Dynamic analysis of concrete pipes under the coupled effects of traffic load and groundwater level fluctuations

Bin Li1,2 | Hongyuan Fang1,2 | Kangjian Yang1,2 | Peiling Tan1,2 | Fuming Wang1,2

1School of Water Conservancy Engineering, Zhengzhou University, Zhengzhou, China
2Collaborative Innovation Center of Water Conservancy and Transportation Infrastructure Safety Protection, Zhengzhou University, Zhengzhou, China

Abstract
Fluctuations in the groundwater level make the working conditions of concrete pipes below the groundwater level more complex than those of concrete pipes above the groundwater level. This paper describes the application of a three-dimensional (3-D) refined finite element (FE) model to the analysis of a buried concrete pipe subjected to the coupled effects of traffic load and groundwater level fluctuations. First, the dynamic responses of the concrete pipes buried in dry soil were investigated with a full-scale test and an FE model. The results show that the FE model predicted results similar to those observed in field pipes. Then, the dynamic responses based on the numerical results of pipes buried in dry soil under a traffic load and those of pipes buried in saturated soil under the coupled effects of a traffic load and groundwater level fluctuations were compared. Finally, a parametric analysis of the effects of the void ratio, permeability coefficient, and fluctuation period in the groundwater level in relation to the dynamic responses of the pipes was conducted. The results show that the mechanical behaviors of the pipes in dry soil were substantially different from those of pipes in saturated soil. Compared with pipes in dry soil under a traffic load, the stresses at the crown, springline, and invert of the bell under the coupled effects of the traffic load and groundwater level fluctuations were lowered by 44.8%, 52.0%, and 50.3%, respectively, while those of the spigot were lowered by 42.0%, 50.3%, and 49.6%, respectively. The maximum principal stress and the vertical displacement were proportional to the void ratio and the permeability coefficient, while their relationship to the groundwater level fluctuation period was less straightforward.

KEYWORDS
concrete pipes, FE model, full-scale test, groundwater level fluctuation, parametric analysis, traffic load
INTRODUCTION

In recent years, the potential safety hazards caused by underground pipes have become prominent. Pipe leakage, subsidence, corrosion, siltation, and other problems are the main causes of waterlogging, road collapse, and urban black odor water, resulting in major economic losses and social impacts. Many scholars have conducted extensive research on the disaster-causing mechanisms of pipes buried in dry soil under the effects of traffic loads, surface subsidence, seismic loads, and so on.1,5 Scholars believed that the buried depth and the seismic loads and traffic loads were important determinants of the pipe stress distribution, shear displacement, and rotation of joints.

However, for some pipes, the buried depths might be lower than the groundwater level (ie, if the pipes are buried in saturated soil).6 In this case, the microscopic heterogeneity of saturated soil will lead to complex macroscopic physical behaviors, which are sensitive to small changes in the fluid content or pipes for different loads7,8 such as traffic,9-12 machinery,13 or seismic waves.14

Finite element (FE) analyses based on a 2-D model were conducted to investigate the effect of traffic loads on the stress distribution of a pipe buried in unsaturated soil. The results showed that the hoop stresses and deformations of pipes in unsaturated soils with 20% and 60% saturation degrees were substantially different from those of the pipes in dry or saturated soils.15 Dynamic analyses of pipes in saturated soil under a traffic load were investigated based on the Galerkin discrete and Newmark-β methods. The results revealed that the soil surrounding the pipes had a larger pore pressure, which was notably affected by the soil permeability and the drainage boundary condition.16 A theoretical model for characterizing the dynamic behavior of the interaction between the pipe and the saturated soil under seismic loading was proposed. The results demonstrated that the dynamic instability of the pipes was caused by a certain combination of various parameters of the interaction between the pipe and the soil.17 The effects of the low relative density backfill and the partially drained soil response on the uplift resistance of the pipes were revealed by the plane strain uplift resistance of pipes in saturated sand. The results suggested that the soil state had a strong influence on the pipe-soil failure mechanism.18 A model test of the pipes buried in very loose, saturated, fine, and uniform sand was pulled vertically upward at different rates to measure the force on the pipe and the excess pore water pressure around the pipe. The results showed that for sand with zero relative density, the capacity decreased at a faster lifting rate.19

