A study on sediment avoidance diversion and the coordinated dispatch of water and sediment at an injection-water supply project on a sediment-laden river

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ABSTRACT
Rivers in the north-western region of China have a high sediment content, and the construction of reservoirs on these rivers must address the problem of sedimentation that results in a loss of reservoir capacity. In this study, the YZD (Yazidang) reservoir was considered as a typical case, to address the problem of sedimentation in injection-type water supply reservoirs in Northwest China. The temporal and spatial characteristics of the sediment in this water diversion project were analyzed based on available 46-year water and sediment data. The concept of ‘sediment avoidance diversion’ was proposed, which reduces the entry of sediment into the reservoir. A one-dimensional numerical model for water-sediment coupling was established, and the characteristics of water-sediment erosion and deposition for different operational modes were analyzed. The results show that during a period of high sediment content, large-scale silting occurs in a water diversion channel. To alleviate sediment deposition in a diversion channel, two control strategies for mitigating silting were formulated. Considering the pros and cons of these control strategies, a reasonable water-sediment joint operation plan has been proposed to extend the service life of the YZD reservoir. This research provides a theoretical basis for sedimentation treatment and control of similar reservoirs.

1. Introduction
The northwest region of China is vast and rich in hydropower resources but most of the potential hydropower resources are associated with rivers that have a high content of sand (Wan, 1982). The resulting sedimentation causes serious problems in the construction of water supply reservoirs on these rivers (Hou et al., 2017).

Reservoir silt deposition is related to the inflow, the amount of silt entering the reservoir, and the operational mode of the reservoir. Therefore, it is necessary to understand the dynamic process of reservoir silt deposition, reasonably select the inflow and sediment conditions, and determine the optimal operational mode of reservoirs located on sediment-laden rivers. These aspects have been the focus of research globally. For example, Du and Zhu (1992) established a stochastic dynamic programing model for reservoir water-sediment joint operation and studied the optimal operation of water and sediment integrated regulation for the Sanmenxia Reservoir. Ezz-Aldeen et al. (2018) used the SWAT (Soil & Water Assessment Tool) model to assess the annual runoff and sediment volume of the Dokanba watershed and provided a theoretical framework for estimating the sedimentation of reservoirs. Wu et al. (2010) introduced the principle of adaptive control for the operation of reservoirs on sand-laden and sediment-laden rivers. Peng et al. (2004) put forth a multi-objective decision-making model for reservoir water-sediment joint operation using multi-objective theories and methods. Li et al. (2016) used back-propagation error training of artificial neural networks to analyze the relationship between the efficiency of sand flushing of the Three Gorges Reservoir and the factors that affect the efficiency. Bao et al. (2007) used the density current total flow differential model to predict the movement of sediment at the dam site of a reservoir and conducted a water-sediment joint operation model to maximize the sediment discharge ratio of the reservoir based on the characteristics of the flood at the dam site. Carriaga and Mays (1995) established a coupling model for an optimal water and sediment joint operation for multi-sediment river water supply reservoirs, evaluated and tested the effectiveness of the model using four sediment transport functions, and proposed a constraint formula considering the uncertainty of sediment transport parameters. Chen et al. (2018) studied the Shimen Reservoir watershed in Taiwan and proposed...
a grid-based comprehensive model of sediment production and transportation at the basin scale, which enabled its sustainable development. Koda et al. (2019) proposed that the use of reservoir sediment for farmland reclamation is an effective way to alleviate reservoir sedimentation and proposed a possible solution to the problem of sedimentation of the Tigray Reservoir. De Cassia Condé et al. (2019) used MODIS remote sensing images to indirectly evaluate the sedimentation of hydropower dams, and studied the relationship between turbidity reduction and sedimentation, and indicated that this technology can be used to monitor large-scale sedimentation processes in reservoirs.

Several researchers have also analyzed the process of sedimentation in reservoirs and the joint operation model (Chen et al., 2017; Huang et al., 2019; Khorrami & Banihashemi, 2019; Tadesse & Dai, 2019; Wang & Hu, 2009). However, the principles of sediment avoidance diversion, coordinated dispatch of water, and sedimentation associated with injected water supply projects need to be further enhanced. This study investigates the YZD project in the northwest region of China, as an example. The project is primarily composed of an open canal section and a reservoir section. It is the source of the Ningdong Water Supply Project and provides water for industry, ecology, and the population in the region. The safe and normal operation of the YZD project is important for the development of Ningdong and the economy of the Ningxia region (Liu et al., 2018). The YZD project is a typical injection-type reservoir located on a sediment-laden river. The problem of sedimentation in the water channel and reservoir area has become more pronounced with the operation of the reservoir (Wang, 2018). This research establishes a mathematical model of water and sediment and analyzes the characteristics of water and sediment in the channel and reservoir area. A concept of sediment avoidance diversion, involving a coordinated dispatch of water, and sedimentation for different operating conditions is proposed. The feasibility of the theory of sediment avoidance diversion for water supply projects has been verified. This research provides a technical and theoretical basis for the treatment and control of sedimentation in reservoirs of the same type as the YZD project.

