

# Study on Inverted AAO+IFAS Process for Treating Simulated Rural Domestic Sewage

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**Abstract:** A wastewater treatment reactor employing an inverted AAO+IFAS process was designed and constructed. Continuous operation and optimization were conducted using simulated rural domestic wastewater, aiming to enhance denitrification efficiency without external carbon sources and validate the process's applicability. The total effective volume of the reactor's anoxic, anaerobic, and aerobic zones was 154.1 L. Simulated wastewater was introduced into the anoxic and anaerobic zones via point-injection to regulate carbon input. The anoxic zone employed an elliptical flow pattern combined with MBBR media (9% packing density) to enhance denitrification, while the aerobic zone utilized flexible media (20% packing density) to improve nitrification stability. Following a 17-day pre-inoculation period for the fillers prior to operation, and after 52 days of continuous operation with multiple rounds of optimization, the average COD in the effluent over the subsequent 21 days was 38.5 mg/L. During the later stage, the average concentrations of ammonia nitrogen,  $\text{NO}_2^-$ -N, and  $\text{NO}_3^-$ -N concentrations in the later stage were 6.5 mg/L, 0.9 mg/L, and 4.6 mg/L, respectively, achieving a total inorganic nitrogen removal rate of 61.19%.

**Keywords:** Rural domestic wastewater; Inverted AAO; IFAS; Elliptical flow; Point-by-point influent

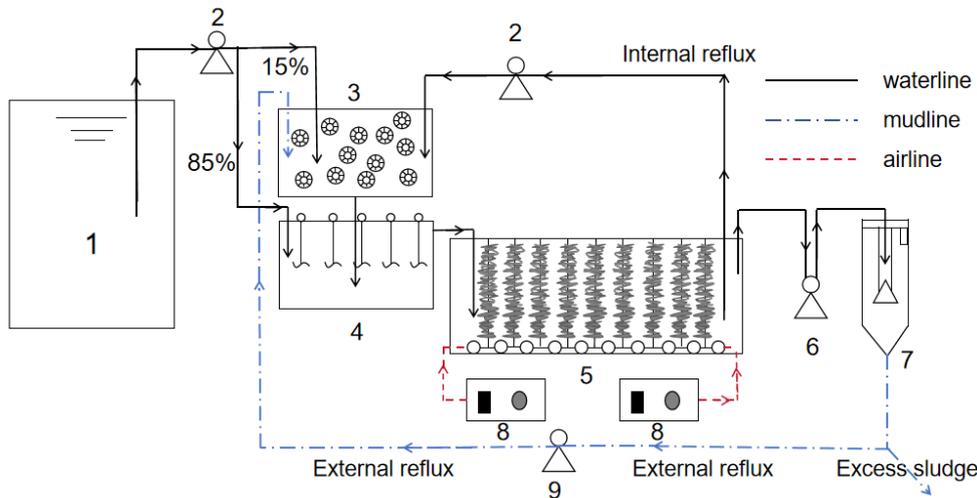
## 1. Introduction

In recent years, China's rapid economic development and advancing urbanization have led to a sharp increase in domestic wastewater discharge. Wastewater treatment plants face multiple challenges, including suboptimal operational performance, regional development imbalances, and demands for sustainable development<sup>[1]</sup>. These issues are particularly pronounced in rural areas, which differ significantly from urban settings. Although individual rural wastewater discharges are small, the total volume is substantial<sup>[2]</sup>. Compared to urban settings, rural wastewater exhibits characteristics such as dispersion, small-scale generation, and high variability. Weak collection systems and operational capabilities often lead to conventional processes facing issues like heavy reliance on external carbon sources, high energy consumption, and elevated maintenance costs in small-scale scenarios<sup>[2]</sup>. The inverted AAO process enhances nitrogen removal by incorporating a pre-anoxic tank into the traditional AAO sequence. It offers advantages such as simultaneous nitrogen and phosphorus removal and low operating costs<sup>[3]</sup>. This aligns with recent requirements for rural domestic wastewater treatment that emphasize site-specific approaches, categorized strategies, and operationally achievable solutions<sup>[4,5]</sup>. This study aims to optimize nitrogen removal efficiency and validate the feasibility of applying the inverted AAO+IFAS process, supporting low-cost operation and maintenance of rural domestic wastewater treatment to achieve stable compliance.