In this study, a full-scale test of a concrete drainage pipe in dry soil was carried out to verify the 3-D refined FE model. The mechanical behaviors of the pipes in dry and saturated soils were compared, and then, a parametric analysis was conducted. Compared with the 2-D FE model in the above references, the 3-D FE model could better reflect the influence of the pipe burial conditions and the microscopic changes in saturated soil on the macrophysical behavior of the pipes. In addition, the effects of excess pore water pressure induced by traffic loads and excess hydrostatic pressure generated by groundwater level fluctuations on the pipe were coupled in this study. The purposes of this study were as follows: 1) to propose a 3-D refined FE model that could be used to predict the coupled effects of traffic loads and groundwater level fluctuations; 2) to investigate the mechanical behavior of concrete pipes under the coupled effects of traffic loads and groundwater level fluctuations; and 3) to provide theoretical guidance for the design and construction of pipes buried below the groundwater level.

FULL-SCALE TEST DESCRIPTION

A full-scale test of the buried concrete pipes was conducted at the National Local Joint Engineering Laboratory of Major Infrastructure Testing and Rehabilitation Technology, located at Zhengzhou University. The geological exploration
of the test field showed that the buried depth of the groundwater level was far deeper than that of the field pipes, so it could be considered that the pipes in this full-scale test were buried in dry soil. The test pipes were a typical concrete pipe defined in Concrete and Reinforced Concrete Pipes (GB/T 11836-2009), with a strength grade of C30, an inner diameter of 1000 mm, a wall thickness of 100 mm, and an effective length of 2.0 m. The test pit had a length of 16 m and a width of 5.0 m, as shown in Figure 1. The thicknesses and physical and mechanical properties of each soil layer are shown in Table 1.

### 2.1 Bedding and backfilling

A 120° arc-shaped sandstone bedding was laid at the bottom of the pipe. The first layer was laid with a thickness of 20 cm, using natural medium-coarse sand with particle sizes of less than 5 mm and a compactness of 95 ± 2%. The second layer was laid with a thickness of 30 cm, utilizing well-graded gravel with diameters of less than 20 mm and a compactness of 93 ± 2%. Both sides of the pipe were symmetrically backfilled at a thickness of no more than 30 cm per layer and tamped with vibration rammer until they were backfilled to the ground, as shown in Figure 1.

### 2.2 Application of the vehicle load

The vehicle load was applied by a single-axle two-wheeled heavy truck moving at a speed of 36 km/h. The truck weighed 130 kN, and each standard block weighed 10 kN. The axle load of the vehicle was obtained by loading standard blocks with a rated mass until the static pressure applied to the ground surface of each tire was 0.70 MPa, which conformed to the BZZ-100 single-axle and double-wheel load design standard specified in Specifications for Design of Highway Asphalt Pavement (JTG D50-2017).

### 2.3 Data acquisition

BQ-80AA-P120 resistance strain gauges manufactured by the AVIC Zhonghang Electronic Measuring Instruments Co., Ltd., with a sensitivity of 2.2 ± 1%, were arranged on the inner and outer walls of the bells, spigots, and barrels of the middle six pipe segments. No strain gauges were attached to the pipes at the ends of both sides due to the influence of the test pit boundary. The surfaces of the strain gauges were coated with epoxy resin and silica gel, which were cured with structural adhesive to prevent the resistance strain gauges from being damaged. In addition, glass and structural glue