2. Numerical model and validation

2.1. The hydrodynamic model

To determine the velocity field, a six-point implicit finite difference scheme is used to numerically solve the continuity and conservation of momentum equations for unsteady flows (Ghalandari et al., 2019), and is applicable for annular networks of flood plains and unsteady flows of rivers and estuaries. The flowchart for this process is shown in Figure 1.

The primary equations used in the model are as follows (Kargar et al., 2020; DHI, 2014):

\[
\begin{align*}
\frac{B_s}{\partial t} + \frac{\partial Q}{\partial x} &= q \\
\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left( \alpha Q^2 \right) + qA \frac{\partial h}{\partial x} + gQ|Q| \frac{C^2AR}{C^2} &= 0 \\
\alpha &= \frac{A}{Q^2} \int_{A} u^2 dA
\end{align*}
\]

where \( x \) and \( t \) are the spatial and time coordinates, respectively, \( Q \) and \( h \) are the sectional discharge and water level, respectively, \( A \) and \( R \) are the cross-sectional area and hydraulic radius, respectively, \( B_s \) is the river width, \( q \) is the lateral inflow, \( C \) is the Chezy coefficient, \( g \) is the acceleration due to gravity, \( \alpha \) is the vertical velocity distribution coefficient, and \( u \) is the average flow velocity over the cross-sectional area.

The numerical scheme adopted for this process is a six-point Abbott-scheme as shown in Figure 2. A set of implicit finite difference equations are solved using a computational grid consisting of alternating \( Q \) and \( h \)-points, where the discharge \( Q \) and water level \( h \), are respectively computed at each time step as depicted in Figure 3. The computational grid is generated automatically by the model based on user requirements. All \( Q \)-points are placed midway between neighboring \( h \)-points. It may be noted that the distances between adjacent \( h \)-points are non-uniform.

The derivatives of the continuity equation at time \( n + 1/2 \), are as follows (DHI, 2014):

\[
\frac{\partial h}{\partial t} \approx \frac{h_{j+1}^n - h_j^n}{\Delta t} \\
\frac{\partial h}{\partial t} \approx \frac{Q_{j+1}^{n+1/2} + Q_{j-1}^{n+1/2}}{2} \Delta x_j
\]

\( B_s \) in Equation (1) is approximated by (DHI, 2014):

\[
B_s = \frac{A_{o,j} + A_{o,j+1}}{\Delta x_j}
\]

where \( A_{o,j} \) is the surface area between the grid points \( j - 1 \) and \( j \), \( A_{o,j+1} \) is the surface area between the grid points \( j \) and \( j + 1 \), and \( \Delta x_j \) is the distance between the points \( j - 1 \) and \( j + 1 \).

Substituting for the derivatives in the continuity equation yields the following formulation (DHI, 2014):

\[
\alpha_j Q_{j-1}^{n+1} + \beta_j h_j^{n+1} + \gamma_j Q_{j+1}^{n+1} = \delta_j
\]

where \( \alpha, \beta, \) and \( \gamma \) are functions of \( B_s \) and \( \delta \), which depend on \( Q \) and \( h \) at time \( n \), and \( Q \) at time \( n + 1/2 \).
The derivatives of the momentum equation are expressed in the following manner (DHI, 2014):

\[
\frac{\partial Q}{\partial t} \approx Q_{j}^{n+1} - Q_{j}^{n} \quad \Delta t \\
\frac{\partial \left( \alpha \frac{Q^2}{x} \right)}{\partial x} \approx \frac{\left( \alpha \frac{Q^2}{x} \right)_{j+1}^{n+1/2} - \left( \alpha \frac{Q^2}{x} \right)_{j-1}^{n+1/2}}{\Delta 2x_j} \\
\frac{\partial h}{\partial x} \approx \frac{(h_{j+1}^{n+1} + h_{j+1}^n)}{2} - \frac{(h_{j+1}^{n+1} + h_{j+1}^n)}{2} \quad \Delta 2x_j
\]

With all derivatives being substituted, the momentum equation can be written in the following form (DHI, 2014):

\[
\alpha_j h_{j-1}^{n+1} + \beta_j Q_j^{n+1} + \gamma_j h_{j+1}^{n+1} = \delta_j
\]

where \( \alpha_j = f(A) \), \( \beta_j = f(Q_j^n, \Delta t, \Delta x, C, A, R) \), \( \gamma_j = f(A) \), and \( \delta_j = f(A, \Delta x, \Delta t, \alpha, q, v, \theta, h_{j-1}^n, Q_{j-1}^{n+1/2}, Q_j^n, h_{j+1}^n, Q_{j+1}^{n+1/2}) \). The parameters have the same representations as described above.

