
Rectangular cross-section node data	Mainly includes basic node data, such as the type, name, width, height, and the like of cross-section.

Table 3.
List of node data.

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Figure 2.
Tainter gate data management interface.

4.1.2 Monitored data management

The monitored data of check gate, dividing gate, and release sluice under the project (at present, some release sluices are used as dividing gates) were reported artificially or automatically obtained from a sluice control system. For details, see **Table 4**.

The module function interface is displayed by taking the flow data query interface for the check gate in Diaohe River (**Figure 3**) for example.

4.2 Traceability simulation

The traceability simulation module calls the pollution source information inversed by the sudden water pollution accidents traceability model depending on abnormal data of some cross-section water quality and then predicts the pollutant diffusion process by using a hydrodynamic water quality model [4]. For details, see **Table 5**.

The module function interface is displayed by taking the call interface of traceability model for example (**Figure 4**).

Type of structure	Monitored item	Monitoring frequency	
		Artificially	Automatically
Check gate	Upstream water stage of check gate, downstream water stage of check gate, gate hole opening, and flow	Every 2 hours	Every 10 minutes
Dividing gate	Upstream water stage of check gate, gate hole opening, flow and the quantity of divided water	At 8:00 AM every morning	
Exit sluice			

Table 4.
List of node data.

Figure 3.
Flow data query interface for check gate in Diaohe River.

Function name	Input item	Output item	Functional description	Effect display
Tracking Traceability	Monitoring start time, monitored pile no., several monitored data at fixed interval	Pollution source intensity and position, release time	The model is called to calculate possible position of a pollution source according to monitoring start time, pollution source intensity and initial start location, and pollutant release time.	The error between simulate value and actual value through a chart.
Diffusion Simulation	Pollution source position pile no., pollution source intensity, and pollutant release time	The diffusion process and peak motion process of concentrations of pollutants in all channel sections of the whole main canal that change with time	Model simulation of pollutant diffusion range and effect is carried out according to pollution source position and intensity, and pollutant release time. The model is a precise simulation model, which can be used to simulate the diffusion process of pollutants in the channel of the whole main canal, peak arrival time, the change process of concentration of pollutants in channel section, etc.	Two methods are used to dynamically display the change in concentration of pollutants along the route. Because a concentration at a different level corresponds to a different color, one method visually displays such change via dynamic change of channel color; the other method displays the change process of pollutants along the route by using the change curve of concentration along the route at different times.

Table 5.
Main functions of traceability simulation.

Figure 4.
Traceability interface.

4.3 Emergency operation

The emergency operation module develops the emergency operation plan for accident channel pool, upstream, and downstream sections of accident channel pool and performs simulation calculation, based on simulation and prediction of sudden water pollution accidents events, in combination of contingency plan and other documents and by using a sluice emergency operation model, and then, it evaluates the effect of such plan [5].

The module function interface is displayed by taking the emergency operation plan setting interface (**Figure 5**) for example:

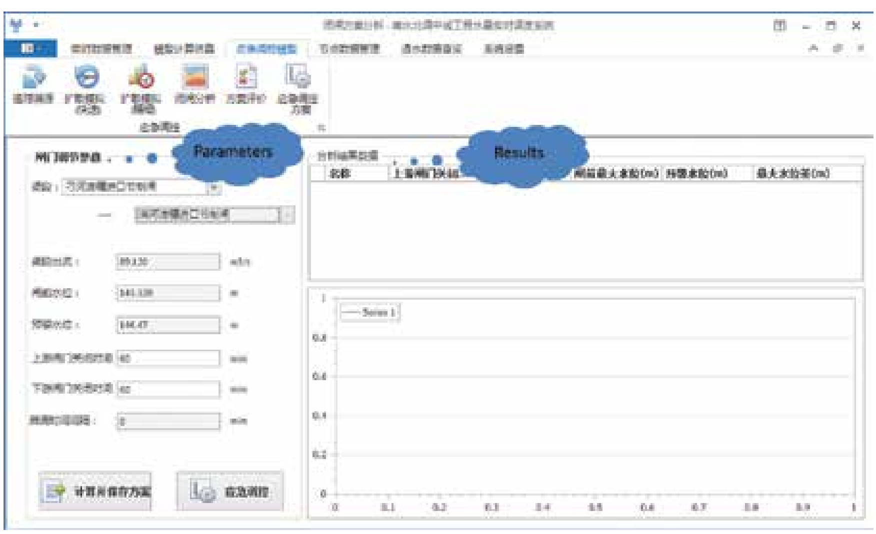

Figure 5.
Interface of Emergency operation Plan.

