Research Article

Active and Passive Earth Pressure Calculation Method for Double-Row Piles considering the Nonlinear Pile Deformation

Yijun Zhou and Yulong Chen

1College of Civil and Architectural Engineering, North China University of Science and Technology, Tangshan 063000, China
2School of Energy and Mining Engineering, China University of Mining and Technology, Beijing 100083, China

Correspondence should be addressed to Yulong Chen; chenyulong@cumtb.edu.cn

Received 9 February 2022; Accepted 25 March 2022; Published 26 April 2022

Copyright © 2022 Yijun Zhou and Yulong Chen. This is an open access article distributed under the Creative Commons Attribution License, which permits unrestricted use, distribution, and reproduction in any medium, provided the original work is properly cited.

The double-row pile supporting structure has been widely used in foundation pit excavations. When analyzing the effect of earth pressure on the pile structure, previous research only considered the double-row piles as the rigid body and the pile-soil interaction has not been examined. In this study, a theoretical model was developed based on Duncan-Chang’s hyperbolic theory to calculate earth pressures in the active and passive zones of the double-row pile supporting structure. The model considered the nonlinear effect of the pile deformation on the active and passive earth pressures. The macroscopic pile-soil interaction was converted into a microscopic stress-strain relationship at a certain point in the soil body, reflecting the nonlinear effect of pile deformation on earth pressure. Numerical simulation and large-scale field tests have been conducted to verify the proposed model. The results show that the average values of the parameters obtained by numerical simulation are $a = 0.38$, $b = -0.253$ for the active zone and $a = 0.00612$, $b = -0.729$ for the passive zone. Based on the values of $a$ and $b$, the predicted active and passive earth pressures stemming from the developed model agreed well with those obtained from field tests. The developed model in this study can be used to predict the distribution of active and passive earth pressures for double-row pile supporting structures.

1. Introduction

Rapid urbanization, in recent years, accelerates the construction of a bunch of tall buildings and subways in large cities [1]. The construction of deep foundation pits in tall building projects might have a significant impact on the surrounding infrastructures, such as metro lines and buildings. Various types of piles have been developed to ensure the stability of the foundation pit during excavation, including the cantilever single-row pile wall, the anchored single-row pile wall, and the double-row pile supporting structure.

In the construction of coal mine towers, the piles also play an important role, which can transfer the upper load to the foundation soil through the piles. The main principle is to transfer the upper load to the soil layer through the pile side friction resistance and the pile end resistance, so as to reduce the possibility of coal mine damage caused by excessive concentrated load. Especially in areas with abundant groundwater, choosing appropriate piles can effectively resist the effect of buoyancy in the soil, thereby improving the safety of the coal mine.

Various types of piles with strong constraints continue to appear, and the double-row pile support structure is one of them. Compared with the cantilever single-row piles, the double-row pile supporting structure has the characteristics of large lateral rigidity and strong overturning resistance. Compared with the anchored single-row piles, the double-row pile supporting structure has strong restraint ability and saves construction space [2].

The double-row pile supporting structure is mainly composed of front-row piles, rear-row piles, connecting beams, and crown beams (Figure 1). It is flexible and diverse in layout. As shown in Figure 2, various types of double piles have been developed, such as plum blossom, rectangular lattice, and zigzag [3]. To date, the design of double-row piles in actual projects relies heavily on empirical methods.
Zigzag porting system is not insu
ear pressure on the stability of the double-row pile sup-
– parameter improvements [4
optimizations, pile deformation mechanisms, and design
research on double-row piles mainly focuses on structural
methods produce variation more or less. The existing
However, because of the complex formation, the empirical
methods produce variation more or less. The existing
research on double-row piles mainly focuses on structural
optimizations, pile deformation mechanisms, and design
parameter improvements [4–9]. The study on the effects of
earth pressure on the stability of the double-row pile sup-
porting system is not insufficient.