### 2.2. The sediment transport model

In this study, the sediment grade was analyzed. The Engelund and Fredsøe (1976) models were used to
calculate the bedload and suspended load, respectively, and the corresponding equations are described below:

### 2.2.1. Bedload equation

Engelund and Fredsøe (1976) obtained the following bedload function:

\[
\Phi_b = 5 \left[ 1 + \left( \frac{\pi}{2} \beta \frac{\theta'}{\theta_c} \right)^4 \left( \sqrt{\theta'} - 0.7 \sqrt{\theta_c} \right) \right]^{1/4} \tag{11}
\]

where \( \Phi_b \) is the dimensionless bedload sediment transport rate, \( \beta \) is the dynamic friction coefficient with a value close to 1 (see DHI, 2014), \( \theta' \) is the dimensionless skin friction, and \( \theta_c \) is the critical dimensionless bed shear stress. The value of \( \theta_c \) after calibration is 0.056. The relationship between the dimensionless skin friction and dimensionless bed shear stress for a dune covered bed can be approximated by:

\[
\theta' = 0.4 \theta^2 + 0.06 \tag{12}
\]

where \( \theta \) is a dimensionless bed shear stress.

### 2.2.2. Suspension equation

The suspended load \( q_s \) is obtained from the integral of the current velocity \( u \) and the concentration of the suspended sediment \( c \) (DHI, 2014; Vinh et al., 2014):

\[
q_s = \int_a^D c u dy \tag{13}
\]

\[
c = c_a \left( \frac{D - y}{y} \frac{a}{D - a} \right)^z \tag{14}
\]

In the above equation \( a \) is the thickness of the bed layer which is approximately equal to 2\( d \) (see DHI, 2014), where \( d \) is the grain diameter, \( D \) is the depth of flow, and \( c_a \) is the sediment concentration at the bed. The current velocity \( u \) at a distance \( y \) above the bed level is described by a logarithmic velocity profile:

\[
u = 2.5 u_f' \ln \left( \frac{30y}{k} \right) \tag{15}\]

where \( k \) is the equivalent sand roughness, and its value is 2.5\( d \) (see DHI, 2014), \( u_f' \) is the bed shear velocity or friction velocity, \( z \) is the Rouse number: \( z = w/(0.4u_f') \), and \( w \) is the settling velocity of the suspended material, given by one of the following expressions depending on the grain size:

\[
w = \begin{cases} 
\frac{1}{18} \frac{(s-1)gd^2}{v} & \text{for } d \leq 0.1 \text{ mm} \\
\frac{10v}{d} \left[ 1 + \frac{0.01(s-1)gd^3}{v^2} \right] - 1 & \text{for } 0.1 \text{ mm} \leq d \leq 1.0 \text{ mm} 
\end{cases} \tag{16}
\]

where \( s \) is the relative density of the sediment, and \( v \) is the coefficient of kinematic viscosity.

### 2.3. Implementation of numerical model

The YZD project is about 60 km from Yinchuan City in Ningxia Autonomous Region and is located in Lingwu City, at 105.59° to 106.37°E longitude and 37.60° to 38.01°N latitude. It is in the mid-temperate continental monsoon climate zone. The dam crest elevation of the second phase project is 1258.00 m, the design water level is 1255.8 m, the design total storage capacity is 44.07 × 10^6 m³, the design water delivery flow rate is 8 × 10^5 m³/d, and the maximum discharge is 1.2 × 10^6 m³/d. The layout of the building is shown in Figure 4.

The area covered by the calculations of this study includes the water delivery channel and reservoir. The length of the water delivery channel is 1230 m. The specific location of the water delivery channel is shown in Figure 4. The distance from one end of the channel to the dam site is 2950 m, with a total of 53 calculated sections. A talweg with a total length of 4180 m is formed by connecting the lowest points of the sections and is shown in Figure 5(a). Fifty-three calculated cross-sectional shapes
were measured using ADCP (Acoustic Doppler Current Profilers) and RTK (Real-time Dynamic Positioning Technology). The measured topography is shown in Figure 5(b).

