



ARTICLE

Coupled Seepage-Stress Modelling of Drainage Pressure Relief in Underground Structures

Haonan Zhang¹, Qiqi Wu¹, Hangming Zhang¹, Liangxu Shen¹, Linhui Song² and Guoxiong Mei^{1,*}

¹Ocean College, Zhejiang University, Zhoushan, China

²School of Mathematical Sciences, Nanjing Tech University, Nanjing, China

*Corresponding Author: Guoxiong Mei. Email: 0022132@zju.edu.cn

Received: 11 July 2026; Accepted: 22 September 2026; Published: 19 October 2026

ABSTRACT: A coupled seepage-stress numerical model, based on the finite element method and groundwater seepage theory, is implemented to evaluate the effectiveness of drainage pressure relief for controlling basement uplift under varying groundwater and structural conditions. The model is used to systematically investigate the effects of groundwater-level fluctuations, the pressure-limiting total head of drainage holes, and diaphragm wall embedment depth on pore-water pressure, drainage demand, groundwater drawdown, and ground deformation. The results show that drainage pressure relief effectively reduces and stabilizes pore-water pressure beneath the foundation, thereby mitigating the risk of basement uplift. Under fluctuating groundwater conditions, a relatively high pressure-limiting total head allows drainage to be activated intermittently during short-duration high-water-level events, reducing unnecessary long-term discharge. In contrast, a lower control head provides stronger pressure relief but prolongs drainage and causes greater disturbance to the surrounding groundwater field. Drainage performance is also strongly influenced by the spatial relationship between the drainage holes and diaphragm wall, with higher discharge rates occurring near the wall because of shorter seepage paths and steeper hydraulic gradients. Under sustained high groundwater levels, increasing the diaphragm wall embedment depth reduces both groundwater drawdown and drainage demand by lengthening the seepage pathway and redistributing hydraulic gradients. In the simulated cases, increasing the embedment depth below the foundation base from 5 to 10 m reduces the groundwater drawdown from approximately 10 to 7.5 m. However, the associated ground-deformation response is non-monotonic, indicating that hydraulic-control efficiency cannot be evaluated independently of potential environmental impacts.

KEYWORDS: Drainage pressure relief; anti-floating; numerical simulation; seepage-stress coupling; pore water pressure; diaphragm wall

1 Introduction

With the rapid advancement of urbanisation, the development and utilisation of underground space, as an essential component of three-dimensional urban development, have continued to expand [1]. In recent years, underground engineering projects have increasingly been characterised by greater burial depths and larger construction scales, resulting in more complex groundwater and hydrogeological conditions [2]. As a critical measure for ensuring the structural safety of basements, the rationality of anti-floating design directly affects the serviceability and overall safety of underground structures. Inadequate anti-floating design may lead to structural damage or even serious engineering accidents. Therefore, basement anti-floating issues have attracted considerable attention from both researchers and practicing engineers.

At present, basement anti-floating methods can generally be classified into passive anti-floating measures [3] and active anti-floating measures [4]. In engineering practice, passive anti-floating measures are still widely adopted. These methods mainly rely on structural self-weight [5], uplift piles [6], anti-floating anchors [7], and additional structural mass [8] to resist groundwater-induced buoyancy. Although passive anti-floating systems have been extensively applied, their limitations are also evident, including long construction periods, high costs, and limited adaptability to fluctuating groundwater conditions [9]. Moreover, when groundwater control measures are inadequate, insufficient anti-floating capacity may lead to basement uplift or even structural failure. Representative cases include the partial or complete failure of anti-floating measures in high-rise residential basements in Changsha [10], uplift-induced damage to basement structures [11], and floating failure caused by flooding in Turkey [12].

Compared with passive anti-floating measures, active anti-floating measures reduce buoyancy by actively regulating groundwater pressure. In such systems, drainage holes are strategically arranged at critical locations, such as basement side walls or floor slabs, to discharge groundwater, reduce local water pressure, and thereby decrease the buoyant forces acting on the structure [13]. The drainage pressure-relief method offers several advantages, including relatively simple construction, a short implementation period, and high cost-effectiveness, giving it considerable potential for application in underground engineering. For example, Luo et al. [14] developed a computational model for active anti-floating systems incorporating pressure-relief wells and analysed the relationship between well-layout parameters and anti-floating performance, thereby providing a theoretical basis for engineering design. Chen et al. [15] conducted numerical investigations under heavy rainfall conditions and demonstrated that pressure-relief drainage through drainage holes can effectively suppress short-term rises in groundwater level, thereby reducing the buoyant forces acting on basement floor slabs. Zhang [16] applied the drainage pressure-relief method to the anti-floating design of a two-storey underground car park. Compared with the original anti-floating anchor scheme, this method reduced the number of pull-out anchors by approximately 9000, lowered the project cost by about 11.94 million Chinese yuan (CNY), and shortened the construction period by nearly two months. Cheng and Zhang [17] further demonstrated the economic feasibility and construction efficiency of drainage pressure-relief technology in the fully underground Lingshanwei vehicle depot project of Qingdao Metro Line 13. Through comparisons of design schemes, applicability assessments, blind-drain layout design, inflow calculations, and groundwater level monitoring, the study confirmed that this method can satisfy anti-floating safety requirements while improving both economic and construction efficiency.

However, existing studies on active anti-floating measures have mainly focused on drainage spacing, backfill compaction, or idealised operating conditions [18]. In practical engineering, groundwater levels often exhibit pronounced time-dependent fluctuations due to rainfall recharge, seasonal variations, and surrounding hydrological conditions [19]. Under such conditions, the pressure-limiting head of drainage holes, drainage duration, and embedment depth of retaining structures jointly influence the evolution and stability of pore water pressure at the foundation base [20,21]. Nevertheless, the coupled effects of these factors on basal pore water pressure, drainage behaviour, and surrounding groundwater disturbance have not yet been sufficiently clarified. Consequently, the anti-floating mechanism and environmental response of drainage pressure-relief systems under fluctuating groundwater conditions remain inadequately understood. Recent advances in unsaturated soil mechanics have further highlighted the strong dependence of pore water pressure and mechanical responses on transient hydraulic conditions. Cui et al. [22] investigated time-step constraints in coupled hydro-mechanical finite-element analysis of unsaturated soils and demonstrated that transient responses are strongly influenced by the nonlinear hydraulic characteristics of unsaturated soils. Finnegan et al. [23], based on long-term field monitoring and numerical modelling, showed that

unsaturated groundwater flow can govern the timing and magnitude of seasonal pore-water-pressure responses and associated ground deformation. Li et al. [24] investigated rainfall-induced infiltration by considering the air-resistance delay effect and lateral seepage in the saturated zone, demonstrating that transient infiltration and seepage processes can substantially influence pore-water-pressure development and slope stability. Eyo and Abbey [25] further demonstrated that rainfall-induced changes in hydraulic conditions can significantly affect pore-water-pressure evolution in unsaturated soils. Recent developments in coupled hydro-mechanical numerical modelling have also enabled the interaction between suction-dependent hydraulic behaviour and soil deformation to be represented more explicitly [26]. Ng et al. [27] developed a physics-informed surrogate model for hydro-mechanical coupling in unsaturated soils, in which the mass balance of water and force equilibrium were incorporated as physical constraints to predict pore water pressure, effective stress, and soil deformation. Their study demonstrated the importance of explicitly considering the interaction between seepage and mechanical deformation when evaluating unsaturated soil behaviour under changing hydraulic conditions. Furthermore, Guo et al. [28] demonstrated that different rainfall patterns can produce pronounced spatial and temporal variations in pore water pressure. These findings further indicate that transient hydraulic processes and hydro-mechanical interactions should be considered when evaluating the performance of drainage pressure-relief systems subjected to fluctuating groundwater conditions.

To address these knowledge gaps, this study develops a seepage–stress coupled finite element model based on an actual engineering case to investigate the drainage pressure-relief method under multiple hydraulic and structural conditions. The coupled effects of groundwater-level fluctuations, the pressure-limiting head of drainage holes, and diaphragm-wall embedment depth on basal pore-water pressure, seepage characteristics, drainage behaviour, and surrounding ground response are systematically investigated. The study aims to clarify the pressure-relief mechanism and transient response of drainage-based anti-floating systems under variable groundwater conditions, and to provide a theoretical basis and technical reference for their design optimisation and environmental risk control.

2 Finite Element Modelling

2.1 Engineering Geological Conditions

A large underground parking facility beneath a commercial plaza in Shenzhen comprises three basement levels. The site is located on a lacustrine–alluvial plain characterised by relatively flat terrain. The alluvial soils are relatively fine-grained and generally well consolidated, whereas the underlying strata exhibit low permeability. In addition, the relatively sparse distribution of surrounding buildings is favourable for the implementation of the drainage pressure-relief method. According to the engineering geological investigation report, the strata at the site, from top to bottom, consist of artificial fill, silty clay, and interbedded silty clay and silt. The underlying bedrock consists of relatively impermeable granite. The principal design parameters of each soil layer are presented in Table 1.