### TABLE 1 Thicknesses and physical and mechanical properties of each soil layer

| Soil type       | Thickness $h$ (m) | Density $\gamma$ (kN/m³) | Modulus $E$ (MPa) | Poisson's ratio, $\mu$ | Cohesion $c$ (kPa) | Friction angle $\varphi$ (°) |
|-----------------|-------------------|-------------------------|-------------------|------------------------|--------------------|-------------------------------|
| Miscellaneous fill | 0.50              | 15.0                    | 5.6               | 0.30                   | 6.00               | 15.2                          |
| Silt            | 1.20              | 18.0                    | 15.2              | 0.25                   | 15.0               | 15.3                          |
| Silty sand      | 3.50              | 18.7                    | 30.4              | 0.35                   | 2.00               | 21.5                          |
| Silt            | 4.20              | 18.3                    | 22.5              | 0.30                   | 14.0               | 17.4                          |
| Silty sand      | 7.50              | 18.5                    | 42.8              | 0.32                   | 1.50               | 24.3                          |
| Fine sand       | 20.0              | 20.7                    | 51.3              | 0.33                   | 0.10               | 32.5                          |

### FIGURE 2 Instrumentation on the pipe section in the full-scale test
were used for the double reinforced signal lines, as shown in Figure 2.

A DH3816N stress-strain test and analysis system manufactured by Jiangsu Donghua Testing Technology Co., Ltd., were used to measure the strains observed by the strain gauges. The system contained 60 channels and could achieve real-time communication with the computer through a Gigabit Ethernet. In addition, the functions of the synchronous automatic measurement of the conductor resistance and the correction of all channels could perfectly meet the precision requirements of the test data.

3 | FINITE ELEMENT MODEL DESCRIPTION

A 3-D FE model with gasketed bell-and-spigot joints was developed to obtain the pipe response under the coupled effects of a traffic load and groundwater level fluctuations. The models of the soils, pipes, and gaskets are shown in Figure 3 (A1-A8 represent the pipe segments, and J1-J7 represent the bell-and-spigot joints). The behavior of the soils was described by a nonlinear elastic perfectly plastic constitutive model. The pipes were modeled by a concrete damaged plasticity model, which could characterize the typical properties of concrete. All the meshes were discretized by Hypermesh 13.0, and the 3-D solid eight-node hexahedral linear reduction integral elements (C3D8R) were used for the elements of the dry soil, pipes, and gaskets. The stress-pore pressure coupling element C3D8RP was used for the saturated soil, as shown in Figure 3. The physical and mechanical properties of the saturated soil and pipe are shown in Table 2.

3.1 | Boundary conditions

There were two types of boundary conditions involved in the EF model.

(a) Force-displacement boundary: An infinite element transmission boundary utilizing the 1/\(r\) far-field attenuation shape function was adopted to reduce the influence of the stress wave reflection at the force-displacement boundary. The shape functions were selected as:

\[ N_i' = 0.25(1 + \eta_i)(1 + \xi z_i) f_i, i = 1, 2, 3, 4 \]  
\[ N_i' = 0.25(1 + \eta_i)(1 + \xi z_i) f_2, i = 5, 6, 7, 8, \]  
where \( f_1 \) and \( f_2 \) are the attenuation functions, expressed as:

\[ f_1 = \frac{-2\xi}{1 - \xi} \]  
\[ f_2 = \frac{1 + \xi}{1 - \xi} \]  

For any point in an infinite element,

\[ r = \sum_{i=1}^{4} N_i' r_1 + \sum_{i=5}^{8} N_i' r_2 \]  
\[ \xi \] can be given as

\[ \xi = 1 - \frac{2(r_1 - r_2)}{r - r_2 + 2(r_1 - r_2)} \]  

It can be verified by Equation (5) that when \( \xi \) is taken to be \(-1, 0, 1\) and \( r \) is equal to \( r_1, r_2, \) or \( \infty \), respectively. Equation (5) can be expressed as:

\[ \xi = 1 - \frac{2(a - 1)}{r + 2(a - 2)r_2} \]  

Equation (6) shows that \( \xi \) exhibits 1/\( r \) attenuation, and the purpose of adjusting the node in progress can be achieved by changing the value of \( \alpha \).

(b) Pore pressure boundary: The initial effective stress of saturated soil can be expressed as (the \( z \)-axis is specified to be vertically upward),

\[ \frac{d\sigma_w}{dz} = -\gamma_w \]  

where \( \gamma_w \) is the user-defined specific weight of the pore fluid.