2.4. Validation of the numerical model

In this study, the daily measured inflow and sediment data obtained from 2007 to 2017 during the first phase of the YZD project was selected to verify the model. The
The channel was based on the measured daily flow as the inflow condition. The measured daily flow was used as the inflow condition at the channel entrance. The normal water level of the dam of 1249.5 m was used as the downstream boundary condition. The median diameter was 0.015 mm, and the sediment data was converted based on the measured data of the Qingtongxia Reservoir (Chen, 2019; Jia et al., 2020). To stabilize the operation of the model, an initial condition of an overall water level of 1249.5 m was set.

The results are shown in Figures 6 and 7; an analysis of the results shows that the calculated water level of the model at the channel entrance obtained was 1250.76 m, and the actual observed water level was 1250.7 m, with a high hydrodynamic fitting accuracy. In the water delivery channel, the flow velocity and sediment transport rate were high, and no sedimentation occurred. In the reservoir, due to an increase in the flow area, the flow velocity and consequently the sediment-carrying capacity of the flow decreased, resulting in sedimentation. As seen in Figure 7, the flow velocity in the channel is large, and the sediment moves forward in a suspended state; hence, the proportion of sediment determines the sediment transport rate. The suspended sediment gradation based on the measured sedimentation of the Yellow River is presented in Table 1.

It can be seen from Table 1 that the proportion of sediment with a grain size of 0.025 mm is the largest, followed by a grain size of 0.005 mm, and that the proportion of sediment with a grain size of 0.01 mm accounts for 13.74%. In the ST (Sediment Transport) model, the simulated maximum sedimentation thickness was 4.95 m and
Table 1. Grading table of suspended sediment.

| Grain size (mm) | 0.25 | 0.1  | 0.05 | 0.025 | 0.01  | 0.005 |
|----------------|------|------|------|-------|-------|-------|
| Percentage (%) | 3.12 | 10.99| 20.76| 26.25 | 13.74 | 25.14 |

the simulated maximum sedimentation elevation was 1247.5 m at about 1800 m (i.e. the peak in Figure 6) from the dam site, and the calculated results coincided with the measured values. The results of a regression analysis were: MSE (Mean Squared Error) = 3.7 m, RMSE (Root Mean Squared Error) = 1.92 m, and $R^2$ (Coefficient of Determination) = 0.921. In conclusion, it is seen that the siltation range and thickness are in good agreement with the measured results. An overall comparative analysis shows that the calculated results obtained from the model have good accuracy and the parameters selected are reasonable. The model can thus be used to study reservoir sedimentation and the coordinated dispatch of water and sediment.

3. Research on sediment avoidance diversion

When the conditions are unfavorable for building a reservoir in the Yellow River, the reservoir is built around the river. The water from the water diversion project is rerouted into the reservoir which provides water for consumption. Such a reservoir is called an injection-type reservoir, and examples of such reservoirs include the Beiyangjian Reservoir, the Cuijiaxia Reservoir, and the Bailong River Water Diversion Project. These reservoirs have the advantages of low cost, large effective utilization coefficients of reservoir capacities, small dam heights, and low flood control pressure. However, there are also some disadvantages, such as a small annual water diversion, a weak self-purification functionality of reservoir water, and a large project area. The Yellow River has a large amount of sand, and with sudden floods, sand peaks rise and fall periodically. This often results in high sediment concentrations over short periods of flooding. Each flood causes a sudden increase in sediment concentration in rivers and reservoirs, resulting in a reduction of the effective reservoir capacity. This situation is unacceptable for water supply projects. Therefore, a concept of sediment avoidance diversion is proposed for an injection-type reservoir built along the Yellow River. Sediment avoidance diversion means that the diversion structures stop working to prevent a large amount of sediment from entering the reservoir during a period of high sediment concentration in the Yellow River. This not only ensures the water quality requirements of the water supply reservoir but also prevents loss of reservoir capacity and slows down sediment depositions in the reservoir. The specifics of the process of sediment avoidance diversion research are described below.

Figure 8 shows the variations of the monthly average sediment concentrations over several years. From an analysis of the Yellow River sedimentation data, it is found that the annual average sediment content of the Yellow River is 4.01 kg/m$^3$. Although the average sediment concentration is relatively large, the higher sediment concentrations occur primarily during the flood season. For example, in 1973 and 1986, the sediment concentration reached 890 and 870 kg/m$^3$, respectively, and the historical maximum sediment concentration was 1110 kg/m$^3$ (June 24, 1971) (Li, 1998). Per design, when the sediment concentration of the Yellow River is more than 30 kg/m$^3$, the pumping station stops operations and provides a regulatory time of 20 days, to reduce abrasive wear of the water pump and sedimentation in the reservoir. For sediment concentrations of 30, 15, and 10 kg/m$^3$, the factors that affect the sand content of the Yellow River were studied and analyzed, to provide a reference for the theory of sediment avoidance diversion.