Table 1: Soil parameters used in the numerical model.

Layer	Soil Type	Thickness (m)	Unit Weight (kN/m ³)	Poisson Ratio	Cohesion (kPa)	Internal Friction Angle (°)	Young's Modulus (MPa)	Permeability (m/d)
1	Miscellaneous fill	10	17.7	0.33	6.8	13.9	3.86	0.01
2	Silty clay	7.2	19.6	0.32	30.7	13.5	8.64	0.005

Table 1: *Cont.*

Layer	Soil Type	Thickness (m)	Unit Weight (kN/m ³)	Poisson Ratio	Cohesion (kPa)	Internal Friction Angle (°)	Young's Modulus (MPa)	Permeability (m/d)
3	Silty clay	9.5	19.1	0.3	7.9	29.8	10.57	0.05
4	Silty clay interbedded with silt	13.7	19.3	0.3	3.6	31.7	9.21	0.03
5	Silty clay interbedded with silt	9.6	19.5	0.3	14.1	24.4	14.58	0.02
6	Sand	1	20.1	0.2	4.5	30.8	202.85	1.50

2.2 Hydrogeological Conditions

Within the site, groundwater primarily comprises perched water and phreatic water, with no confined aquifers identified. The perched water is mainly distributed within the soil layers at depths of less than approximately 10 m and is strongly influenced by rainfall infiltration. Groundwater is widely distributed throughout the site, and the design groundwater level for anti-floating is taken as the ground surface elevation.

2.3 Seepage Control Equation

Groundwater flow through the soil is governed by Darcy's law. The specific discharge can be expressed as:

$$q = ki = k \frac{H_1 - H_2}{L} \quad (1)$$

where, q denotes the Darcy velocity or specific discharge; H_1 and H_2 are the hydraulic heads at the upstream and downstream ends of the soil specimen, respectively; L denotes the seepage path length; k represents hydraulic conductivity; i represents hydraulic gradient.

For transient saturated–unsaturated seepage, the Richards equation was adopted:

$$C(h) \frac{\partial h}{\partial t} = \nabla \cdot [k(h) \nabla (h + z)] + Q \quad (2)$$

where, h is the pressure head; z is the elevation head; $k(h)$ is the pressure-head-dependent hydraulic conductivity; Q is the volumetric source or sink term; $C(h)$ is the specific moisture capacity, defined as:

$$C(h) = \frac{\partial \theta}{\partial h} \quad (3)$$

where θ is the volumetric water content.

The total hydraulic head is defined as:

$$H = h + z \quad (4)$$

and the pressure head is related to the pore water pressure by:

$$h = \frac{u_w}{\gamma_w} \quad (5)$$

where u_w is pore water pressure and γ_w is the unit weight of water.

For a two-dimensional transient seepage, Eq. (2) can be written as:

$$\frac{\partial}{\partial x}(k_x(h)\frac{\partial H}{\partial x}) + \frac{\partial}{\partial y}(k_y(h)\frac{\partial H}{\partial y}) + Q = C(h)\frac{\partial h}{\partial t} \quad (6)$$

where, k_x and k_y are the hydraulic conductivity in x and y direction, t is time.

Under quasi-static conditions, the mechanical equilibrium of the soil is governed by:

$$\nabla \cdot \sigma + b = 0 \quad (7)$$

where σ is the total stress tensor and b is the body-force vector, including gravitational loading. For a two-dimensional problem, Eq. (7) becomes:

$$\begin{aligned} \frac{\partial \sigma_{xx}}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + b_x &= 0 \\ \frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_{yy}}{\partial y} + b_y &= 0 \end{aligned} \quad (8)$$

where σ_{xx} and σ_{yy} are the normal stress components in the x and y directions, respectively; τ_{xy} is the shear-stress component; b_x and b_y are the components of the body-force vector.

The mechanical response of the soil skeleton is governed by the effective stress. For saturated soil, the effective stress tensor is expressed as:

$$\sigma' = \sigma - \alpha u_w I \quad (9)$$

where σ' is the effective stress tensor; α is Biot's coefficient; and I is the second-order identity tensor. For fully saturated soil, α is commonly approximated as unity.

For the saturated and nearly saturated zones governing the basement response, the effective stress tensor is expressed as:

$$d\sigma' = D^{ep} d\varepsilon \quad (10)$$

where D^{ep} is the elastoplastic tangent constitutive matrix and $d\varepsilon$ is the incremental strain vector.

The shear strength is governed by the Mohr–Coulomb criterion:

$$\tau_f = c' + \sigma'_n \tan \phi' \quad (11)$$

where τ_f is the shear strength, c' is the effective cohesion, σ'_n is the effective normal stress, and ϕ' is the effective internal friction angle.

In the coupled analysis, changes in effective stress induce deformation of the soil skeleton, thereby modifying its pore structure. The volumetric strain is defined as:

$$\varepsilon_v = \frac{\Delta V}{V_0} \quad (12)$$

where V_0 is the initial total volume and ΔV is the change in total volume. In the present formulation, volumetric expansion is defined as positive.

Assuming incompressible soil particles, the current porosity can be expressed as

$$n = 1 - \frac{1 - n_0}{1 + \varepsilon_v} \quad (13)$$

where n_0 and n are the initial and current porosities, respectively. The corresponding stress-dependent hydraulic conductivity is expressed as:

$$k(\sigma) = k_0 \left[\frac{n(1 - n_0)}{n_0(1 - n)} \right]^3 \quad (14)$$

where k_0 is the initial hydraulic conductivity and $k(\sigma)$ is the hydraulic conductivity associated with the current stress state.

2.4 Finite Element Modelling Method

The numerical simulations were conducted using GeoStudio, with SEEP/W employed for transient saturated–unsaturated seepage analysis and SIGMA/W for stress–deformation analysis. These modules are based on the finite-element method and have been widely applied to coupled groundwater-flow and deformation problems in saturated and unsaturated soils [29]. A two-dimensional computational domain with a width of 90 m and a height of 50 m was established, with the stratigraphic boundaries defined according to the geological investigation data. A basement structure measuring 30 m in width and 20 m in height was incorporated into the model. Considering geometric symmetry, a half-model was adopted along the symmetry axis. The diaphragm wall was modelled with a thickness of 1 m and an embedment depth of 25 m.

In SEEP/W, transient groundwater seepage was calculated based on Darcy's law and the Richards equation. The saturated hydraulic conductivity of each soil layer was assigned according to the engineering hydrogeological investigation data. For the unsaturated zone, the hydraulic properties were described using the soil-water characteristic curves (SWCCs) and unsaturated hydraulic conductivity functions. The variation in hydraulic conductivity and volumetric water content with matric suction was therefore considered to represent the transition between unsaturated and saturated states, as shown in Figs. 1 and 2.

For the mechanical analysis in SIGMA/W, all soil layers, including the sand and clay layers, were simulated using the Mohr–Coulomb elastoplastic constitutive model. The corresponding mechanical parameters, including elastic modulus, Poisson's ratio, cohesion, friction angle, and unit weight, were assigned according to the engineering properties of each soil layer. An initial geostatic stress state was established considering soil self-weight and the initial pore-water-pressure distribution before the subsequent hydro-mechanical analysis.

Based on the geological and hydrogeological data, the initial groundwater level was assumed to be located 35 m below the ground surface. Accordingly, the soil below the groundwater table was treated as saturated, while the soil above the groundwater table was modelled as an unsaturated seepage zone. The transient analysis adopted a time step of 1 day according to the temporal resolution of the groundwater monitoring data and the relatively slow seepage response of the low-permeability soil layers.

The computational domain was discretised using a structured quadrilateral finite-element mesh with a global element size of approximately 1 m, resulting in 3984 elements and 4124 nodes. The hydraulic and mechanical boundary conditions, together with the implementation of the drainage openings, are described separately in Section 2.5.

2.5 Boundary Condition

The finite-element model, geometric configuration, and principal boundary conditions are illustrated in Fig. 3. Because the groundwater recharge sources are located far from the site, the following boundary conditions were adopted for the seepage–stress coupled analysis. The right boundary was specified as a hydraulic head boundary and was fully restrained against displacement in both directions. Along the symmetry axis (left boundary), displacement in the x direction was constrained. The diaphragm wall and the bottom boundary of the model were defined as impermeable boundaries.

Three drainage openings were arranged beneath the basement slab at distances of 10 m, 20 m, and 30 m from the right boundary, respectively. Each drainage opening was represented by a 1 m-long boundary segment in the two-dimensional finite-element model. The drainage openings were treated as hydraulic control boundaries rather than explicitly modelled as drainage pipes. A prescribed pressure-limiting total head was assigned at each drainage location according to the working condition to represent the pressure-control function of the drainage system.