## 2. Experimental Setup and Materials

### 2.1 Reactor Structure

The reactor, based on the inverted AAO+IFAS process, sequentially comprises an anoxic tank, anaerobic tank, aerobic tank, and secondary sedimentation tank, as shown in Figure 1. Operation is driven by peristaltic pumps. Inlet water is supplied via point-by-point feeding<sup>[6]</sup>.



1-Raw water tank 2-Multi-channel peristaltic pump 3-Anoxic zone 4-Anaerobic zone 5-Aerobic zone  
 6-Lift pump 7-Settling tank 8-Air pump 9-Sludge return pump

Figure 1 Inverted AAO + IFAS process flow diagram

The anoxic tank has an effective volume of approximately 52.3 L, featuring an elliptical structure with a nested 6 L small ellipse. A propeller is installed inside, creating a ring-shaped channel configuration that enhances reaction efficiency<sup>[7]</sup>. Inflow enters the smaller ellipse via overflow from the outer ring channel, ultimately overflowing at the tank center and discharging downward. The tank is filled with MBBR media at a packing ratio of approximately 9%.

The anaerobic tank dimensions are  $L \times W \times H = 0.60 \text{ m} \times 0.34 \text{ m} \times 0.20 \text{ m}$ , with an effective volume of 30 L. Positioned below the anoxic tank, it receives the majority of influent flow and overflow discharge from the anoxic tank. A mechanical agitator is installed internally.

The aerobic tank dimensions are  $L \times W \times H = 0.96 \text{ m} \times 0.34 \text{ m} \times 0.28 \text{ m}$ , with an effective volume of approximately 71.8 L. Air is distributed via a diffuser pipe, with elastic fillers vertically fixed on the pipe at a packing ratio of about 20%. This media features a central filament bundle core wrapped in fibers to form a braided configuration, reducing excess sludge production<sup>[8]</sup>. The resulting IFAS system improves sludge settling performance<sup>[9]</sup>. Level sensors installed at the structure's top maintain a narrow water level range, controlling intermittent operation of lift pumps.

A vertical flow settling tank with an effective volume of approximately 50 L is installed at the end. Effluent from the aerobic tank enters the central pipe via a lift pump, falls onto the baffle plate, and exits through an opening at the tank bottom connected to a sludge return pump (which intermittently recirculates sludge via a timer-controlled outlet). A T-joint in the pipeline facilitates both sludge return and excess sludge discharge.

## 2.2 Experimental Water, Sludge Inoculation, and Analytical Methods

### 2.2.1 Experimental Water

The water used in this study was artificially synthesized to simulate rural domestic sewage. Glucose ( $\text{C}_6\text{H}_{12}\text{O}_6$ ) as the carbon source, ammonium chloride ( $\text{NH}_4\text{Cl}$ ) and urea ( $\text{CH}_4\text{N}_2\text{O}$ ) provided ammonia nitrogen and organic nitrogen respectively as nitrogen sources, and potassium dihydrogen phosphate ( $\text{KH}_2\text{PO}_4$ ) served as the primary phosphorus source<sup>[10]</sup>. The experimental wastewater composition is shown in Table 1.

Table 1 Inlet Water Quality

Contaminant	Concentration (mg/L)	Mean (mg/L)
COD	140–552	276
$\text{NH}_4^+\text{-N}$	15.6–33.4	22.8
$\text{CH}_4\text{N}_2\text{O}$	10.0–11.5	10.7
$\text{NO}_3^-\text{-N}$	0.3–1.0	0.7
$\text{NO}_2^-\text{-N}$	0.0003–0.3674	0.0853
TP	3.5–6.9	4.8

Note: Urea ( $\text{CH}_4\text{N}_2\text{O}$ ) concentration represents the theoretical dosage value; concentrations of other parameters are experimental values.