If the shear stress is ignored, the equilibrium in the vertical direction is,

\[ \frac{d\sigma_{zz}}{dz} = \rho_d g + \eta \gamma_w \]  

where \( \rho_d \) is the dry density, and \( \eta \) is the fraction of the pore pressure, which represents the ratio of the difference between the total stress and the effective stress to the pore water pressure. It has been suggested that \( \eta \) is equal to the porosity \( n \).

In ABAQUS, the positive stress is specified to be tensile stress. Therefore, the effective stress \( \sigma \) can be defined by the total stress \( \sigma' \),

\[ \sigma' = \sigma + u_w I, \]  

where \( I \) is a unit matrix.

The effective stress of saturated soil at any depth below the surface can be given by:

\[ \sigma = \int_{z_0}^{z} \frac{d\sigma_{zz}}{dz} = \rho_d g + \frac{e_0}{1 + e_0} \gamma_w (z - z_0). \]
Combining the above with Equation (9), the initial effective stress formula of saturated soil can be expressed as:

\[ \sigma' = \left( \rho_d g + \frac{e_0}{1+e_0} \gamma_w \right) (z-z_0) - \gamma_w (z-z_0) \]

\[ = \left[ \rho_d g - \gamma_w \left( 1 - \frac{e_0}{1+e_0} \right) \right] (z-z_0) \]

\[ = \left[ \rho_d g - \gamma_w \left( \frac{1+e_0-e_0}{1+e_0} \right) \right] (z-z_0) \]

\[ = \left[ \rho_d g - \gamma_w \frac{1}{1+e_0} \right] (z-z_0). \]

### 3.2 | Pipe-soil interaction

The separation and sliding properties of the pipe-soil interface were simulated by the contact element. In the tangential direction of the interface, the penalty method, which permitted some relative motion of the surfaces when they should have been adherent, was used. While the surfaces were sticking, the magnitude of sliding was limited to this elastic slip. Analyses in which \( \mu \) (the friction coefficient) varied between 0.2 and 0.4 suggested that the effects of those changes in \( \mu \) on the behavior of the pipe were negligible. Therefore, all the analyses featured a \( \mu \) value of 0.3. In the normal direction of the contact surface, the classic Lagrange multiplier method of constraint enforcement was used.

### 3.3 | Simplification of the traffic load

The effects of the traffic load induced by the vibration of the vehicle itself and the roughness of the road on a certain point of pavement can be expressed as:

\[ F(t) = p + q(t), \]

where \( p \) is the static load, and \( q(t) \) is the wheel dynamic overload, as given in Equations (14) and (15):

\[ q(t) = q_{\text{max}} \sin^2 \left( \frac{\pi}{2} + \frac{\pi t}{T} \right) \]

### Table 2: Physical and mechanical properties of the simulated materials

| Parameter                                      | Unit  | Value   |
|-----------------------------------------------|-------|---------|
| **Saturated soil**                            |       |         |
| Density, \( \rho_d \)                         | kg/m³ | 1400    |
| Elastic modulus, \( E \)                      | MPa   | 18.0    |
| Poisson's ratio, \( \mu \)                    |       | 0.25    |
| Friction angle, \( \varphi \)                 | °      | 15.0    |
| Damping, \( \alpha \)                         |       | 0.32    |
| Permeability coefficient, \( k \)             | m/s   | 1e-07   |
| Void ratio, \( e \)                           |       | 0.8     |
| Saturation, \( S \)                           |       | 1.0     |
| **Pipes**                                      |       |         |
| Elastic modulus, \( E \)                      | MPa   | 30 000  |
| Poisson's ratio, \( \mu \)                    |       | 0.2     |
| Density, \( \rho \)                           | kg/m³ | 2400    |
| Dilation angle, \( \psi \)                    | °      | 30.0    |
| Eccentricity, \( e \)                         |       | 0.1     |
| Ratio of the initial equibiaxial compressive yield stress to the initial uniaxial compressive yield stress, \( n \) |       | 1.16    |
| Ratio of the second stress invariant on the tensile meridian to the second stress invariant on the compressive meridian at an initial yield for any given value of the pressure invariant, such that the maximum principal stress is negative, \( k \) |       | 0.67    |
| Viscosity parameter, \( \nu \)                |       | 0.005   |
where \( q_{\text{max}} \) is the magnitude of the wheel dynamic overload, \( T \) is the period, \( l \) is the length of the tire tread imprint, generally taken as 15 cm, and \( v \) is the vehicle speed.