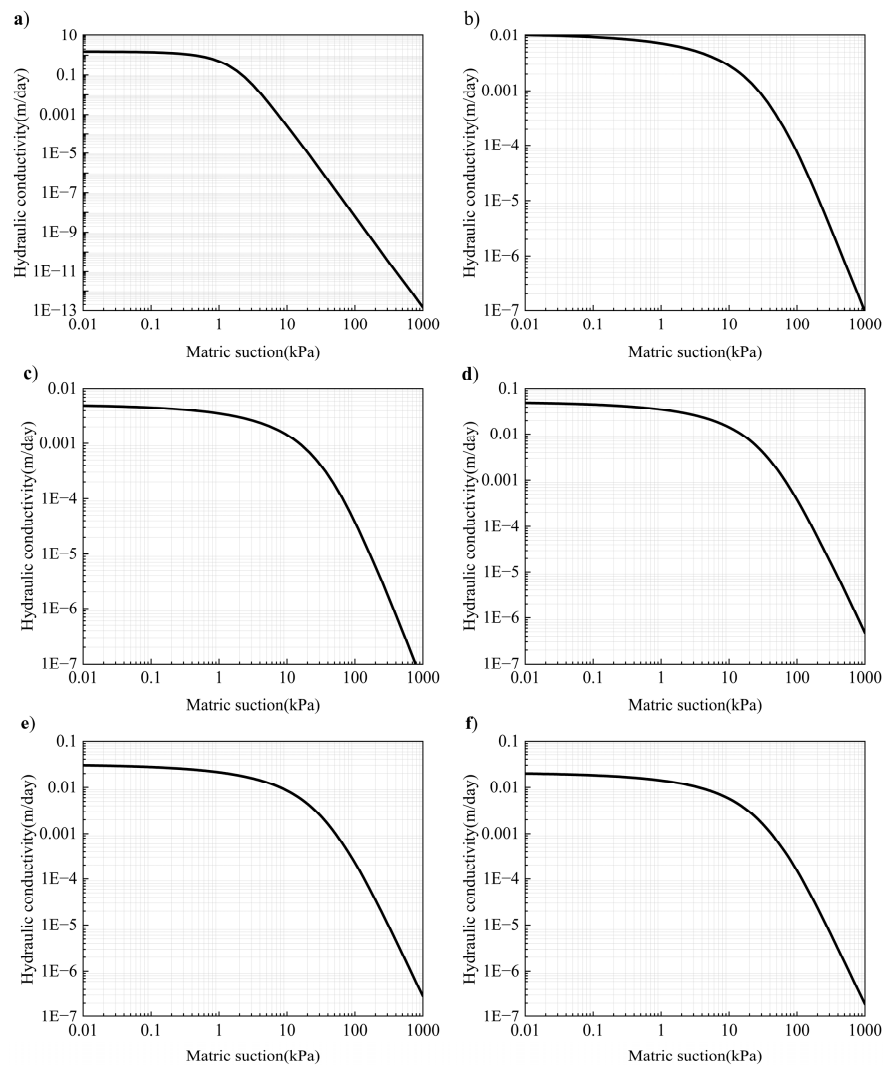


Figure 1: Soil permeability curve: (a) Sand; (b) Miscellaneous fill; (c) Clayey silt 1; (d) Clayey silt 2; (e) Silty clay and silty sand interlayer 1; (f) Silty clay and silty sand interlayer 2.

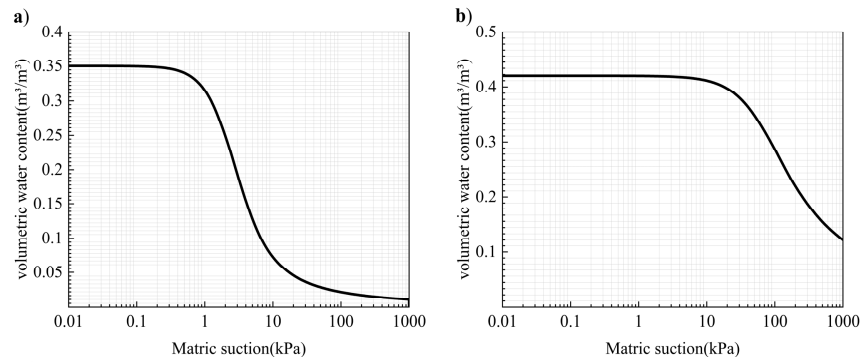


Figure 2: Cont.

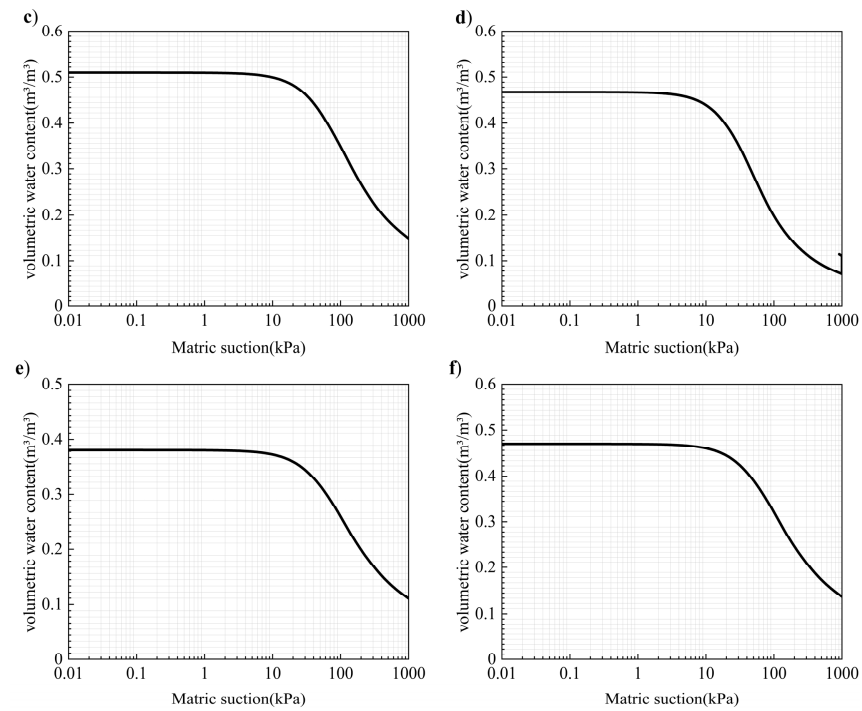


Figure 2: Soil-water characteristic curve: (a) Sand; (b) Miscellaneous fill; (c) Clayey silt 1; (d) Clayey silt 2; (e) Silty clay and silty sand interlayer 1; (f) Silty clay and silty sand interlayer 2.

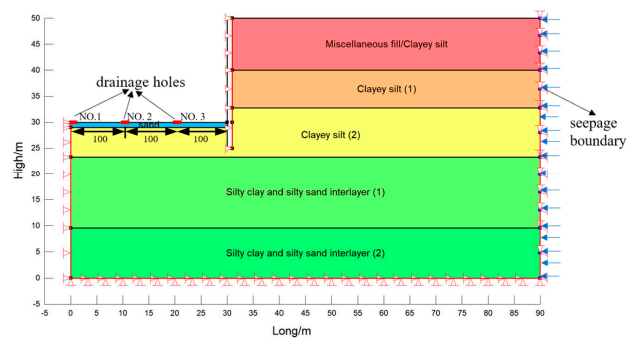


Figure 3: Finite-element model.

2.6 Test Schemes

Four operational scenarios were designed to evaluate the effectiveness of the drainage pressure-relief method in controlling buoyancy and its associated environmental effects. These scenarios considered variable groundwater level boundaries, different pressure-limiting heads of drainage holes, and varying diaphragm wall embedment depths, as summarised in Table 2.

The numerical programme followed a progressive design framework. First, the feasibility of drainage pressure relief under fluctuating groundwater conditions was evaluated. The influence of pressure-limiting head settings on pressure-relief efficiency and environmental disturbance was then quantified. Finally, the effect of diaphragm wall embedment depth on drainage demand and groundwater drawdown was assessed under an adverse condition of a fixed high groundwater level.

A pressure-limiting total head of 43 m was selected as a relatively conservative active pressure-relief condition to evaluate the reduction in buoyancy achieved by moderately lowering the basal pressure head while maintaining anti-floating safety. At a foundation-base elevation of approximately 30 m, a total head of 43 m corresponds to a pressure head of approximately 13 m. Compared with the uncontrolled condition corresponding to the 50 m design groundwater elevation, this represents a theoretical reduction of approximately 35% in the groundwater pressure acting on the basement slab.