### 2.2.2 Sludge inoculation

The inoculum sludge was collected from the Third Wastewater Treatment Plant in Lingchuan County, Guilin City, Guangxi. This plant primarily treats wastewater from the Huajiang Campus of Guilin University of Electronic Technology and surrounding townships. The sludge characteristics are as follows: sludge concentration approximately 6000–8000 mg/L, SV30 approximately 70–80%. The collected sludge was taken from the aerobic tank just before entering the secondary sedimentation tank.

### 2.2.3 Experimental Analysis Items and Methods

Water quality analysis followed the corresponding Chinese national standards (GB) and environmental industry standards (HJ) issued by the Ministry of Ecology and Environment. The specific method for each parameter is provided in Table 2. These officially promulgated standards are technically aligned with international methodologies, ensuring the reliability and comparability of the data.

Table 2 Test Items and Analytical Methods

Test Items	Analysis Method	Source
NH <sub>4</sub> <sup>+</sup> -N	Nessler's Reagent Spectrophotometric Method	HJ 535-2009
NO <sub>2</sub> <sup>-</sup> -N	N-(1-Naphthyl)-Ethylenediamine Spectrophotometric Method	GB 7493-87
NO <sub>3</sub> <sup>-</sup> -N	Ultraviolet Spectrophotometric Method	HJ/T 346-2007
COD	Potassium Dichromate Method	HJ 828-2017
TP	Ammonium Molybdate Spectrophotometric Method	GB 11893-89
MLSS	Weight Method	-
DO	Electrochemical Probe Method	-
pH	Precision Test Paper	-

## 3. Experimental Process and Results

Reactor operation comprises three phases: pre-inoculation, trial operation, and optimization (subdivided into initial and advanced stages).

### 3.1 Pre-Inoculation Stage

He Qiang et al.<sup>[11]</sup> Research on pre-inoculation treatment of anaerobic biofilm reactor fillers revealed that aerobic pre-inoculation promotes the formation of favorable filamentous microbial structures on the filler surface. This structure facilitates the attachment and growth of anaerobic microorganisms, significantly enhancing the start-up rate of anaerobic biofilm reactors. Zhang Jinsheng et al.<sup>[12]</sup> In external circulation three-phase bio-fluidized bed carrier biofilm formation experiments, it was found that low sludge loading rates are beneficial for biofilm formation in fluidized beds.

The pre-inoculation process was conducted prior to reactor operation, lasting 17 days. Sludge with an MLSS of approximately 1900 mg/L was used for inoculation via a sealed aeration method. The COD concentration of the feed water was gradually increased from 200 mg/L to 290 mg/L. Water temperature was maintained between 23–27°C. Water was changed every two days. During water changes, aeration was stopped, the system was allowed to settle for 30 minutes, the supernatant was discarded, the sludge retained, and fresh feed water was added to restore the original volume. As shown in Figures 2 and 3, pre-filming enabled initial attachment on the packing. Based on this pre-filming stage, the packing exhibited a dense, uniform biofilm after long-term operation.