Through the analysis of the measured water and sediment data of Qingtongxia Station on the Yellow River, it can be seen from Figure 9 that the sediment content is extremely high, and that the weight of sand transported within a few days accounts for most of the weight of the sand transported over the entire year. It is observed from the data that the largest proportion of sediment weight over 30 kg/m$^3$ is 77%, the annual average sediment weight is more than 40%, and the average annual duration is only 4 days. The largest proportion of sediment weight over 15 kg/m$^3$ is 88%, the annual average sediment weight is more than 55%, and the average annual duration is only 13 days. The largest proportion of sediment weight over 10 kg/m$^3$ is 90%, the annual average sediment weight is more than 60%, and the average annual duration is only 19 days.
In this study, the sediment content ranging from 10 to 30 kg/m³ was divided into two intervals, 10–15 kg/m³ and 15–30 kg/m³, and the number and proportion of days were calculated for each interval. The results are shown in Figure 10. The statistical results show that from 1991 to 2001, the sediment content was concentrated in the interval of 15–30 kg/m³, with an average annual ratio of more than 53%; from 2002 to 2012, the sediment content changed to being more concentrated in the interval of 10–15 kg/m³, with an average annual proportion of more than 54%.

Figure 11 shows that the days the sediment concentrations were greater than 10 kg/m³ over 46 years. Combining the data presented in Figures 10 and 11, it is inferred that a sediment content greater than 10 kg/m³ accounts for 60% of the total sediment over the year, at the Qingtongxia Hydrological Station. However, the duration of this sediment content is very short, implying that the duration of flooding is short and that the sand content is high. The high sand content occurs mainly during July and August. For injection-type water supply reservoirs, the water diversion time can be adjusted based on the design conditions. If the days with high sediment content are avoided by water diversion, then the average annual sand content in the reservoir can be greatly reduced. To achieve such a reduction of the annual average sand content, the results show that by avoiding 4 days of sediment content greater than 30 kg/m³, the annual sediment concentration can be reduced to 1.99 kg/m³. Similarly, avoiding 13 days of sediment content greater than 15 kg/m³ reduces the annual average sediment concentration to 1.49 kg/m³, and avoiding 19 days of sediment content greater than 10 kg/m³ reduces the annual average sediment concentration to 1.25 kg/m³.
4. Study on the regularity of water and sediment in the channel and reservoir area during sediment avoidance diversion

4.1. Conditions for numerical simulations

According to the theory of sediment avoidance diversion and the actual situation of the reservoir, three sediment transport schemes were set up. The first transport scheme, named operating condition 1 (C1), involved avoiding sediment transport conditions of greater than 10 kg/m³; the sediment content of this operating condition is consistent with the measured sediment content during the first phase of the YZD project. The average annual sediment concentration for the C1 condition is 1.25 kg/m³. The second transport scheme avoided sediment transport during periods for which the sediment content was greater than 15 kg/m³, and was named condition 2 (C2), and used as the main coordinated dispatch of water and sediment condition. The average annual sediment concentration for the C2 condition is 1.49 kg/m³. The third transport scheme, named operating condition 3 (C3), avoided sand transport during periods for which the sediment content was greater than 30 kg/m³, and the average annual sand content for this operating condition was 1.99 kg/m³. The results are shown in Table 2, and it can be seen that the high sand transport months during the year are from May to November, with the largest sand transport month being August.

4.2. Analysis of water and sediment for a constant water level

4.2.1. Analysis of maximum thickness of deposition in diversion channel

The maximum siltation in a channel for each condition described above is shown in Figure 12. For C1, when the reservoir operational water level is higher than 1252.60 m, the water conveyance channel is silted. If

Table 2. Monthly average sand content of series of three design sand transport conditions.

| Month | C1 (kg/m³) | C2 (kg/m³) | C3 (kg/m³) |
|-------|------------|------------|------------|
|       | 0.239      | 0.525      | 0.5         |
| 2     | 0.271      | 0.505      | 0.53        |
| 3     | 0.401      | 0.769      | 0.5         |
| 4     | 0.43       | 0.536      | 0.7         |
| 5     | 0.893      | 1.804      | 2.23        |
| 6     | 1.079      | 2.374      | 3.73        |
| 7     | 1.729      | 3.088      | 7.11        |
| 8     | 3.271      | 3.543      | 8.28        |
| 9     | 1.661      | 2.277      | 3.86        |
| 10    | 1.395      | 1.432      | 1.7         |
| 11    | 1.061      | 1.094      | 0.45        |
| 12    | 0.434      | 0.833      | 0.41        |