During the excavation of the foundation pit, the pile
body deforms due to soil pressure and the soil and pile inter-
act with each other. Considering the pile-soil interaction, the
magnitude and distribution of earth pressure undergo com-
plex changes with the deformation of the supporting struc-
ture to reach a new equilibrium. Some scholars have
studied the impacts of earth pressure on the double-row pile
supporting structure based on the finite soil theory [10–14].
The finite soil theory is the earth pressure value calculated
according to the actual stress state of the soil body not the
semi-infinite state of space. The earth pressure value is in
line with the actual situation. In addition, soil arching effects
on the double-row piles were investigated, when they were
applied on stabilizing slopes [15–19].

However, previous research only considered the double-
row pile model to be the rigid body when calculating the
earth pressure. The pile-soil interaction during the evacua-
tion of foundation pits has not been examined [20–27].
Several limitations of previous works are summarized as
follows. (1) Soils are assumed as ideal elastic materials and
subjected to the Mohr-Coulomb strength theory [28]. (2)
In terms of the limit equilibrium method for analyzing earth
pressure, the earth pressure is distributed in a triangle shape
from top to bottom along the pile body. However, the earth
pressure distributed is not in a triangle shape in the actual
engineering; it is just a simplifying assumption. (3) When
considering the effect of earth pressure on the supporting
structure, the structure is always regarded as a rigid body.
In other words, the supporting structure only moves in
translation or rotation corresponding to the variations of
earth pressure and the pile-soil interaction is not considered.
(4) The calculation method adopted in Chinese Standard
JGJ120-2012, “Technical Specification for Building Founda-
tion Pit Support,” is based on Winkler’s elastic foundation
beam theory. That is, when calculating the earth pressure
in the passive zone in front of the front row of piles, the
passive earth pressure coefficient ($K_s$) of the soil is simplified
as the stiffness coefficient of the soil spring. However, the
method in the standard only considers the linear effect of
the deformation of the retaining wall when calculating the
earth pressure for the retaining structure [29].

Based on the Duncan-Zhang hyperbolic model, this
study focuses on the influence of the deformation of the
double-row piles on the earth pressure in the active and pas-
sive areas during the excavation of the foundation pit. In
this study, the macroscopic pile-soil interaction is transformed
into the microscopic stress-strain relationship at a certain
point in the soil. Subsequently, a calculation model of the
earth pressure in the active and passive areas of the
double-row pile supporting structure considering the in-
fluence of the pile body deformation was established; the
proposed model is verified by using field test data.

2. Materials and Methods

2.1. Duncan-Chang Hyperbolic Model under the Unloading
State of the Foundation Pit

2.1.1. Stress-Strain Relationship of Soil in the Unloaded State of the Foundation Pit. To adapt to the new equilibrium rela-
tionship, the double-row pile supporting structure will have
a complex interaction with the surrounding soil, during the
excavation of the foundation pit. With the excavation of
the foundation pit, below the excavation surface, the stress
change trend of a certain point in the soil in front of the
front row piles is as follows: the vertical stress decreases,
and the horizontal stress continues to increase. The stress
change trend of a certain point in the soil behind the back
row piles with the excavation of the foundation pit is as
follows: the vertical stress remains unchanged, and the
horizontal stress continues to decrease. This is similar to
the stress path unloading test. The axial pressure of the for-
mer decreases and the confining pressure increases, and the
axial pressure of the latter does not change but the confining
pressure decreases. It is worth mentioning that through a
series of model test studies, Yang and Lu found that the soil
outside the slip surface of the foundation pit is slightly
affected by the deformation of the supporting structure
[30]. It can be considered that the deformation of the sup-
porting structure only affects the soil within the slip surface.
Therefore, this article assumes that during the excavation of
the foundation pit, the deformation of the pile body of the
double-row pile supporting structure only affects the earth

![Figure 1: Schematic diagram of typical double-row piles.](image1.jpg)