A pressure-limiting total head of 35 m was selected as an enhanced pressure-relief condition to evaluate the maximum potential of active drainage under a lower control head. At the foundation base, this total head corresponds to a pressure head of approximately 5 m. Compared with the 50 m anti-floating design groundwater elevation, the corresponding groundwater pressure is theoretically reduced by approximately 75%. This condition therefore provides a reference for assessing the potential of active drainage to reduce or optimise the demand for conventional passive anti-floating measures, such as uplift piles and anti-floating anchors.

In this study, the pressure-limiting total head of the drainage holes refers to the prescribed maximum total hydraulic head maintained at the drainage-hole boundaries. Once the groundwater-induced total hydraulic head tends to exceed this prescribed value, drainage is activated to prevent a further increase in pore water pressure. The corresponding pressure head depends on the elevation of the drainage hole and is calculated as the difference between the total hydraulic head and the elevation head. For example, when the pressure-limiting total head is set to 43 m and the drainage-hole elevation is approximately 30 m, the corresponding pressure head is approximately 13 m.

Table 2: Test schemes.

Working Condition	Groundwater Level Condition	Pressure-Limiting Total Heads at the Drainage Holes (m)	Buried Depth of Diaphragm Wall (m)
1	Variable water level	43	25
2	Variable water level	35	25
3	Fixed groundwater level (50 m head)	35	25
4	Fixed groundwater level (50 m head)	35	30

3 Result and Discussion

3.1 Working Condition 1

Working Condition 1 was designed as an economical drainage-control strategy under variable groundwater conditions. The pressure-limiting total head of the drainage holes was set at a relatively high level, allowing the drainage system to operate only during short-duration high-groundwater level events. This

scenario was used to assess whether intermittent drainage could effectively control basal pore water pressure while avoiding unnecessary long-term discharge. A total simulation period of 1050 d was adopted to represent a complete groundwater fluctuation cycle.

The pressure-limiting total head of the drainage holes was set to 43 m, corresponding to a pressure head of 13 m at the foundation base. Under this condition, the uplift force associated with the 13 m pressure head was jointly resisted by the anti-uplift piles and the self-weight of the structure. Compared with the anti-floating design groundwater level defined at the ground surface elevation, the controlled total head was reduced by 7 m, which may substantially reduce the cost of anti-floating measures for large underground structures.

Fig. 4 shows that the groundwater level exceeded an elevation of 43 m for only approximately 10% of the simulation period, indicating that high-groundwater level events were short-lived and infrequent. Therefore, relying exclusively on costly passive anti-floating measures to resist such transient events may be economically inefficient. The following analyses further evaluate the feasibility of drainage pressure relief, with particular emphasis on its ability to control basal pore water pressure under peak groundwater conditions and its influence on the surrounding groundwater environment.

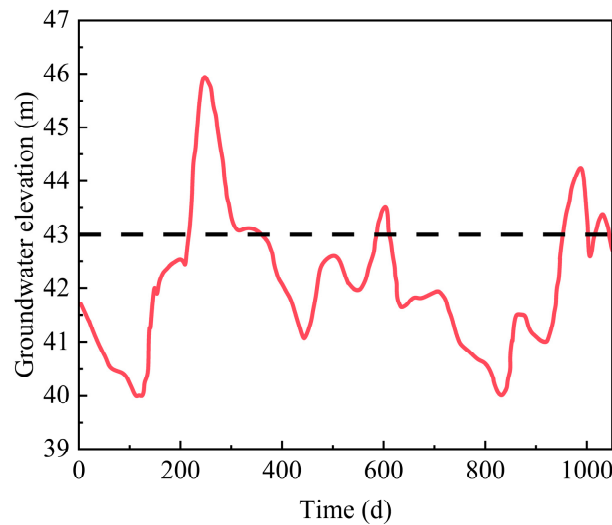


Figure 4: Groundwater level fluctuation curve.

3.1.1 Distribution of Pore Water Pressure

As illustrated by the seepage fields corresponding to the maximum and minimum groundwater levels in Fig. 5 and the basal total-head distribution in Fig. 6, the pore water pressure at the foundation base is maintained within the prescribed range when the pressure-limiting total head of the drainage holes is set to 43 m. When the groundwater elevation falls below the preset pressure-limiting total head, reverse seepage may occur through the drainage-hole boundaries, resulting in groundwater recharge to the surrounding soil. However, because the groundwater elevation remains below the anti-floating design groundwater level, structural safety is still ensured. In contrast, under high-groundwater level conditions, when the pressure head at the foundation base tends to exceed 13 m, the drainage system is activated to discharge groundwater, thereby limiting the basal pore water pressure to the prescribed level.

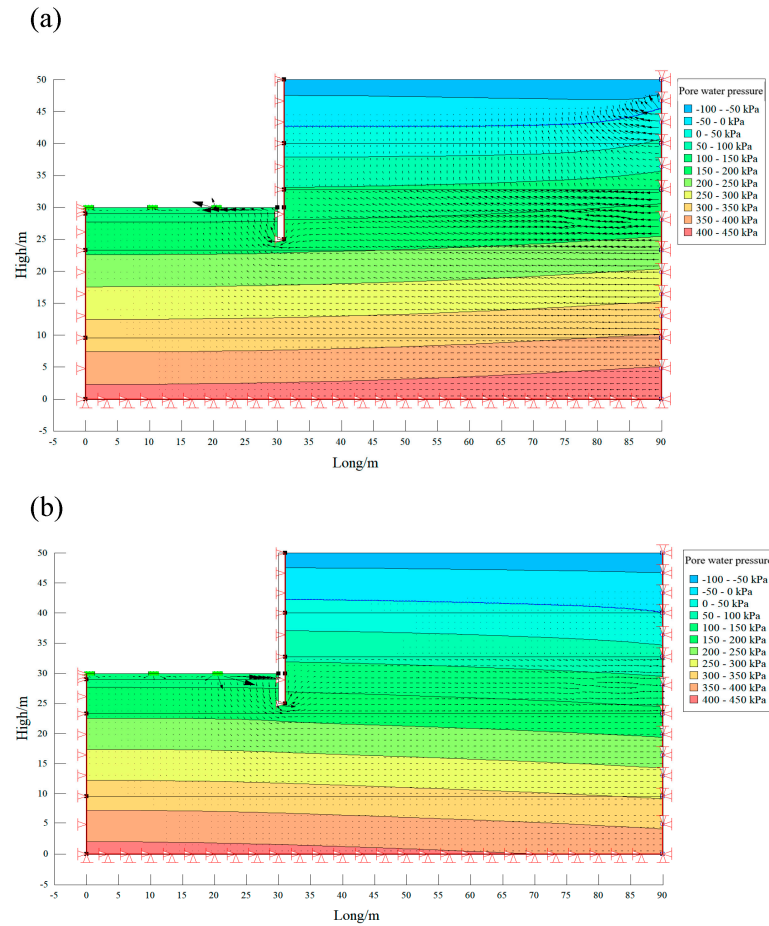


Figure 5: Pore water pressure distribution and seepage velocity vectors at (a) the maximum groundwater level and (b) the minimum groundwater level.

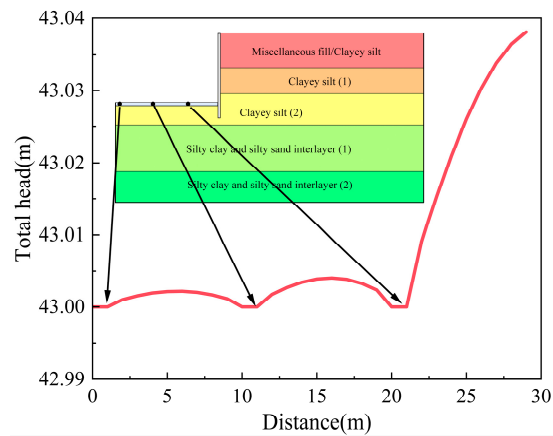


Figure 6: Distribution of total water head at the base of the foundation.

3.1.2 Seepage Flow

Under Working Condition 1, Fig. 7a shows the temporal variations in the groundwater levels at the foundation base and on the right side of the diaphragm wall. Because a relatively high pressure-limiting

total head was assigned to the drainage holes, drainage occurred only when the groundwater elevation exceeded this threshold. The resulting short operating duration of the drainage system is therefore an important contributor to its cost-effectiveness.

As shown in Fig. 7b, pore water pressure fluctuations at monitoring points located 10 m, 20 m, and 40 m from the diaphragm wall become progressively more pronounced with increasing distance from the drainage holes. This trend demonstrates that the drainage holes are effective in stabilising pore water pressure beneath the foundation base.

Fig. 8 shows that, over the 1050 d simulation period, the three drainage holes discharged only approximately 1200 m³ of water because drainage occurred intermittently. As shown in Fig. 9, the drainage-velocity distribution indicates that discharge is concentrated primarily at the drainage hole closest to the ground–diaphragm wall junction, with the discharge rate increasing as the distance to this junction decreases. The higher discharge near the structural boundary can be attributed to the shorter seepage path and the resulting larger hydraulic gradient, which together promote groundwater flow towards the drainage holes. This behaviour is consistent with the findings of Ekmekci et al. (2017) [18], who demonstrated through numerical simulations that relief drains installed at appropriate locations beneath concrete slabs could effectively reduce uplift pressure in dam engineering.