4.4 Emergency consultation

The emergency consultation module provides a consultation environment for management units and related decision makers according to the basic information of a sudden event and the simulation results of a professional model group. The module mainly has the following functions: consultation procedure management, consultation information collection and summary, group consultation, file management and information release, program review, and knowledge update.

4.4.1 Consultation procedure management

A decision consultation system should clearly carry out the specific process of consultation in order to make the system user know related information to be prepared and related work to be done at different stage of consultation and to make the system user able to understand progress of current consultation, realizing the management of the consultation process.

4.4.2 Collection and summary of information for consultation

Before the beginning of the consultation, the relevant information should be prepared for the topic of the consultation. According to actual situation, the consultation organization specifies requirements related to required information for units to participate in the consultation, which then submit related information to the consultation organization for review. After the review, the information is collected and arranged, and then it is uploaded to the decision consultation system for query and use in the consultation process.

4.4.3 Group consultation

Organize decision making personnel and assistants and other relevant personnel to carry out the consultation. During the consultation process, a template provided by the decision consultation system is used to display prepared information and other collected information related to such consultation; then, the decision making personnel carries out analysis, discussion, and the like according to the above information and current actual situation and develops one set or more sets of programs for selection, thus making decisions and obtaining the conclusion.

4.4.4 File management and information release

After completion of the consultation, the consultation result is arranged and summarized to form related information, and then, such information is arranged and filed by using the decision consultation system; at the same time, the information related to the consultation can be released as required.

4.4.5 Consultation review and knowledge update

After the consultation is finished, the files and information are reviewed and evaluated; useful information is extracted therefrom; and existing plans, rules, and knowledge are perfected, and the blank or missing parts are supplemented.

The module function interface is displayed by taking the event information input interface (**Figure 6**) for example:

Figure 6.
Event information input interface.

5. System safety and maintenance

5.1 System safety

5.1.1 Network security design

The security measures of the system in the network layer mainly include firewall protection and intrusion detection technology. Connection with the Internet is done by using firewall protection; different types of firewall protection systems are used to protect the connection between Web server and back-end database.

Firewall protection functions include packet filtering based on state detection; multistage 3D access control mechanism; management mechanism oriented to objects; supporting multiple connection methods and transparent router; supporting OSPF, IPX, NETBEUI and SNMP and other protocols; having bidirectional address conversion ability; transparent application proxy; one-time password authentication mechanism; bandwidth management; having some built-in functions of intrusion detection or capacity to interact with intrusion detection equipment; remote management capacity; hot standby; load balancing; supporting dynamic IP address; embedded VPN function support; and flexible audit and log functions.

The functions provided by the network intrusion detection system include realizing real-time, distributed, and collaborative intrusion detection in the network environment to fully detect possible intrusion; timely identifying various hacker attacks, and when an attack is found, blocking and weakening attack behaviors, recording detailed records, creating an intrusion detection report, and timely giving a warning to the administrator; performing multilayer scan as required by the administrator, and configuring multiple scans according to specific time, width and fineness demands; supporting parallel detection and being able to perform multiple detection execution of a large network conveniently and simultaneously; detection and scan should not have an effect on normal network connection service and network efficiency; the feature library of detection should be comprehensive and can be updated timely; security detection strategy can be set by the user, the grades of detection intensity and disk degree are managed, and the user can choose detection strategies according to different needs; helping build security strategy, having detailed help database, and helping the administrator to realize network security and developing practical, enforceable network security policies.

5.1.2 Data security design

5.1.2.1 Database management

5.1.2.1.1 Security strategies for database user

The security policy of database users includes the security of general users and end users.

The security of general users is solved by password management and privilege management. If a user confirms its identity through a database, then connect to the database by using password encryption. Because of large number of users, rich data types, and a large amount of data in the system, a "role mechanism" is used to effectively manage authorities.

For security of end users, security policies must be developed for end users. The database has a certain scale. Security managers determine user group classification, create user roles for these user groups, grant required authorities and application roles to each user role, and assign an appropriate user role for each user. When dealing with special application requirements, security managers also must explicitly grant specific permission requirements to the user.

5.1.2.1.2 Security policies for database managers

After the database is created, immediately change the password of the user with management authorities to prevent illegal users' access to the database; protect the connection of the administrator with database; and use roles to manage the authority of the administrator.