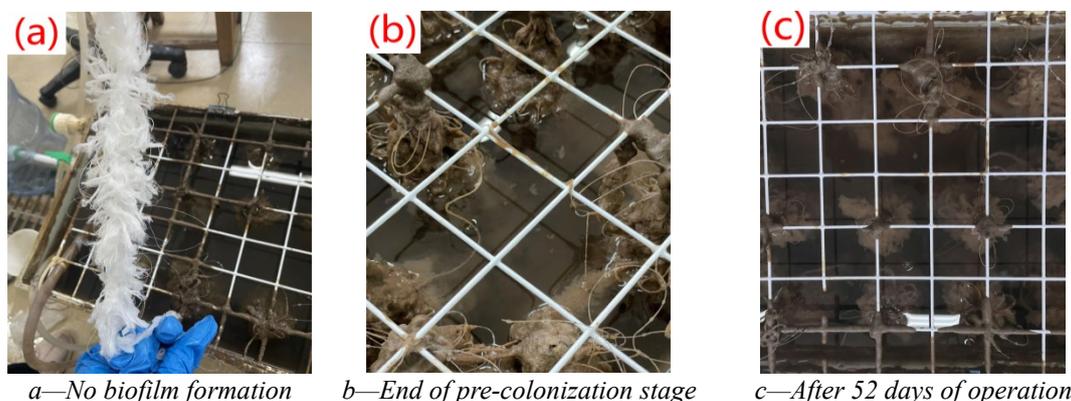


Figure 1 The membrane growth situation of the elastic filler

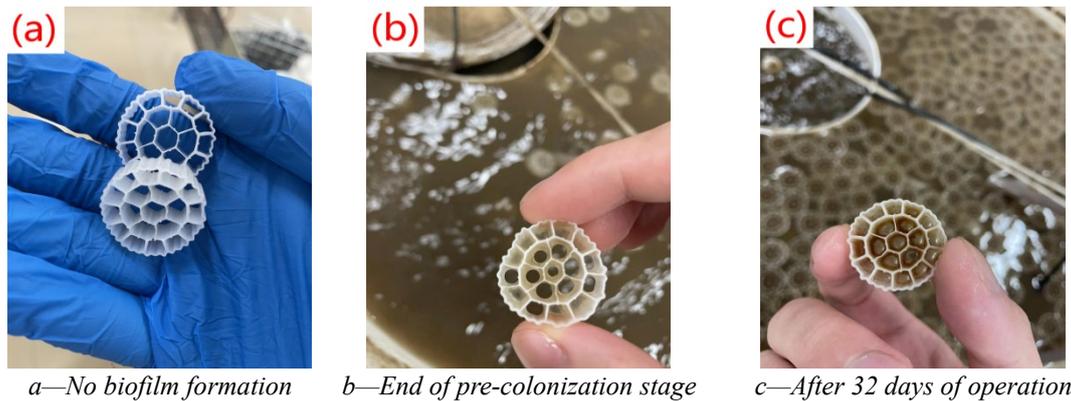


Figure 3 The membrane growth situation of the MBBR filler

### 3.2 Trial Operation Phase

Days 1–12 constitute the reactor startup and stabilization phase. The initial influent flow rate was 144 L/d (corresponding to HRT = 25.7 h). Sludge was added to achieve a sludge concentration of approximately 3100 mg/L. The flow rate was increased by 20% every two days until reaching the target flow rate of 288 L/d (corresponding to HRT = 12.8 h), after which it was maintained at a constant level. The influent temperature was maintained at  $24.0 \pm 3.0^\circ\text{C}$ , with an internal recirculation rate of 200%. Operational monitoring revealed dissolved oxygen (DO) levels exceeding 2 mg/L in the anoxic zone, accompanied by abnormally high oxidation-reduction potential (ORP).

To ensure uniform sludge concentration and prevent localized settling in the anaerobic tank, submersible propeller mixers were installed at the bottom<sup>[13]</sup>. The mixer was installed at an upward angle of  $35\text{--}45^\circ$  relative to the horizontal, directed toward the effluent outlet of the anaerobic tank. This orientation allowed the jet to rise directly to the overflow control surface. However, the resulting liquid surface disturbance rapidly homogenized the entrained DO throughout the tank, severely disrupting the anaerobic environment and necessitating improvements.

Under simulated wastewater conditions using tap water as the substrate, the initial dissolved oxygen (DO) in tap water was approximately 6.8–7.2 mg/L. Within about 7 hours after chemical addition, the barrel rapidly transitioned from aerobic to anaerobic conditions and underwent acidification, manifested as milky turbidity, massive suspended solids formation, and a pH drop to approximately 6. This resulted in partial consumption of biodegradable organic matter before entering the reactor. Simultaneously, the low pH environment suppressed nitrifying bacteria activity<sup>[14]</sup>. The combined effects directly weakened the reactor's denitrification performance. Experiments revealed that nitrogen gas deoxygenation of simulated wastewater significantly slowed turbidity increase and acidification during storage. However, due to high nitrogen consumption and costs, chemical deoxygenation was selected for improvement.

### 3.3 Optimization Operation Phase

#### 3.3.1 Initial Phase

Starting on Day 13, improvements were implemented to address issues identified during the trial operation phase. All modifications were completed by Day 21, with additional sludge added to achieve a reactor sludge concentration of approximately 2000 mg/L. The reactor schematic is shown in Figure 4.