Taking the dynamic load coefficient as 1.2, the final formula of the traffic load can be given as:

\[
F(t) = p + 0.2p \sin^2 \left( \frac{\pi t}{T} \right). \tag{16}
\]

The traffic load formula was compiled into the “Dload” subroutine, which could make the traffic load change with time and space according to the specified speed and amplitude.

### 3.4 Model of the groundwater level fluctuation

To simulate the case in which the groundwater level was lower than the buried depth of the pipes, it was assumed that the highest groundwater level was at 0.7 m (the covering depth of the pipe was 1.5 m), and the lowest groundwater level was at 4.7 m (i.e., the fluctuation amplitude was 2.0 m), as shown in Figure 4. Since the groundwater level fluctuated in every moment, it was very difficult to determine the continuous variation of the soil saturation with the fluctuations in the groundwater level. Therefore, an assumption was made that the surface with the highest groundwater level was the interface between the dry soil and the saturated soil (Figure 4).

The fluctuation of the pore pressure caused by the groundwater level was assumed to be a harmonic fluctuation, given as,

\[
p = p_0 \sin \omega t, \tag{17}
\]

where \( p \) is the pore pressure, \( p_0 \) is the pore pressure fluctuation amplitude \( (p_0 = \gamma_w h_0) \), where \( \gamma_w \) is the specific weight of the water, and \( h_0 \) is the fluctuation amplitude of the groundwater level), \( \omega \) is the fluctuation circle frequency, and \( t \) is the fluctuation time.

The groundwater level fluctuation formula was compiled into the user subroutine “Disp”. The basic parameters related to groundwater in the model are listed in Table 3.

### 4 RESULTS AND DISCUSSION

#### 4.1 Verification of the FE model using the field test data

The FE model was first verified using the circumferential strains on the outer and inner walls of the spigot (L1-L1) and the bell (R1-R1) of the field pipes shown in Figure 2. As shown in Figure 5, the outer walls of the crowns \((315^\circ-45^\circ)\) and inverts \((135^\circ-225^\circ)\) of the bell and spigot were under compression, while those of the springlines \((67.5^\circ-112.5^\circ\) and \(247.5^\circ-292.5^\circ)\) of the bell and spigot were under tension. However, the strains at the shoulders and haunches of the bell and spigot were very small. In addition, it was found that the circumferential strain on the outer wall of the bell was larger overall than that at the spigot. The analysis indicated that the convex structure of the bell was more restrained by the soil, while the spigot was inside the bell, and its strain was buffered by the gasket. Figure 6 shows that the inner walls of the crowns \((315^\circ-45^\circ)\) and inverts \((135^\circ-225^\circ)\) of the bell and spigot were in tension, while those of the springlines \((45^\circ-135^\circ, 225^\circ-315^\circ)\) of the bell and spigot were in compression. Similarly, the circumferential strain on the inner wall of the bell was significantly greater than that of the spigot.

![FIGURE 4](image)

**FIGURE 4** Fluctuation range of the groundwater level in the numerical model

| TABLE 3 | Basic parameters of the groundwater level |
|----------|------------------------------------------|
| Parameter (unit) | Value |
| Specific weight of water, \( \gamma_w \) (kN) | \( \gamma_w g \) |
| Initial groundwater level, \( Z_0 \) (m) | 5.3 |
| Highest groundwater level, \( Z_1 \) (m) | 7.3 |
| Lowest groundwater level, \( Z_l \) (m) | 3.3 |
| Fluctuation amplitude, \( h_0 \) (m) | 2.0 |
| Fluctuation circular frequency, \( \omega \) | \( 2\pi/T \) |
| Fluctuation period in the groundwater level, \( T \) (s) | 2.592e6, that is, the period is 30 d. |
| Total fluctuation time, \( t \) (s) | 3.1104e7, that is, the total time is 1 y. |
For concrete materials, damage is mainly caused by the loading exceeding the tensile strength. Therefore, the failure of concrete pipes can be predicted from the tension stress or tension strain. By comparing the tension strains on the inner and outer walls of the bell and spigot, it was found that the inner wall of the invert was the most easily damaged position.