Figure 12. Relationship between reservoir water level and channel sedimentation thickness.
the design water level is 1255.80 m, the maximum siltation of the water conveyance channel is 3.481 m. For C2, when the reservoir operational water level is higher than 1251.87 m, the water conveyance channel is silted. If the design water level is 1255.80 m, the maximum siltation of the water conveyance channel is 3.771 m. For C3, when the reservoir operational water level is higher than 1251 m, the water conveyance channel is silted. If the design water level is 1255.80 m, the maximum siltation of the water conveyance channel is 4.8 m.

4.2.2. Analysis of the development of sediment deposition in the reservoir area

Figures 13, 14, and 15 show the siltation conditions for the above three working conditions for reservoir design water levels of 1249.5 m and 1255.8 m.

Figure 15 shows the simulation results of the siltation of the reservoir during 70 years of operation at different reservoir operating water levels for C3. During 70 years of reservoir operation, sedimentation has been continuously advancing towards the front of the dam in the form of a siltation delta. Compared with Figures 13 and 14, the siltation front of C3 has advanced to the front of the dam after 50 years of operation. C3 has a high sediment content, and sedimentation and propulsion are evident, which are not conducive to the operation of the reservoir. For C1 and C2 with a reservoir water level lower than 1252 m, and C3 with a reservoir water level lower than 1251 m, there is no siltation of the water delivery channel (distance from the dam ranges from 2950 m to 4180 m) during the operation of the reservoir for 10–70 years. With a rise in water level, sedimentation in the water conveyance channel gradually silted from the end to the
mouth of the canal, and the maximum siltation reached an elevation of 1252.8 m, at a depth of approximately 4.8 m. A large amount of sediment was deposited in the reservoir, and the maximum siltation depth reached 22.3 m. Since the water level in front of the dam remained unchanged, the sedimentation in the reservoir was in the form of a delta siltation.

The distance between sedimentation at the front edge of the reservoir and the dam site for each working condition is shown in Figures 16 and 17, and the variation of the maximum depth of siltation is shown in Figures 18 and 19.

It can be seen from Figures 16 and 17 that for higher water levels, the siltation storage volume is larger, and the advancement of the siltation front towards the dam is slower. If the reservoir is operated at 1255.8 m at the second-stage design water level, for C1, the siltation front is as yet 440 m away from the dam site after 70 years of operation. For C2, after 70 years of operation, the siltation front is 350 m away from the dam site. For C3, regardless of the operating water level, siltation occurs in front of the dam after 70 years of operation. With time, the rate of increase of sediment elevation gradually declines. The main reason is that for longer operational durations of the reservoir, the sedimentation front is closer to the main reservoir area. This leads to a larger deposition area which results in a slower rate of growth of the sediment elevation. Figures 18 and 19 show that the maximum siltation thickness increases with an increase in the number of operating years, and the corresponding siltation thickness increases as the water level rises. For the same water level, the deposition thickness for C3 is greater than that for C1, mainly because the average sediment concentration of C3 is greater than that of C1.

5. Research on the coordinated dispatch of water and reservoir sedimentation

The above simulation results show that for low water levels in a reservoir, siltation does not occur in the channel. When the reservoir operates at a high water level, it is
Figure 15. Deposition morphology along the reservoir at different operation times for C3, for reservoir water levels of 1249.5 m (a), and 1255.8 m (b).

Figure 16. Distance between the siltation front and dam site for different reservoir water levels in C1.
affected by backwater, the water depth in the channel is large, the gradient is small, and the flow rate is slow, all of which lead to channel siltation. The effective depth in the open channel is also reduced, which affects the water delivery function of the channel. Therefore, it is necessary to study the reservoir control plan to alleviate channel siltation and ensure smooth flow in the channel. Based on the characteristics of water and sediment in the reservoir, two control strategies have been formulated for C2. One strategy is to lower the water level of the reservoir during periods of high sand content. Under this strategy, when the sand content is high, the water level of the reservoir is lowered, and the flow velocity is increased so that the sediment can be directly transported into the reservoir to reduce channel siltation. This strategy is referred to as HCLL (High Content Lower Level). The other strategy is to lower the water level during the low sand content period. Under this strategy, the reservoir operates at a high water level during the high sedimentation period, allowing sediment to accumulate in the channel, and the water level is lowered during the low sedimentation period to scour the channel sediment. This strategy is referred to as LCLL (Low Content Lower Level).