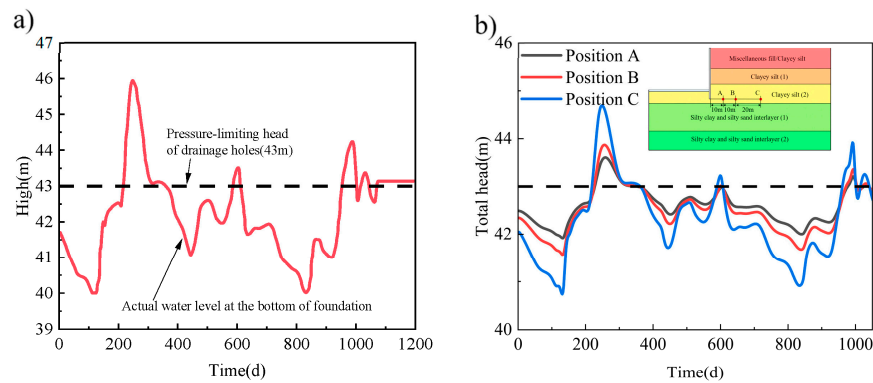


Figure 7: (a) Groundwater level fluctuation curve (b) pore water pressure at the points located 10 m, 20 m, and 40 m from the diaphragm wall.

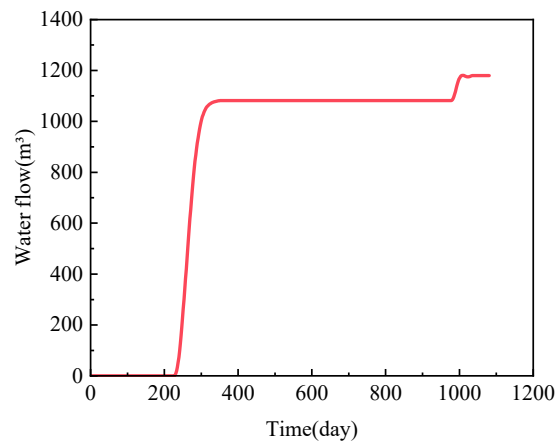


Figure 8: Cumulative drainage volume through the three drainage holes.

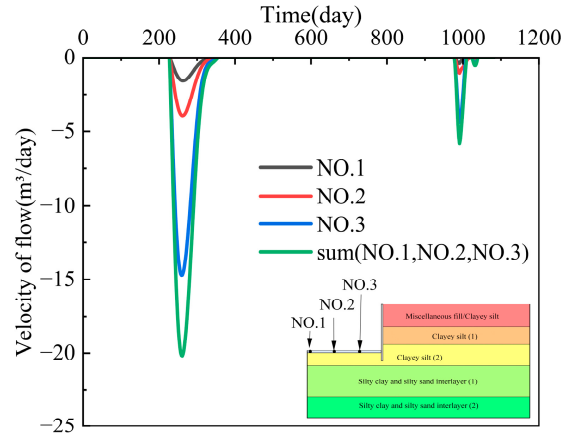


Figure 9: Distribution of drainage velocity around the drainage holes.

3.1.3 Environmental Influences

Fig. 10 shows the vertical soil displacement under the maximum groundwater level condition. The maximum settlement is approximately 0.016 m and occurs at a location about 40 m from the edge of the building. Although the groundwater level within the soil mass fluctuates periodically and additional surface loads may be present, the ground-surface settlement does not exhibit a distinct periodic response. Instead, the overall settlement gradually tends towards a stable state over time, although a slow rebound of the soil may occur.

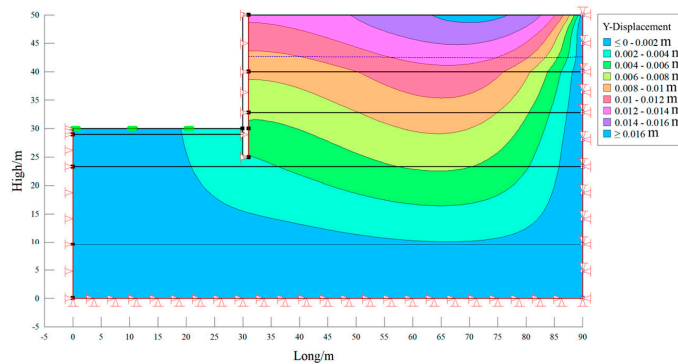


Figure 10: Vertical displacement contour under Working Condition 1.

3.2 Working Condition 2

To examine the effect of the pressure-limiting total-head setting, Working Condition 2 adopted a lower pressure-limiting total head under the same variable groundwater level condition. Compared with Working Condition 1, this case represents a more conservative pressure-control strategy, with the drainage system remaining active for a longer period. The comparison highlights the trade-off between anti-floating safety and environmental disturbance.

The pressure-limiting total head of the drainage holes was reduced to 35 m, corresponding to a pressure head of 5 m at the foundation base. This controlled total head is 15 m lower than the anti-floating design groundwater elevation at the ground surface. Because the prescribed pressure-limiting total head remained below the natural groundwater elevation throughout the simulation, drainage occurred continuously.

3.2.1 Distribution Law of Pore Water Pressure

Fig. 11 shows that the drainage pressure-relief method effectively reduces basal pore water pressure. The total hydraulic head at each monitoring point remains nearly stable at the prescribed pressure-limiting total head of 35 m, as indicated by the nearly horizontal time-history curves. This confirms the feasibility and safety of using the prescribed pressure-limiting total head as the control criterion for anti-floating under drainage conditions.

Near the diaphragm wall, the phreatic surface is approximately 2.5 m higher than the prescribed pressure-limiting total head, resulting in a local hydraulic head difference.

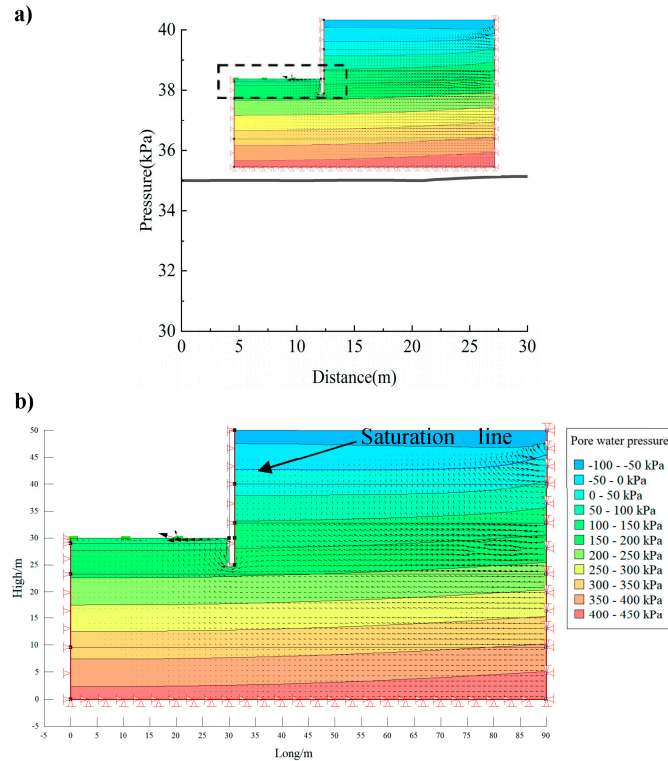


Figure 11: (a) Pore water pressure distribution at the model base; (b) Pore water pressure distribution and seepage velocity vectors.

Fig. 12 presents the groundwater-head responses at horizontal distances of 10 m, 20 m, and 40 m from the right side of the diaphragm-wall base, together with the far-field groundwater level. The monitoring points generally fluctuate in phase with the far-field groundwater level, indicating that groundwater-head variations around the diaphragm wall are primarily governed by external groundwater fluctuations. Nevertheless, continuous drainage through the base drainage holes substantially attenuates the response of the nearby monitoring points to far-field fluctuations, resulting in markedly smaller fluctuation amplitudes. This attenuation becomes increasingly pronounced with decreasing distance to the diaphragm wall and drainage holes.

These findings indicate that the base drainage holes can effectively reduce and stabilise the total hydraulic head beneath the foundation base while limiting the transmission of far-field groundwater fluctuations to the groundwater pressure around the structure, thereby enhancing the safety and reliability of drainage-based anti-floating measures.