(1) The anaerobic tank pusher was removed. A  $90^\circ$  elbow and PVC pipe were installed at the outlet, positioning the pipe opening 5 cm above the tank bottom to form a communicating vessel that utilizes the liquid level difference to drive effluent discharge. This configuration prevents higher sludge concentration in the anaerobic tank compared to other tank sections and reduces surface disturbance and oxygen intrusion.

(2) Replace the elliptical anoxic tank's mixer with a low-power model to reduce shear force damage to sludge flocs. Simultaneously, the reduced flow velocity decreases turbulence within the tank, mitigating excessive dissolved oxygen levels.

(3) Raw water barrels adopt alternating use of two barrels + pre-treatment chemical deoxygenation with stirring eliminated: Two 200 L barrels are used in rotation. The idle tank pre-doses a small amount of

anhydrous sodium sulfite (5-6 g per 200 L) without catalysts. After approximately 12 hours of static reaction, the initial DO of tap water is reduced from 6.8–7.2 mg/L to 4.2–4.5 mg/L. Simulated water quality chemicals are then added, and the water is immediately put into use.

- (4) Sodium bicarbonate ( $\text{NaHCO}_3$ ) was added to the raw water tank and aerobic tank as a pH buffer.
- (5) The internal recirculation ratio was increased from 200% to 468%.