A comprehensive analysis of Figures 5 and 6 showed that the strain distributions and values predicted by the FE model were similar to those measured by the strain gauges. To more intuitively display the difference between the experimental and simulated data, the data with the greatest difference are shown in Table 4 (the test data were regarded as the assumed real values). The maximum difference between the experimental and simulated data was 12.6%, reflecting reliable FE model predictions.

### Table 4

| Location     | Type   | Values (με) | Relative error |
|--------------|--------|-------------|----------------|
| Outer-bell   | Tested | −28.5       | 12.6%          |
|              | Calculated | −32.1     |                |
| Outer-spigot | Tested | −22.5       | 10.7%          |
|              | Calculated | −24.9     |                |
| Inner-bell   | Tested | 31.7        | 11.0%          |
|              | Calculated | 35.2     |                |
| Inner-spigot | Tested | −30.2       | 9.9%           |
|              | Calculated | −33.2     |                |

#### 4.2 Comparison of the pipes in dry soil and the pipes in saturated soil with the fluctuations in the groundwater level

In this section, the pipes in dry soil affected by the traffic load and the pipes in saturated soil affected by the traffic load and the fluctuations in the groundwater level were compared based on the validated FE model. The circumferential maximum principal stresses along the outer surfaces of the bell and spigot at joint J4 are plotted in Figure 7. It can be seen that the stresses of the bell and spigot of the pipe buried in saturated soil were much smaller than those of the pipe buried in dry soil. Compared with the pipe in dry soil under the traffic load, the stresses at the crown, springline, and invert of the bell under the coupled effects of the traffic load and the groundwater level fluctuation were reduced by 44.8%, 52.0%, and 50.3%, respectively, while those of the spigot were reduced by 42.0%, 50.3%, and 49.6%, respectively.

It was found that the circumferential stress of the pipe in unsaturated soil was lowered by 10% to 80% when compared with that of the pipe in dry soil. It was believed that this reduced stress was because the enhanced stiffness induced...
by the suction in the wet soil resisted the pipe mobilization and constrained its deformations during the traffic loads. However, the mechanical behaviors of the pipe in dry and unsaturated soil were quite different from those of the pipe in saturated soil. A large amount of total stress induced by the traffic load was shared by the pore water pressure for the pipes in saturated soil. In addition, the buoyancy effect of the pore water pressure on the pipes weakened the shear displacements at the joints, which greatly reduced the stress of the pipe.

The comparison of the pipes in dry soil under the effect of the traffic load and the pipes in saturated soil under the coupled effects of the traffic load and the groundwater level fluctuation indicated that the groundwater level fluctuation had an obvious influence on the pipes. Therefore, a parametric analysis of the void ratio, permeability coefficient, and fluctuation period of the groundwater level that affect the dynamic response of the pipes was necessary.

4.3 | Parametric analysis

4.3.1 | Effect of void ratio

The void ratio is the ratio of the pore volume in soil to the volume of the solid particles. In general, the void ratio is a major factor affecting the water content and effective stress of saturated soils. The parameters adopted in this section are the basic parameters listed in Tables 2 and 3 and with the exception of the void ratio $e$. $e = 0.5, 0.8$, and $1.0$. Figure 8 illustrates the fact that the pore water pressure was inversely proportional to the void ratio. Furthermore, when the void ratio increased from 0.5 to 0.8, the decrease in the pore water pressure was slightly greater than that when the void ratio increased from 0.5 to 0.8.

Figure 9 reveals the fact that the circumferential stresses of the bell and spigot were proportional to the void ratio. This finding was because the pore water pressure decreased with the increasing void ratio, which led to an increase in the effective stress, thus enhancing the effect on the pipe. Moreover, when the void ratio increased from 0.8 to 1.0, the increment of the circumferential stress was slightly larger than the increment when the void ratio increased from 0.5 to 0.8.