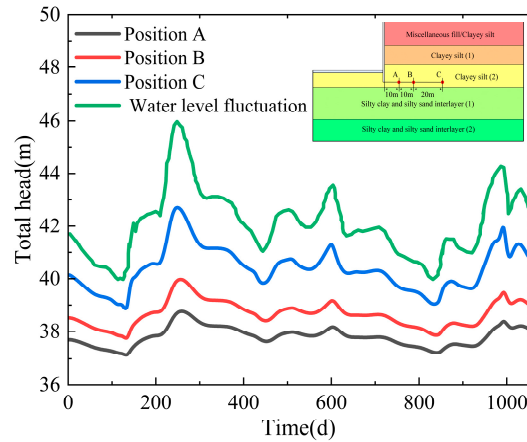


Figure 12: Temporal variations in total head at monitoring points A, B, and C and groundwater level.

3.2.2 Seepage Flow

Fig. 13 shows the temporal variation in the drainage discharge through the drainage holes. The discharge generally follows the fluctuations in groundwater level, with the maximum total discharge reaching $130 \text{ m}^3/\text{day}$. The impermeable boundary formed by the diaphragm wall alters the seepage path and concentrates groundwater flow in the vicinity of the wall.

Drainage performance varies markedly among the drainage holes. The discharge rate increases as the drainage hole approaches the diaphragm wall, consistent with the trend observed under the 43 m pressure-limiting total-head condition. These findings suggest that drainage capacity should be enhanced near impermeable boundaries in drainage-based anti-floating systems. This can be achieved by increasing the diameter of drainage holes in these regions when the number of holes is fixed, or by increasing the number of holes near the diaphragm wall when the hole diameter is constrained.

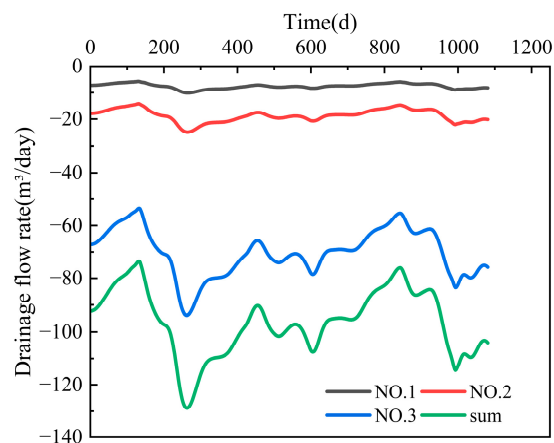


Figure 13: Temporal variations in drainage flow rates through the three drainage holes.

3.2.3 Environmental Influences

For drainage-based anti-floating systems, ground settlement represents a key environmental constraint in addition to pore pressure reduction and seepage discharge. As shown in Fig. 14, under a prescribed pressure-limiting total head of 35 m, the maximum vertical settlement reaches 0.024 m and is mainly concentrated within approximately 40 m of the underground structure.

Reducing the pressure-limiting total head of the drainage holes increases the hydraulic gradient and enhances groundwater discharge. However, owing to the highly permeable sand cushion beneath the basement slab, groundwater pressure is rapidly redistributed within the drainage layer. Consequently, lowering the pressure-limiting total head primarily results in a relatively uniform reduction in pressure head beneath the basement slab, rather than the formation of a localised depression cone around individual drainage holes.

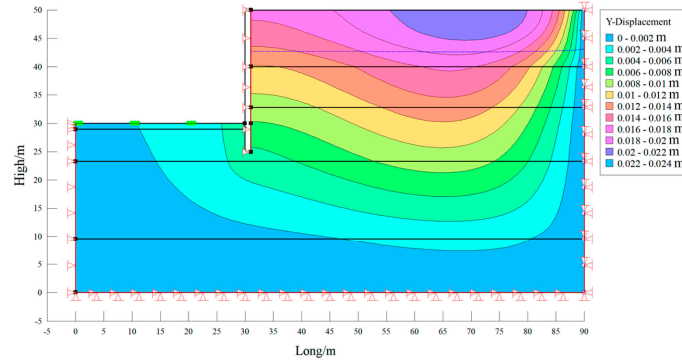


Figure 14: Vertical displacement contour under Working Condition 2.

3.3 Working Condition 3

Following the assessment of drainage-head effects under variable groundwater levels, Working Condition 3 adopted a fixed high groundwater elevation of 50 m to represent an adverse long-term hydraulic condition. This scenario was used to evaluate the seepage response, drainage demand, and settlement risk under sustained high-groundwater level conditions.

Because seepage pathways strongly influence groundwater flow through clay strata, this scenario examines the performance of drainage pressure relief under a fixed groundwater elevation of 50 m, corresponding to the anti-floating design groundwater level. The pressure-limiting total head of the drainage holes was set to 35 m, corresponding to a pressure head of 5 m at the foundation base, and the diaphragm wall extended 5 m below the excavation base. All other model parameters were identical to those adopted in Working Condition 1.

3.3.1 Distribution of Pore Water Pressure

As shown in Fig. 15a, under constant groundwater level and pressure-limiting total-head conditions, the pore water pressure at the foundation base remains stable. When the diaphragm wall extends 5 m below the foundation base, the drainage pressure-relief system maintains the basal pore water pressure at the level corresponding to the prescribed pressure-limiting total head.

This pressure-control effect, however, causes a noticeable disturbance to the surrounding groundwater field. The maximum groundwater drawdown reaches approximately 10 m near the structure, while a total-head difference of about 4 m develops across the diaphragm wall. This indicates that impermeable barriers such as diaphragm walls can effectively impede seepage and reshape the local hydraulic field.

Fig. 15b shows the pore water pressure histories at monitoring points located 10 m, 20 m, and 40 m from the diaphragm wall, while Fig. 15c shows the locations of these monitoring points around the structure. After approximately 100 d, the seepage field approaches a quasi-steady state, and the final pore water pressure distribution exhibits a typical drawdown-cone profile.

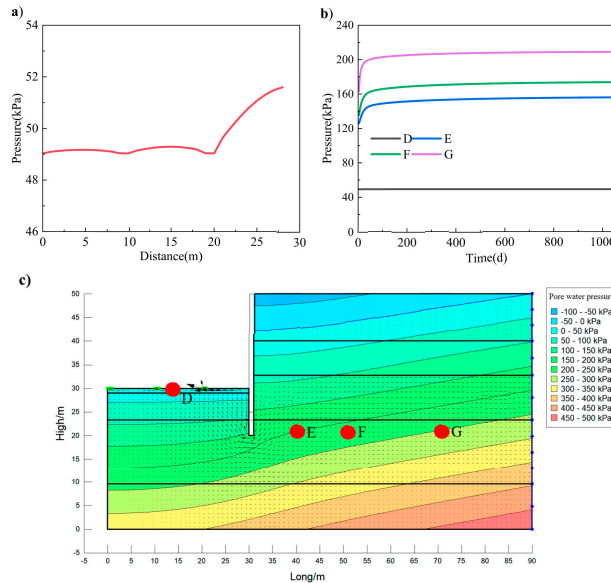


Figure 15: Pore pressure profile: (a) Bottom pore pressure of foundation; (b) The change of pore pressure with time at D, E, F, G; (c) Model measuring point distribution map.

3.3.2 Seepage Flow

Fig. 16 shows the discharge-rate distribution at the drainage holes. The discharge from each outlet increases during the initial drainage stage and gradually stabilises after approximately 100 d, consistent with the evolution of the groundwater drawdown. Drainage rates are higher for outlets located closer to the ground–diaphragm wall interface, and the maximum combined discharge from the three outlets reaches approximately 200 m³/day.

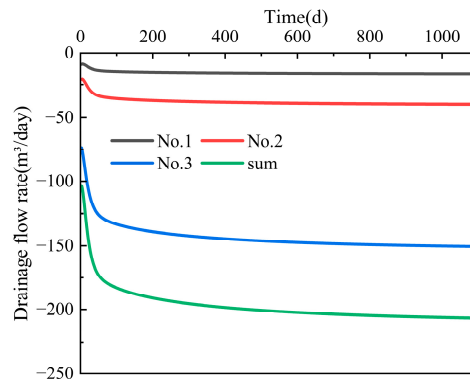


Figure 16: Temporal variations in drainage flow rates through the three drainage holes under Working Condition 3.

3.3.3 Environmental Influences

As shown in Fig. 17, the vertical settlement around the building perimeter initially increases with distance from the structure and then decreases, with a maximum settlement of approximately 0.16 m occurring about 40 m from the building edge. Under the same basal pore water-pressure condition, Working Condition 3 results in substantially greater settlement in the surrounding ground than Working Condition 2. This difference is primarily attributed to the larger groundwater drawdown, which increases the effective stress in the surrounding soil.

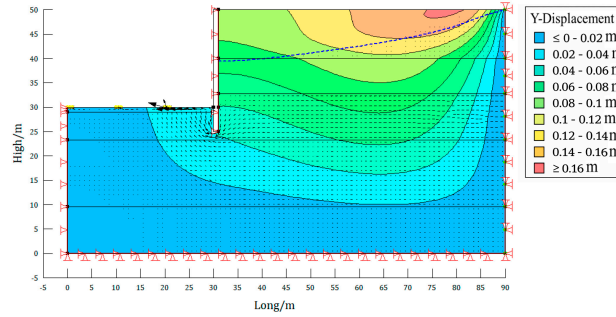


Figure 17: Vertical displacement contour under Working Condition 3.