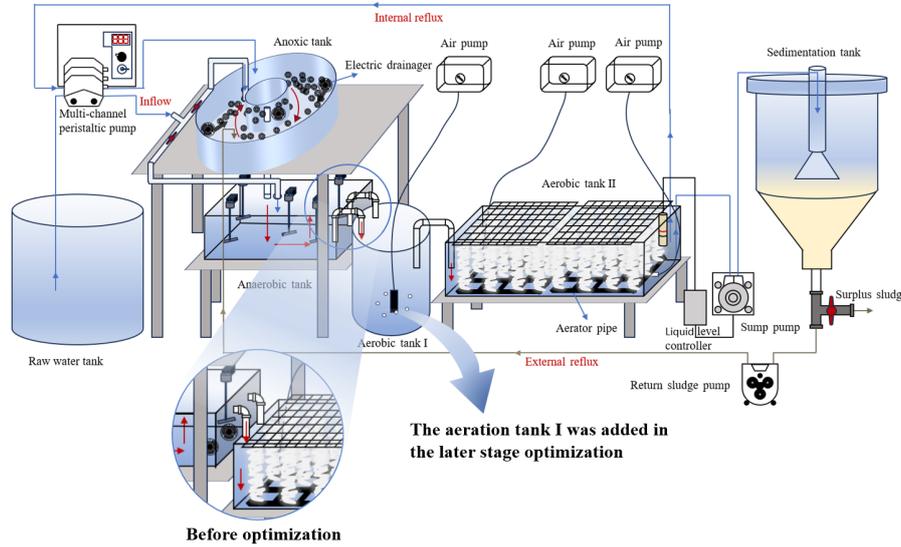


Figure 4 Reactor schematic diagram

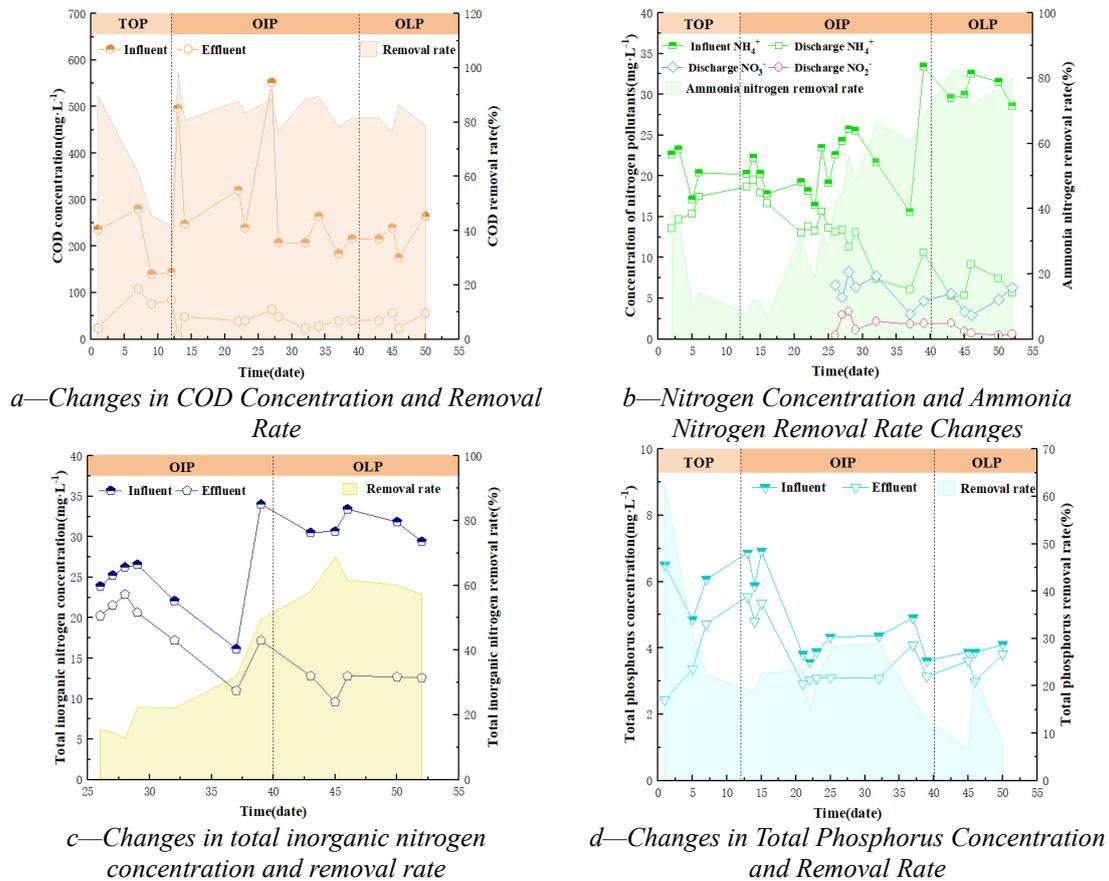


Figure 5 Changes in various pollutant indicators

Note: TOP—Trial Operation Phase; OIP—Optimization of Initial Phase; OLP—Optimization of Later Phase

As ammonia nitrogen removal efficiency significantly underperformed expectations, a small amount of commercial aerobic bacteria was introduced into the reactor on Day 29 under operating conditions maintaining flow rate at 310–330 L/d, average HRT of approximately 11.5 h, internal recirculation ratio of 468%, and external recirculation ratio of 55–85%. During the raw water tank water change period, sodium bicarbonate was added to both the raw water tank and aerobic tank to replenish alkalinity.

Adding deoxidizing agents to the influent tank effectively extended the storage time of simulated sewage and mitigated whitening and acidification of the influent. During this phase, the turbidity of the sludge supernatant was significantly lower than during the trial operation phase, with SV30 maintained at 15–20% and DO in the anoxic tank ranging between 0.2–0.5 mg/L. Due to the intermittent operation mode of the lift pump from the aerobic tank to the sedimentation tank, the pump power consumption was relatively high during operation. This led to sludge loss in the reactor, resulting in the reactor sludge concentration consistently remaining below 2300 mg/L. Concurrently, it is speculated that due to the prolonged generation time of nitrifying bacteria and insufficient bacterial population [15], the nitrification efficiency of suspended sludge was low. Consequently, nitrification efficiency became overly reliant on biofilm activity [16]. At DO levels of 3–4 mg/L in the aerobic section, outer-layer biofilm dominated nitrification. Ammonia nitrogen removal rates gradually increased during this phase. Using data from day 31 onwards as a reference (Fig. 5b), the average ammonia nitrogen removal rate reached approximately 63.7%, with an average effluent concentration of 8.79 mg/L. However, overall reactor nitrification efficiency remained insufficient.

### 3.3.2 Later Stage

On the 40th day of operation, to address the issue of ammonia nitrogen effluent failing to meet the Municipal Wastewater Discharge Standard Grade A (GB18918-2002), a cylindrical aerobic tank (designated as Aerobic Tank I, effective volume approx. 13 L) was added upstream of the existing aerobic tank. This new tank, connecting via a communicating vessel, has a volume approximately 18% that of the original aerobic tank (Aerobic Tank II). The newly added Aerobic Tank I contained no packing material. This tank primarily oxidized a portion of the ammonia nitrogen to alleviate nitrification load, providing a buffer and preparation for subsequent deep nitrification.