![Figure 2: The forms of double-row piles.](image2.jpg)
Because the strain in the common condition, and the stress and the minimum principal stress, respectively. The Duncan-Chang hyperbolic model can not only reflect the nonlinearity of soil deformation but also reflect the elastoplastic characteristics of the soil to a certain extent. This model was proposed by Conner in 1963 through a large number of triaxial stress tests. This curve is about $\sigma_3$ ~ $\varepsilon_3$ and can be applied to most soils. The curve is shown in Figure 3(a), and its expression is as follows:

$$\sigma_1 - \sigma_3 = \frac{\varepsilon_a}{a + b\varepsilon_a}. \quad (1)$$

In the formula, $\sigma_1$ and $\sigma_3$ are the maximum principal stress and the minimum principal stress, respectively. $\varepsilon_a$ is the strain in the common condition, and $\varepsilon_1$ is the maximum principal strain. The unit is kPa, while $a$ and $b$ are the test constants. In the conventional triaxial compression test, because $\varepsilon_a = \varepsilon_1$, formula (1) can also be deformed as follows:

$$\frac{\varepsilon_1}{\sigma_1 - \sigma_3} = a + b\varepsilon_1. \quad (2)$$

Sorting and simplifying according to the form of $(\varepsilon_1/ (\sigma_1 - \sigma_3)) ~ \varepsilon_1$, they are linearly distributed, as shown in Figure 3(b).

2.2. Earth Pressure Calculation in the Passive Zone

2.2.1. Establishment of a Calculation Model. The calculation model adopts a conventional cantilever double-row pile supporting structure. The area below the excavation surface of the foundation pit is regarded as the passive zone, and the earth pressure calculation model of the passive zone is established as shown in Figure 4.

$z$ is the calculated depth of a certain point of the soil in the disturbance zone below the excavation surface; $d$ and $h$ are the pile length and the excavation depth of the foundation pit, respectively; $\beta$ is the angle between Rankine’s passive slip surface and the horizontal plane; $L$ is the horizontal distance from the slip surface at depth $z$ to the front row of piles; $\eta(z)$ is the horizontal displacement of the pile at depth $z$.

2.3. Force Analysis. According to the Moore-Coulomb strength failure criterion, the angle $\beta$ between the passive slip surface and the horizontal plane is $45° - \varphi/2$ and $\varphi$ is the internal friction angle of the soil in the passive zone. The displacement $(\eta(z))$ of the pile body at depth $z$ mainly affects the horizontal deformation of the soil between the pile body and the slip surface. Assume that the displacement $\eta(z)$ of the pile body is equal to the deformation of the soil in the disturbance zone at the same depth and the strain of the soil between the pile body and the slip surface shows a linearly decreasing trend. Then, the strain of the soil at any point in the disturbance zone from the horizontal distance of the pile body can be expressed as follows:

$$\varepsilon = \frac{2(L-x)\eta(z)}{L^2}. \quad (3)$$

Establish equation (4) according to equation (3):

$$\int_0^L \frac{2(L-x)\eta(z)}{L^2} dx = \eta(z). \quad (4)$$

When the pile body does not move, it can be assumed that the soil at any point in the passive zone is in a state of isobaric consolidation stress. In other words, the stresses in the three main axis directions are equal at this time, as shown in equation (5). Since the soil is in a state of isobaric consolidation, the horizontal direction can be regarded as the direction of the maximum principal stress. At this time,
the maximum principal stress is the earth pressure value in the passive zone.

\[ \sigma_m = \frac{1}{3} \left( 1 + 2K_0 \right) rz. \]  

(5)

In equation (5), \( \sigma_m \) is the average consolidation pressure (unit: kPa); \( K_0 \) is the coefficient of earth pressure at rest; \( r \) is the gravity of the soil (unit: kN/m\(^3\)); \( z \) is the depth of the calculation point (unit: m).

With the excavation of the foundation pit, the horizontal stress of the soil in the passive zone continues to increase. Therefore, the earth pressure \( p_p \) in the passive zone is the maximum principal stress \( \sigma_1 = \sigma_m \) and \( \sigma_3 = \sigma_m \). The strain of the soil at the retaining wall position (\( x = 0 \)) is \( \varepsilon_1 = \frac{2\eta(z)}{L} \).