The curves of the maximum values of the maximum principal stress and the vertical displacement versus time were obtained using the ABAQUS Plug-ins and are plotted in Figure 10. It can be seen that the stress and vertical displacement significantly increased with the increasing void ratio. In addition, Figure 10 reveals that decreases in the groundwater level led to increases in the maximum principal stress and the vertical displacement of the pipe. When the void ratio increased from 0.5 to 0.8 and 1.0, the

FIGURE 7 Comparison of the circumferential maximum principal stresses $\sigma_\theta$ (MPa) between the pipes buried in dry soil and saturated soil: (a) bell and (b) spigot at joint J4

FIGURE 8 Isogram of the pore water pressure with different void ratios: (A) $e = 0.5$, (B) $e = 0.8$, and (C) $e = 1.0$

FIGURE 9 Comparison of the circumferential maximum principal stresses $\sigma_\theta$ (MPa) between the pipes buried in dry soil and saturated soil: (a) bell and (b) spigot at joint J4

FIGURE 10 Isogram of the pore water pressure with different void ratios: (A) $e = 0.5$, (B) $e = 0.8$, and (C) $e = 1.0$
4.3.2 | Effect of the permeability coefficient

The permeability coefficient is a quantitative indicator of the soil permeability that characterizes the transfer efficiency of water in soils. The parameters described in this section were the basic parameters shown in Tables 2 and 3, except for the permeability coefficient $k$. $k = 1\times10^{-5}$, $1\times10^{-6}$, and $1\times10^{-7}$ m/s were considered in this section. Figure 11 shows that the pore water pressure increased with the decreasing permeability coefficient, which means that the smaller the permeability coefficient was, the greater the pore water pressure around the pipe was. This phenomenon occurred because a smaller permeability coefficient led to a decrease in the soil drainage capacity, thus slowing the dissipation rate of the pore water pressure.

The circumferential stresses along the outer surfaces of the bell and spigot at joint J4 presented in Figure 12 revealed that the circumferential stresses were proportional to the permeability coefficient because the pore water pressure decreased with an increasing permeability coefficient (Figure 11), resulting in an increasing effective stress of the soil and, thus, enhancing the constraint effect on the pipe. Moreover, when the permeability coefficient increased from $1\times10^{-7}$ to $1\times10^{-6}$, the stress increments of the bell and spigot were slightly smaller than those when the permeability coefficient increased from $1\times10^{-6}$ to $1\times10^{-5}$. This result demonstrates that the greater the increment of the permeability coefficient was, the greater the increment of pipe stress was.

The curves of the maximum values of the maximum principal stress and vertical displacement versus time are plotted in Figure 13. It can be seen that the maximum principal stress and the vertical displacement were proportional to the permeability coefficient. Furthermore, when the permeability coefficient increased from $1\times10^{-7}$ to $1\times10^{-6}$ and $1\times10^{-5}$ m/s, the increments of the peak stress were 0.036 and 0.045 MPa, while those of the vertical displacement were 0.18 and 0.14 mm, respectively.

4.3.3 | Effect of the fluctuation period of the groundwater level

The fluctuation period of the groundwater level characterizes the rising and falling frequency of the groundwater level and is one of the critical factors affecting the mechanical
The isobaric contours of the pore water pressure under the different fluctuation periods presented in Figure 14 display the fact that the pore water pressure with a fluctuation period of 20 days was the largest, followed by a fluctuation period of 10 days. This finding indicates that the influencing mechanism of the fluctuation period of the groundwater level on the pore water pressure was not straightforward.