During this phase, influent flow rate and internal/external recirculation rates remained unchanged from the previous stage (though HRT adjusted to approximately 12.5 hours due to increased total volume). The influent ammonia nitrogen concentration was appropriately elevated while maintaining the same urea dosage, resulting in an average influent ammonia nitrogen concentration of approximately 29.25 mg/L (Fig. 5b). As the later-stage sludge concentration stabilized at approximately 1800 mg/L, the anaerobic tank dissolved oxygen (DO) remained below 0.2 mg/L, while the anoxic tank DO ranged between 0.2 and 0.5 mg/L. Compared to the earlier stage (after Day 31), the average ammonia nitrogen removal rate in this phase increased by approximately 14.2%, with an average effluent concentration of 6.48 mg/L (Fig. 5b). As shown in Figure 6, comparing the changes in nitrogen concentration along the reactor between the later stage and the period after Day 31 of the earlier stage [17], the  $\text{NO}_3^-$  concentration in the anoxic tank showed no significant difference from that in the aerobic tank during the earlier stage. However, in the later stage, the  $\text{NO}_3^-$  concentration in the anoxic tank was on average 1.6 mg/L lower than that in the aerobic tank. Simultaneously, the total inorganic nitrogen concentration stabilized at approximately 12.09 mg/L, indicating an overall improvement in the reactor's denitrification efficiency.

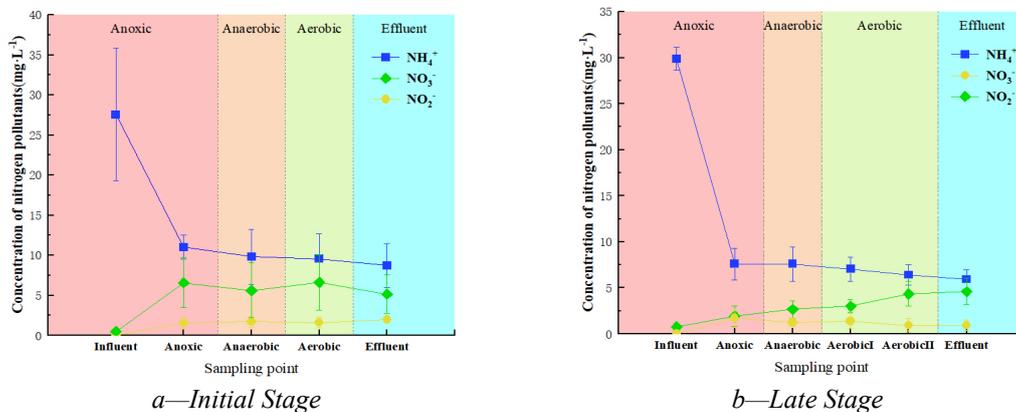


Figure 6 Optimize the removal effect of pollutants along the route during the operation stage

#### 4. Conclusion

(1) The point-injection design adopted in the inverted AAO+IFAS process of this reactor demonstrated stable COD removal performance, with an average effluent COD concentration of 38.5 mg/L over the final 21 days (Fig. 5a). Without external carbon source addition, the combined concentrations of  $\text{NO}_3^-$ -N and  $\text{NO}_2^-$ -N in the anoxic tank during the late stage averaged 3.7 mg/L (Fig. 6b). The primary bottleneck in nitrogen removal currently lies in the nitrification efficiency of the aerobic stage.

(2) During the later operational phase, while achieving an average ammonia nitrogen removal rate of 77.8%, the average concentrations of  $\text{NO}_2^-$ -N and  $\text{NO}_3^-$ -N in the effluent were 0.9 mg/L and 4.6 mg/L, respectively. The average ammonia nitrogen removal rate in this phase increased by approximately 14.2% compared to the earlier phase after day 31, while the total inorganic nitrogen removal rate simultaneously improved by 26.69%. This indicates that the synergistic effect of the elliptical circulation design and MBBR media achieves excellent denitrification performance.

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