5.1. Formulation of reservoir operation scheme

As can be seen in Figure 12, for the control strategy C2, the water conveyance channel is silted when the reservoir operational water level is higher than 1251.87 m. In the HCLL strategy, the lowest water level of schemes 1–4 is 1252 m during the periods of high sediment concentration, and the water levels in other months are determined
by instantaneous decreases and multi-level and symmetrical changes. The exact water level is obtained from many attempts to determine the level. From the results of the calculations based on the above scheme, it can be seen that sediment deposition does not occur when the water level is 1252 m, but if the highest water level does not decrease, significant sedimentation can still occur. Therefore, in scheme 5, the primary intent is to lower the maximum water level and raise the minimum water level. The HCLL strategy is shown in Figure 20. Due to high water demand from June through September, the guarantee of water supply with this strategy is low. Therefore, the LCLL strategy is proposed, which is to increase the operating water level of the reservoir from June through September to guarantee the water supply of the reservoir, and then wash away the sediment deposited in the channel in other months. The LCLL strategy is shown in Figure 21.

### 5.2. Silting analysis of water conveyance channel in different schemes

From the simulation of the two strategies, it was found that the operational scheme of low water level in the period of high sediment concentration does not cause large-scale silting during the period of high sediment concentration. During the period of high sediment concentration, the water conveyance channel rapidly accumulates silt, which is scoured rapidly with a decrease in

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**Figure 19.** Maximum deposition depth for different reservoir water levels in C1.

**Figure 20.** HCLL operational plan design diagram.

Note: the highest water level of schemes 1, 2, and 3 is 1255.8 m; the highest water level of scheme 4 is 1254.8 m; scheme 5 has only 1253 and 1254 m.
Figure 21. LCLL operational plan design diagram.

Note: the highest water level of schemes 1, 2, and 3 is 1255.8 m, and the lowest water levels are 1254, 1253, and 1252 m in turn; scheme 4 has only water levels of 1254.8 and 1254 m.

Figure 22. HCLL diagram of channel siltation in the operation plan.

Figure 22 shows the results of schemes 1, 2, 3, 4, and 5 in the HCLL strategy. By comparing the results of schemes 1–4, it is seen that the sand control effect of this strategy is not very effective. In these schemes, the channel sediment is washed away by lowering the water level in the high sediment transport month, which is characterized by an increase in the number of sediment control months. The results show that as the number of months of sediment control increases, the channel siltation elevation changes as depicted by curve 1 (i.e. the channel siltation line when the reservoir water level is 1255.8 m). However, if the highest water level of the reservoir is not decreased, the channel remains silted up at all times. For scheme 5, due to the lowering of the maximum water level, the channel siltation undergoes a fundamental change. During the months with low sand content, the channel does not silt up, and only slight siltation occurs in the months with the highest sediment content. The maximum siltation depth is 0.32 m, which is washed away again during the months with low sand content. Therefore, in the HCLL strategy, scheme 5 is recommended as the sediment control plan of the YZD water conveyance channel.

Figure 23 shows the results of schemes 1, 2, 3, and 4 in the LCLL strategy. A comparison of the results of these four schemes shows that the siltation height at the inlet is determined by the maximum water level of the reservoir. In schemes 1–3, the highest water level of the reservoir is 1255.8 m, and the maximum siltation height is 3.05 m; in scheme 4, the highest water level of the reservoir is 1254.8 m, and the maximum siltation height is 2.07 m. The similarity of the four schemes is that the reservoirs are operated in the same manner, i.e. high water level operation during high sedimentation periods and low water level during the period of low sediment concentration.
water level operation during low sedimentation periods. From simulations, it is found that lowering the water level during the low sedimentation period always washes away the silt deposited at the channel entrance. Hence, as long as the maximum water level of the reservoir is optimized, the sedimentation height at the channel entrance can be controlled during the high sedimentation period when the strategy is applied. Therefore, in the LCLL strategy, scheme 4 is recommended for the control of the water delivery channel of the YZD project.

Figure 23. LCLL diagram of channel siltation in the operation plan.

Figure 24. Sediment deposition in the reservoir area of scheme 5 for the HCLL strategy.