Put the above parameters into equation (1), and we can get equation (6):

\[ p_p - \frac{1}{3} \left( 1 + 2K_0 \right) rz = \frac{2\eta(z)}{a + 2b\eta(z)/L}. \]  

(6)

It can be seen from the geometric relationship in Figure 4 that \( L = \tan \left( 45^\circ + \phi/2 \right) \cdot (d - z) = \sqrt{K_p} \cdot (d - z) \). Substitute this geometric relationship into equation (6).

\[ p_p = \frac{1}{3} \left( 1 + 2K_0 \right) rz + \frac{2\eta(z)}{a\sqrt{K_p} \cdot (d - z) + 2b\eta(z)}. \]  

(7)

Equation (7) is the calculation formula for the earth pressure in the passive zone below the excavation surface of the foundation pit, which takes the influence of pile deformation into account. The test constants \( a \) and \( b \) in equation (7) can be obtained by the following four approaches.

1. Carry out the indoor routine triaxial test, fit a straight line according to the relationship of \( \varepsilon_1/\sigma_1 - \sigma_3 \sim \varepsilon_1 \), and get the values of \( a \) and \( b \)
2. Derive \( a \) and \( b \) values through indoor model tests
3. Derive the values of \( a \) and \( b \) based on the measured earth pressure on site
4. Derive \( a \) and \( b \) values through numerical simulation

2.4. Earth Pressure Calculation in the Active Zone

2.4.1. Establishment of a Calculation Model. The earth pressure calculation model in the active zone is established, as shown in Figure 5. \( z \) is the calculation point depth of the soil in the disturbance zone behind the double-row piles. \( d \) and \( h \) are the pile length and the excavation depth of the foundation pit, respectively. \( \gamma \) is the angle between Rankine’s active slip surface and the horizontal plane. \( L' \) is the horizontal distance from the slip surface at depth \( z \) to the rear row of piles.

2.5. Force Analysis. Liu and Hou used the stress path triaxial tester to test the stress-strain relationship of the excavation and unloading of the soft soil foundation pit [31–41].
Table 1: The parameter of strata.

| Stratum                  | Dry weight (kN/m³) | Cohesion (kPa) | Internal friction angle (°) | Elastic modulus (MPa) | Poisson’s ratio | Stratum thickness (m) |
|--------------------------|--------------------|----------------|----------------------------|-----------------------|----------------|-----------------------|
| Miscellaneous fill       | 16.5               | 0              | 13                         | 10                    | 0.34           | 2.4                   |
| Silt                     | 19.7               | 21             | 17                         | 21                    | 0.36           | 5.6                   |
| Silty fine sand          | 20.8               | 0              | 38                         | 30                    | 0.29           | 4.0                   |
| Granular pebbles         | 21.5               | 0              | 40                         | 70                    | 0.28           | 4.0                   |
| Silty clay               | 19.8               | 29             | 25                         | 40                    | 0.35           | 8.0                   |
| Sandy pebble             | 21.5               | 0              | 45                         | 100                   | 0.26           | 16                    |

Figure 7: Excavation model of the foundation pit.
results show that the stress-strain relationship of the soil during the unloading process can be normalized by the average consolidation stress $\sigma_m$. Therefore, the soil unloading stress-strain formula represented by the Duncan-Chang hyperbolic function can also be normalized. Equation (2) can be written as follows:

$$\frac{\sigma_m \varepsilon_1}{(\sigma_1 - \sigma_3) - (\sigma_1 - \sigma_3)} = a + b \varepsilon_1. \quad (8)$$

In equation (8), $\sigma_1$ and $\sigma_3$ are the vertical stress and horizontal stress when the soil is consolidated (unit: kPa). $\varepsilon_1$ is the strain corresponding to $\sigma_1$, $\sigma_m$ is the average consolidation stress (unit: kPa). $a$ and $b$ are test constants.