FIGURE 11 IsoGram of the pore water pressure with different permeability coefficients, (A) \( k = 1 \times 10^{-05} \) m/s, (B) \( k = 1 \times 10^{-06} \) m/s, and (C) \( k = 1 \times 10^{-07} \) m/s

FIGURE 12 Comparison of the circumferential maximum principal stresses \( \sigma_\theta \) (MPa) of the (A) bell and (B) spigot at joint J4 under different permeability coefficients

FIGURE 13 Comparison of the time history curves of the (A) maximum principal stress and (B) vertical displacement under different permeability coefficients

properties of a pipe buried below the groundwater level. \( T = 10, 20, \) and 30 days were considered as examples in this section, while the other parameters are shown in Tables 2 and 3. The isobaric contours of the pore water pressure under the different fluctuation periods presented in Figure 14 display
Figure 15 shows that the circumferential stresses along the outer surfaces of the bell and spigot with a fluctuation period of 30 days were the largest, with the values of 0.2 and 0.13 MPa, while those of the fluctuation period of 20 days were the smallest, with values of 0.09 and 0.05 MPa. These results were consistent with the effect of the fluctuation period on the pore water pressure, as shown in Figure 14.

To further elucidate the influence mechanism of the fluctuation period of the groundwater level on the pipes, the stresses of the crowns, springlines, and inverts of the bell and spigot are shown in Table 5. It can be seen that when the fluctuation period was 30 days, the stresses at the crown, springline, and invert of the bell were 0.06, 0.07, and 0.08 MPa greater, respectively, than those with a fluctuation period of 10 days and 0.12, 0.13, and 0.12 MPa greater, respectively, than those with a fluctuation period of 20 days. Additionally, for the 30 days fluctuation period, the stresses at the crown, springline, and invert of the spigot were 0.05, 0.04, and 0.11 MPa greater, respectively, than those with a fluctuation period of 10 days and 0.11, 0.08, and 0.16 MPa greater, respectively, than those with a fluctuation period of 20 days.

The curves of the maximum values of the maximum principal stress and vertical displacement versus time are shown in Figure 16, which shows that the variation curves of the stress and the vertical displacement versus time corresponding to the fluctuation periods of 10, 20, and 30 days were repeated 36 times, 18 times, and 12 times, respectively. The peak stress and peak vertical displacement with a fluctuation period of 30 days are 0.10 MPa and 0.34 mm larger, respectively, than those with a fluctuation period of 20 days are 0.04 MPa and 0.15 mm larger, respectively, than those with a fluctuation period of 10 days.

5 | CONCLUSION

The coupled effects of traffic load and groundwater level fluctuation on the mechanical properties of pipes below the groundwater level were investigated using an advanced field test and a 3-D FE model, and the following conclusions were drawn:

1. The 3-D FE model could accurately predict the strain distributions and the values observed by the strain gauges, indicating that the 3-D FE model could be used to evaluate the mechanical behaviors of the concrete pipes under the coupled effects of traffic loads and groundwater level fluctuations.
2. The stresses and vertical displacements of the pipes in dry soil were significantly different from those of the pipe in saturated soil. Compared with pipes in dry soil under a traffic load, the stresses at the crown, springline, and invert of the bell under the coupled effects of a traffic load and groundwater level fluctuations were lowered by 44.8%, 52.0%, and 50.3%, respectively, while those of the spigot were lowered by 42.0%, 50.3%, and 49.6%, respectively.

3. The parametric analysis indicated that the stress and the vertical displacement were directly proportional to the void ratio and the permeability coefficient, while the pore water pressure was inversely proportional to the void ratio and the permeability coefficient. However, the influence of the groundwater level fluctuation period on the stress and the vertical displacement was slightly more complicated than the influence of the void ratio and the permeability coefficient.

### Table 5

| Location | Period (days) | Crown (MPa) | Springline (MPa) | Invert (MPa) |
|----------|---------------|-------------|------------------|--------------|
| Bell     | 10            | −0.47       | 0.14             | −0.44        |
|          | 20            | −0.41       | 0.08             | −0.40        |
|          | 30            | −0.53       | 0.21             | −0.52        |
| Spigot   | 10            | −0.30       | 0.09             | −0.27        |
|          | 20            | −0.24       | 0.05             | −0.22        |
|          | 30            | −0.35       | 0.13             | −0.38        |

### Figure 16

Comparison of the time history curves of the (A) maximum principal stress and (B) vertical displacement under different groundwater level fluctuation periods

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### ORCID

Bin Li https://orcid.org/0000-0002-2213-2704
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