3.4 Working Condition 4

Working Condition 4 increases the embedment depth of the diaphragm wall while maintaining the same fixed high groundwater elevation and pressure-limiting total head as those adopted in Working Condition 3. This scenario was designed to evaluate whether extending the seepage path can reduce drainage demand and mitigate disturbance to the surrounding groundwater environment.

The diaphragm wall was embedded to a depth of 30 m, extending 10 m below the foundation base. The pressure-limiting total head of the drainage holes was set to 35 m, corresponding to a pressure head of 5 m at the foundation base, while the groundwater elevation was fixed at 50 m.

3.4.1 Distribution of Pore Water Pressure

As shown in Fig. 18, the basal pore water pressure in Working Condition 4 remains nearly constant at the prescribed value of 50 kPa, with no significant variation over time. Consistent with the results obtained under Working Conditions 1–3, this finding confirms that the drainage pressure-relief method can effectively stabilise pore water pressure beneath the foundation base under complex hydraulic conditions and thereby provide reliable anti-floating control.

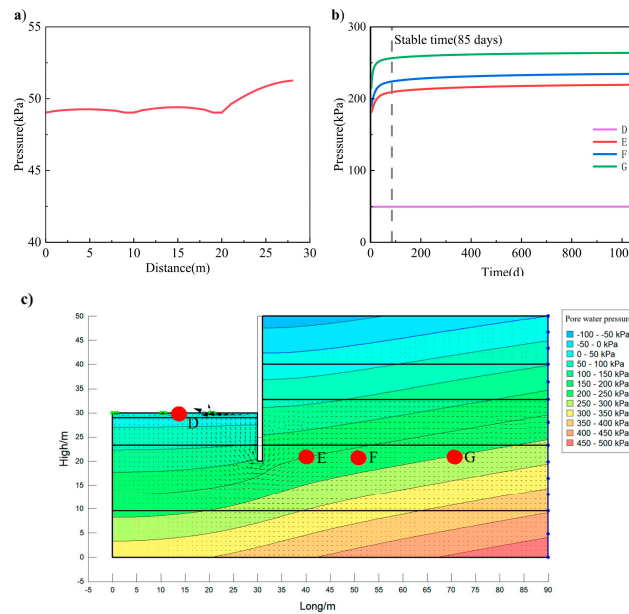


Figure 18: Pore pressure profile: (a) Bottom pore pressure of foundation; (b) The change of pore pressure with time at D, E, F, G; (c) Model measuring point distribution map.

The groundwater drawdown is approximately 7.5 m, which is markedly lower than the 10 m observed in Working Condition 3. This indicates that increasing the embedment depth of the diaphragm wall can reduce groundwater drawdown by modifying the seepage path and hydraulic-gradient distribution, thereby mitigating disturbance to the surrounding groundwater environment.

3.4.2 Seepage Flow

As shown in Fig. 19, the discharge rates of the three drainage holes stabilise after approximately 85 d, consistent with the stabilisation of pore water pressure at monitoring points D, E, and F. The drainage hole closest to the diaphragm wall exhibits the highest discharge rate, reaching approximately 110 m³/day, followed by Drainage Hole No. 2 at approximately 40 m³/day and Drainage Hole No. 3 at approximately 20 m³/day. This confirms that the discharge rate increases as the distance to the diaphragm wall decreases, consistent with the trends observed under the previous working conditions.

Compared with Working Condition 3, increasing the embedment depth of the diaphragm wall reduces the total drainage volume under otherwise identical conditions. This demonstrates that diaphragm-wall embedment depth plays an important role in controlling drainage demand: a deeper diaphragm wall extends the seepage path and reduces the discharge required to achieve the prescribed pressure-relief target. Therefore, increasing the embedment depth of the diaphragm wall can be an effective measure for reducing drainage demand and mitigating groundwater disturbance in drainage-based anti-floating systems.

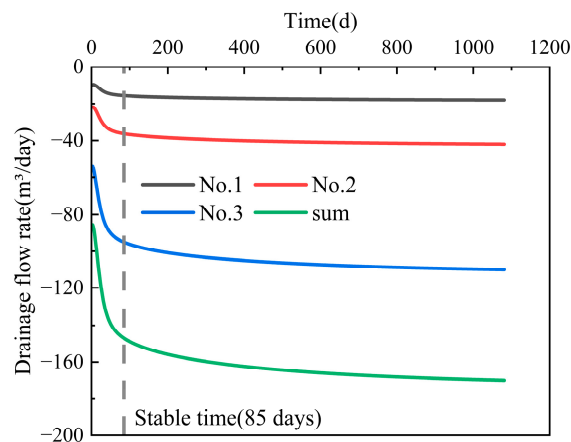


Figure 19: Temporal variations in drainage flow rates through the three drainage holes under Working Condition 4.

3.4.3 Environmental Influences

As shown in Fig. 20, the maximum vertical settlement under Working Condition 4 reaches approximately 0.22 m, and the overall settlement pattern remains generally similar to those observed under the previous working conditions. Compared with Working Condition 3, Working Condition 4 exhibits a slightly larger local maximum settlement, indicating that the influence of diaphragm-wall embedment depth on ground deformation is not monotonic. Although a greater embedment depth extends the seepage path and reduces groundwater inflow and drainage demand, it also alters the spatial distribution of pore water pressure and effective stress, which may redistribute the deformation field. Compared with Working Conditions 1 and 2, the fixed high-groundwater-level condition produces a broader settlement trough and greater overall deformation.

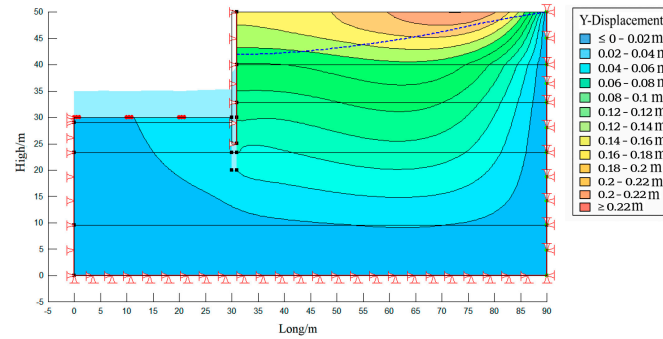


Figure 20: Vertical displacement contour under Working Condition 4.

3.5 Comparison

A comparison of Working Conditions 1 and 2 shows that lowering the pressure-limiting total head enhances basal pressure relief but changes the drainage regime from intermittent to continuous operation, thereby increasing cumulative discharge and disturbance to the surrounding groundwater field. Therefore, the pressure-limiting total head should not be selected solely to minimise basal pore water pressure; instead, its selection should balance anti-floating safety, drainage duration, and environmental impact.

Although Working Conditions 2 and 3 adopt the same pressure-limiting total head, they produce markedly different environmental responses. Under variable groundwater level conditions, the surrounding groundwater field remains primarily governed by external fluctuations, and drainage-induced settlement is relatively limited. In contrast, a fixed high groundwater level provides sustained hydraulic recharge, leading to the formation of a stable drawdown cone and substantially greater settlement. Therefore, long-term high-groundwater level conditions should be regarded as a critical consideration in environmental risk assessment.

The comparison between Working Conditions 3 and 4 indicates that increasing the embedment depth of the diaphragm wall lengthens the seepage path and reduces drainage demand. This highlights the dual function of the diaphragm wall as both a retaining structure and a hydraulic barrier. Greater embedment depth reduces the dependence of the anti-floating system on continuous drainage and mitigates groundwater drawdown outside the structure.

4 Conclusions

This study establishes a theoretical and numerical basis for drainage-based anti-floating control and evaluates its performance under variable groundwater levels, different pressure-limiting total-head settings, and varying diaphragm-wall embedment depths. The principal conclusions are as follows:

- (1) Drainage pressure relief can provide effective anti-floating control by actively regulating pore water pressure beneath the foundation base. Its performance depends strongly on site-specific hydrogeological conditions and operating parameters. The method is particularly suitable for sites characterised by pronounced groundwater level fluctuations and prolonged periods of relatively low groundwater levels.
- (2) Drainage performance exhibits a clear boundary effect. Drainage holes located near structural boundaries show higher discharge rates because of shorter seepage paths and larger hydraulic gradients. Therefore, pressure-relief performance can be enhanced by increasing either the number or the diameter of drainage holes near these boundaries.