Equation (8) is applicable to the stress-strain relationship of the soil in the unloaded state. For the soil in the active area behind the double-row piles, due to the excavation of the foundation pit, the vertical stress of the soil in the active area behind the rear-row piles remains unchanged, while the horizontal stress becomes smaller. Therefore, we have $\sigma_1 = \sigma_1 = rz$, $\sigma_3 = k_0rz$. That is to say, $\sigma_3$ is the strength of earth pressure in the active area behind the double-row piles and its value is $P_a$. It is assumed that the horizontal strain of the soil in the active zone and the axial strain becomes a linear relationship. That is $\varepsilon_3/\varepsilon_1 = \nu$, then, we have $\varepsilon_1 = 2\eta(z)/L' \nu$ and $L' = \tan(45^\circ - \varphi/2) \times (d - z) = \sqrt{K_s} \times (d - z)$, $\nu$ is the Poisson’s ratio of the soil.

Simplify equation (8) to obtain the calculation formula of earth pressure strength in the active zone behind the double-row piles:

$$P_a = k_0 rz - \frac{2\eta(z)(1 + 2k_0) rz}{3av\sqrt{k_a(d - z) + 6bn(z)}. \quad (9)$$

In equation (9), $k_0$ is the coefficient of earth pressure at rest; $k_a$ is the active earth pressure coefficient; $\eta(z)$ is the horizontal displacement of the pile at depth $z$ (unit: m).

2.6. Engineering Verification. Numerical simulation and field test methods are used to verify the applicability of the calculation model obtained. First, the values of parameters $a$ and $b$ in the earth pressure formula are derived through numerical simulation, and secondly, the actual earth pressure values are measured through field tests and compared with the theoretical formula results.

2.6.1. Engineering Background. A deep foundation pit project of an underground comprehensive pipeline gallery in Beijing was selected for the field test. The project is located in Yufa town and Lixian town, Daxing district, Beijing, and Guangyang district, Langfang city, Hebei province. The location is shown in Figure 6. The formation parameters are shown in Table 1.

2.6.2. Numerical Model Establishment and Analysis. The finite difference software FLAC 3D is used for simulation. The stratum parameters, construction sequence, and pile geometry parameters are consistent with the field test. The excavation sequence of the foundation pit is shown in Figure 7, and the supporting structure parameters are shown in Tables 2 and 3. To make the calculation simple and easy to solve, the following assumptions are made on the model:

(1) The top of the pile is rigidly connected to the crown beam. That is, only the bending moment is generated here without deformation.

(2) Since precipitation has already been carried out before construction, the impact of groundwater seepage is not considered.

(3) The supporting structure satisfies the basic assumption of the plane strain problem.

The FLAC 3D modeling and excavation process are shown in Figure 8.

Derive the values of $a$ and $b$ in the previous equation through the results of numerical simulation. Take the pile displacement value $\eta(z)$ corresponding to different depth $z$ to calculate the $a$ and $b$ values of the passive zone and the active zone. The calculation results are shown in Tables 4 and 5.

It can be seen in Tables 4 and 5 that the derivation of $a$ and $b$ values fluctuates slightly. The average values of $a$ and $b$ in the passive zone are $\bar{a} = 0.00612$ and $\bar{b} = -0.729$, respectively. In the same way, the values of $a$ and $b$ in the active zone are $\bar{a} = 0.38$ and $\bar{b} = 0.253$, respectively.
Figure 8: Continued.

(a) Meshing of solid elements

(b) Pile element and beam element
(c) First step of excavation

(d) The second step of excavation

Figure 8: Continued.
(e) The third step of excavation

(f) The fourth step of excavation

Figure 8: Continued.
(g) The fifth step of excavation

(h) The sixth step of excavation

Figure 8: Continued.
The seventh step of excavation

(j) The eighth step of excavation

Figure 8: Continued.
2.6.3. Field Test and Analysis. In this field test, the soil pressure and horizontal displacement on the pile body during the excavation of the foundation pit were measured. Earth pressure box, inclinometer tube, and corresponding data acquisition equipment are used for measurement. The monitoring instrument is shown in Figure 9, the layout of the monitoring points is shown in Figure 10, and the field test process is shown in Figure 11.