5.1.2.1.3 Security policies for application developers

Authorities for application developers: Database application developers are the unique database users who need special authorities to complete their work. However, only some special system authorities are granted to developers to limit their operation of the database. Application developers should not compete with end users for database resources and should not harm other applications for database.

5.1.2.2 Audit function

Audit function is a very important security measure, which is used to monitor the actions applied by users on the database. There are two ways of audit, namely user audit and system audit. During user audit, an audit system records all attempts to access their own tables or views (including successful and unsuccessful accesses, the user name, time, operation code of each operation, and other information). Such information is generally recorded in a system table, and by using the information, users can carry out audit analysis. System audit is carried out by the system administrator, and it mainly involves the Level 1 commands of the system and the use of the database.

5.1.2.3 Database backup

Hardware redundancy at any degree cannot completely guarantee single-point data security, so do RAID technology and mirror technology; and even dual machine backup cannot replace the importance of data backup. Remote backup of data is very effective and important when the client computer application system encounters a single point emergency or natural disaster. Good backup strategies and tools cannot only improve the degree of backup automation but also well recover data after they are destroyed.

The system adopts a backup plan, which combines complete backup and difference backup. Difference backup is performed every 12 hours; complete backup, every 7 days; offsite data backup, once a month.

5.1.3 Application safety design

5.1.3.1 Application server

The application-level security measures provided by a Web application server include SSL support (SSL protocol), authentication strategies (a group of strategies for user identity which provide view access network resource), authority strategy (providing the range of network resource accessed by a user or a user group and the definition of authorities), authorization policy (providing an authorization policy to make user temporarily access specific network resources after authorization), and secure API (providing a unified API for all security features).

5.1.3.2 Application service

According to system features, an enhanced, targeted security guarantee mechanism is provided, which includes program interface security, system login security, user role control security, input data security inspection, application system database access security, and the security strategies to prevent online password from being stolen.

According to role division, the system is divided into two roles: system administrator and system user.

The system administrator manages the users and authorities in the whole system.

All system users can use this system according to their authority assigned by the system administrator.

5.1.3.3 System monitoring and log

In addition to the use of security technology to ensure the security of the system, the system also uses system monitoring, log management, and other ways to ensure safe operation of the system. A good security monitoring function can greatly improve the overall security of the system so as to detect and eliminate security risks as soon as possible. By using a WWW server, a database server system, the system provides monitoring log for application access to understand who have visited the system, which services have been used, and whether there is someone trying to attack the system or violate the restrictions of the system.

5.2 Operation and maintenance

5.2.1 Operating environment

5.2.1.1 Hardware device

In order to obtain good operation effect, the hardware for operating this system should be up to the following standard:

Client:

1) CPU: above Intel Core i3
2) Internal memory: above 4G
3) Hard disk: above 320G
4) Graphics card: above 1G

Server:

1) CPU: above Intel Core i3
2) Internal memory: above 4G
3) Hard disk: above 500G.
4) Graphics card: above 1G

Note: the available space of the server hard disk is above 500G due to large amount of data in the database. If the amount of file data in the database is too large, the database should be backed up at intervals.

5.2.1.2 Support software

1) Operating system: Windows7 32-bit OS
2) Operating platform: .Net framework 4.0, Visual Studio, SQL, ArcGIS

5.2.2 System maintenance

In order to ensure normal operation of the platform, it is necessary to establish an efficient information system operation and maintenance mechanism, implement a responsibility system, and improve the level of operation and maintenance of the information system; improve the capacity and methods of monitoring and emergency response of the information system to ensure safe and stable operation of the information system; establish perfect system operation and maintenance methods and include operation and maintenance outlays of information system into departmental budget.

6. Summary

This chapter designs and develops an emergency management system of sudden water pollution with complete functions for the middle route under MRP depending on engineering characteristics, management department demands, and the emergency response procedures for sudden water pollution and through the integration of simulation model, traceability model, and control model. The system is deployed in each management department of the project, providing technical support for the management to scientifically cope with sudden water pollution events that may occur in the normal water supply process of the middle route.

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The Middle Route of the South-to-North Water Diversion Project (MRP) is strategically important for China, and has made a great contribution to the sustainable development of society and economy in North China. Various potential risks of sudden water pollution accidents are distributed along the Grand Canal, which may result in huge losses and endanger the safety of water supply. In order to provide technical support for emergent operations in the MRP, the emergency operation technologies for sudden water pollution accidents were developed. This book introduces these emergency operation technologies, including simulation technology for hydrodynamic and water quality in the main canal, traceability technology, emergency operations, and an emergency management system for sudden water pollution accidents.

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