- (3) Appropriate pressure-limiting total-head control can reduce cumulative discharge while maintaining effective pressure relief. This provides a practical strategy for limiting disturbance to the surrounding groundwater field and reducing the risk of ground settlement induced by groundwater drawdown.
- (4) Increasing the embedment depth of diaphragm walls or seepage cut-off structures extends the seepage path and reduces drainage demand. In the simulated cases, doubling the embedment depth below the foundation base reduced the total seepage discharge by approximately 25%, demonstrating the important hydraulic-control function of diaphragm walls in drainage-based anti-floating systems.
- (5) The numerical results provide design-oriented guidance for drainage pressure-relief systems within the investigated parameter range. The pressure-limiting total head of drainage holes should not be determined solely on the basis of maximising buoyancy reduction; instead, anti-floating safety, pressure-relief efficiency, drainage demand, and potential environmental impacts should be considered together. Increasing the embedment depth of the diaphragm wall can extend the seepage path and reduce groundwater inflow and drainage demand. However, its influence on ground settlement is not necessarily monotonic because changes in the seepage field redistribute pore water pressure and effective stress within the surrounding soil. Therefore, the pressure-limiting total head and diaphragm-wall embedment depth should be jointly considered in engineering design with respect to hydraulic-control efficiency, deformation response, and environmental impacts. Given the limited number of working conditions examined, the results should be interpreted as comparative engineering trends rather than universally applicable optimum design parameters.

Acknowledgement: Not applicable.

Funding Statement: This work was financially supported by the Key Program of the National Natural Science Foundation of China (Grant No. 52538008). This work was also supported by the Hainan Provincial Natural Science Foundation of China (Grant No. ZDYF2026SHFZ043).

Author Contributions: Haonan Zhang: conceptualization, methodology, software, formal analysis, data curation, writing—original draft. Qiqi Wu: validation, data curation, writing—review and editing. Hangming Zhang: investigation, visualization. Liangxu Shen: software, validation, investigation. Linhui Song: methodology, supervision, writing—review and editing. Guoxiong Mei: resources, supervision. All authors reviewed and approved the final version of the manuscript.

Availability of Data and Materials: The data and materials supporting the findings of this study are available from the corresponding author upon reasonable request.

Ethics Approval: Not applicable.

Conflicts of Interest: Given his role as [Guest Editor] of this journal, [Liangxu Shen] had no involvement in the peer review of this article and had no access to information regarding its peer review. Full responsibility for the editorial process for this article was delegated to another journal editor. The authors declare no other conflicts of interest.

References

1. von der Tann L, Sterling R, Zhou Y, Metje N. Systems approaches to urban underground space planning and management—A review. *Undergr Space*. 2020;5(2):144–66. [[CrossRef](#)].
2. Vitali OPM, Celestino TB, Bobet A. Buoyancy effect on shallow tunnels. *Int J Rock Mech Min Sci*. 2019;114:1–6. [[CrossRef](#)].
3. Yang M, Wu J, Lu Q, Li P. Optimization of uplift piles for a base plate considering local anti-floating stability. *Appl Sci*. 2024;14(12):5000. [[CrossRef](#)].

4. Song L, Guo Y, Yang T, Mei G. A new method for anti-floating of underground structures: Experimental study on active–passive combined anti-floating. *Transp Geotech.* 2026;59:101951. [[CrossRef](#)].
5. Chaudhary MTA. Structural condition assessment and functional appraisal of a large basement affected by groundwater leakage. *Eng Struct.* 2013;52:384–93. [[CrossRef](#)].
6. Majumder M, Chakraborty D. Uplift capacity and failure mechanism of under-reamed piles in clay based on lower bound finite element limit analysis. *Proc Natl Acad Sci Ind Sect A Phys Sci.* 2022;92(4):647–58. [[CrossRef](#)].
7. Čas B, Šporin J. Buoyancy effects on structures: A case study on the controlled lowering of a raised underground garage. *Structures.* 2025;71:108121. [[CrossRef](#)].
8. Wong IH. Methods of resisting hydrostatic uplift in substructures. *Tunn Undergr Space Technol.* 2001;16(2):77–86. [[CrossRef](#)].
9. Kong D, Guan Y, Yuan H. Study on the anti-floating water level of the underground structure's comprehensive anti-floating. *Structures.* 2023;56:104921. [[CrossRef](#)].
10. Wu Z, Li Z, Fan J, Cao H, Huang B, Liu H. Up-floating destruction and reinforcement measures of damaged basement based on the bending moment and deformation analysis. *Buildings.* 2023;13(8):1918. [[CrossRef](#)].
11. Yang M, Fu X, Liu H, Wang L, Li Q. Influencing factors, design methods, and buoyancy reduction measures for basement anti-flotation engineering. *Buildings.* 2024;14(9):3005. [[CrossRef](#)].
12. Bikçe M, Örnek M, Cansız ÖF. The effect of buoyancy force on structural damage: A case study. *Eng Fail Anal.* 2018;92:553–65. [[CrossRef](#)].
13. Kim J, Kim JH, Jung K, Yun ST, Lee S. Integrated conceptual–numerical modeling of high-flux groundwater seepage during deep excavation on coastal reclaimed land. *Hydrogeol J.* 2026. [[CrossRef](#)].
14. Luo G, Yang F, Li H, Pan H, Cao H. Simplified calculation method for active anti-floating of elliptical basements by relief wells. *Appl Sci.* 2023;13(23):12647. [[CrossRef](#)].
15. Chen L, Wang W, Bu A. Numerical simulation study of basement water discharge pressure relief method under rainstorm conditions. *Buildings.* 2023;13(11):2772. [[CrossRef](#)].
16. Zhang J. Application of drainage and pressure reduction schemes in basement buoyancy control design: A case study of the Wuhan New City Puyue Mendi Project. *Eng Front.* 2020;5(15):19–20. (In Chinese). [[CrossRef](#)].
17. Cheng S, Zhang Z. Application of water-discharging and pressure-relieving technique in anti-floating design of metro underground parking structure. *Railway Eng.* 2019;59(2):106–9.
18. Ekmekci M, Kalaycioglu NE, Acikel S, Namkhani O, Akman SB. Numerical simulation of uplift pressure and relief drains at a dam reservoir. *Environ Eng Geosci.* 2017;23(2):79–96. [[CrossRef](#)].
19. Stein E, Langford J, Kahlström M. Time series modelling: Applications for groundwater control in urban tunnelling. *Bull Eng Geol Environ.* 2023;82(10):391. [[CrossRef](#)].
20. Azizi S, Salmasi F, Abbaspour A, Arvanaghi H. Weep hole and cut-off effect in decreasing of uplift pressure (case study: Yusefkand Mahabad Diversion Dam). *J Civ Eng Urban.* 2012;2(3):97–101.
21. Pujades-Garnes E, Badiella G, Jurado A, Carrera J, Vazquez-Suñe E. An approach for the design of dewatering systems: The case of an excavation for the construction of the assembly shaft of a tunnel boring machine. *Bull Eng Geol Environ.* 2024;83(7):296. [[CrossRef](#)].
22. Cui W, Jing H, Potts DM, Dong L, Pedone G, Zdravkovic L, et al. Time-step constraints in coupled hydro-mechanical finite element analysis of unsaturated soils. *Comput Geotech.* 2024;165:105914. [[CrossRef](#)].
23. Finnegan NJ, Perkins JP, Nereson AL, Handwerker AL. Unsaturated flow processes and the onset of seasonal deformation in slow-moving landslides. *J Geophys Res Earth Surf.* 2021;126(5):e2020JF005758. [[CrossRef](#)].
24. Li L, Lin H, Qiang Y, Zhang Y, Liang S, Hu S, et al. Stability analysis of rainfall-induced landslide considering air resistance delay effect and lateral seepage. *Sci Rep.* 2024;14:8377. [[CrossRef](#)].
25. Eyo E, Abbey S. Numerical modelling of rainfall influence on the susceptibility of a slope to sliding under induced compression. *Appl Sci.* 2023;13(23):12692. [[CrossRef](#)].
26. Sabetamal H, Sheng D, Carter JP. Coupled hydro-mechanical modelling of unsaturated soils; numerical implementation and application to large deformation problems. *Comput Geotech.* 2022;152:105044. [[CrossRef](#)].
27. Ng CWW, Zhou Q, Zhang Q. A novel surrogate model for hydro-mechanical coupling in unsaturated soil with incomplete physical constraints. *Comput Geotech.* 2025;180:107091. [[CrossRef](#)].

28. Guo H, Ng CWW, Zhang Q. Three-dimensional numerical analysis of plant-soil hydraulic interactions on pore water pressure of vegetated slope under different rainfall patterns. *J Rock Mech Geotech Eng.* 2024;16(9):3696–706. [[CrossRef](#)].
29. Tran TV. Coupled and uncoupled approaches for the estimation of 1-d heave in expansive soils due to transient rainfall infiltration: A case study in central Vietnam. *Int J GEOMATE.* 2019;17(64):152–7. [[CrossRef](#)].