Figure 25. Sediment deposition in the reservoir area of scheme 4 in the LCLL strategy.
5.3. Analysis of siltation in the reservoir area of the recommended scheme

Figures 24 and 25 show the sediment deposition for the recommended schemes under different strategies. Based on the recommended scheme 5 for the HCLL strategy, the reservoir operation was simulated for 20, 30, 40, and 70 years, and the distance between the sedimentation front and the dam site obtained was 1330, 1180, 730, and 430 m (Figure 24), respectively. It can be seen that the channel siltation for scheme 5 is beneficial and that the front edge of sedimentation is at a large distance from the dam site. For the recommended scheme 4 for the LCLL strategy, the reservoir operation was simulated for 20, 30, 40, and 70 years, and the distance between the sedimentation front and the dam site obtained was 1240, 937, 774, and 180 m (Figure 25), respectively. In the recommended scheme 4, due to the long duration of the low water level period, the reservoir area is, in general, close to the dam.

6. Conclusions

Based on an injection-water supply project on a sediment-laden river, this study proposes a theory of sediment avoidance diversion and coordinated dispatch of water and sediment. The application of the theory to the YZD project resulted in the following findings:

1. Sediment avoidance diversion can effectively reduce the sediment content in a reservoir. In the case of the Yellow River, there are 13 days each year when the sand content is greater than 15 kg/m³. The mass of sediment transported during these 13 days accounts for 88% of the annual mass of sediment transported, and more than 55% of the average mass of sediment that was transported during the year. If sediment avoidance diversion is applied, the reservoir did not receive water from the river for only 13 days per year, which can reduce 55% of the sediment into the reservoir.

2. A coordinated plan for the dispatch of water and sediment can effectively alleviate siltation in water diversion channels. The HCLL strategy can reduce the canal siltation height from 3.05 m to 0.32 m, but the disadvantage of this strategy is that the reservoir water level is low in summer and a guaranteed water supply is a major challenge. The LCLL strategy can reduce the canal siltation height from 3.05 m to 2.07 m. This strategy has a high reservoir water level, a guaranteed high water supply rate, and a low sand content at the water inlet.

This study demonstrates that the theory of sediment avoidance diversion and the scheme of coordinated dispatch of water and sediment can extend the service life of a reservoir, reduce sediment deposition in water channels, and ensure the quality of water at the inlet, and thus lays the foundation for reaping comprehensive benefits from a reservoir. However, in this study, a one-dimensional numerical model was used, which does not accurately reflect the siltation characteristics. It is anticipated that the GPU solution of the three-dimensional model can be realized in follow-up research.

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Nomenclature

The following symbols are used in this paper:

\( a \) thickness of bed layer (m);
\( A \) the cross-sectional area (m²);
\( A_{o,j} \) the surface area between grid point \( j-1 \) and \( j \) (m²);
\( A_{o,j+1} \) the surface area between grid point \( j \) and \( j+1 \) (m²);
\( B_s \) the river width (m);
\( c \) concentration of suspended sediment (kg/m³);
\( C \) the Chezy coefficient (m\(^{1/2}\)/s);
\( c_a \) reference sediment concentration (kg/m³);
\( d \) grain size (m);
\( D \) total flow depth (m);
\( g \) acceleration due to gravity (m/s²);
\( h \) the sectional water level (m);
\( k \) equivalent sand roughness (mm);
\( R \) hydraulic radius (m);
\( R^2 \) coefficient of determination (-);
\( q \) the lateral inflow (m³/s);
\( q_s \) suspended load transport rate (m³/s);
\( Q \) the sectional discharge (m³/s);
\( s \) specific gravity of sediment (N/m³);
\( t \) the time coordinates (s);
\( RMSE \) root mean squared error (m);
\( MSE \) mean squared error (m);
\( \text{Nomenclature} \)
\( u \) the average flow velocity over the cross-sectional area (m/s);
\( u' \) bed shear velocity or friction velocity (m/s);
\( w \) fall velocity (m/s);
\( x \) the spatial coordinates (m);
\( z \) Rouse number (-);
\( \alpha \) the vertical velocity distribution coefficient (-);
\( \alpha_j \) coefficient matrix (-);
\( \beta \) dynamic friction coefficient (-);
\( \beta_j \) coefficient matrix (-);
\( \gamma_j \) coefficient matrix (-);
\( \theta \) dimensionless bed shear stress (-);
\( \theta_c \) critical dimensionless bed shear stress (-);
\( \theta' \) dimensionless skin friction (-);
\( \delta_j \) the right-hand side of the equation (-);
\( \Phi_b \) dimensionless bed load sediment transport rate (-);
\( v \) coefficient of kinematic viscosity \(( \times 10^{-6} \text{ m}^2/\text{s})\);
\( \Delta x_j \) the distance between point \( j - 1 \) and \( j + 1 \) (m).

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