### Table 4: Values of $a$ and $b$ at each depth of the passive zone.

| Depth (m) | $\eta(z)$ (mm) | $a$     | $b$     |
|-----------|----------------|---------|---------|
| 17        | 11.46          | 0.005812| -0.841  |
| 18        | 10.71          |         |         |
| 19        | 10.31          | 0.006334| -0.711  |
| 20        | 10.20          |         |         |
| 21        | 10.28          | 0.006215| -0.636  |
| 22        | 10.15          |         |         |

### Table 5: Values of $a$ and $b$ at each depth of the active zone.

| Depth (m) | $\eta(z)$ (mm) | $a$     | $b$     |
|-----------|----------------|---------|---------|
| 3         | 16.97          | 0.461   | -0.22   |
| 5         | 16.54          |         |         |
| 7         | 16.2           | 0.354   | -0.18   |
| 9         | 15.68          |         |         |
| 11        | 15.1           | 0.325   | -0.36   |
| 13        | 13.8           |         |         |

3. Results

According to the earth pressure value measured in the field test, the applicability of the theoretical formula in this study is verified. As shown in Figure 12, the theoretical earth pressure value is basically the same as the field measured earth pressure value along the pile body. Their numerical changes are relatively close, and the earth pressure values gradually...
Figure 9: Monitoring equipment.

(a) Earth pressure box

(b) Inclinometer

Figure 10: Monitoring point layout map.

(a) Layout plan of the inclinometer pipe

(b) Elevation view of the arrangement of the inclinometer pipe and the earth pressure box
Figure 11: Field test construction process.

- (a) Install steel cage and inclinometer tube
- (b) Install the inclinometer tube
- (c) Excavation of the foundation pit
- (d) The excavation of the foundation pit is completed
increase as the depth increases. It can be seen that the monitoring point is 20 meters deep in the active zone. The theoretical value here is 121.2 kPa and the measured value here is 132.5 kPa. As for the 22-meter depth monitoring point in the passive zone, the theoretical value here is 172.5 kPa and the actual value here is 188.3 kPa. As a whole, the earth pressure in the active zone is less than the earth pressure in the passive zone.

4. Conclusions

Numerical simulation and field measurement had been used in this study to theoretically derive the earth pressure on the double-row piles during the excavation of the foundation pit, relying on a deep foundation pit project in Beijing area.

During the excavation of the foundation pit, due to the influence of the deformation of the double-row pile supporting structure, the stress state of the soil in the active zone and the passive zone is different. During the excavation process of the foundation pit, the horizontal stress of the soil in the active area (soils after the rear row of piles) continuously decreases, while the vertical stress is basically unchanged. The horizontal stress of the soil in the passive zone increases continuously, and the vertical stress decreases with the excavation of the foundation pit.

This study considers the nonlinear effects of soil deformation. Based on the Duncan-Chang hyperbola theory, a calculation model of earth pressure after excavation of a foundation pit supported by double-row piles is established. Subsequently, the calculation formula of earth pressure in the active and passive zones is proposed. The values of parameters $a$ and $b$ are derived through numerical simulation. The sequence of the numerical simulation working conditions is consistent with the field experiment. The average values of parameters $a$ and $b$ in the active zone are 0.38 and −0.253, respectively, and the average values of parameters $a$ and $b$ in the passive zone are 0.00612 and −0.729, respectively.

The large-scale field test is used to verify the theoretical calculation formula derived in this paper and then compares the earth pressure values of two points in the active zone and the passive zone, which are a 20-meter-depth monitoring point in the active zone and 22-meter-depth monitoring point in the passive zone. The theoretical earth pressure value is basically the same as the field measured earth pressure value along the pile body, and the numerical change is relatively close. This result shows that the theoretical earth pressure calculation formula deduced in this study is reasonable and correct.

Data Availability

The data used to support the findings of this study are included within the article.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

Acknowledgments

This study is supported by the Hebei Provincial Universities’ Basic Scientific Research Operating Expenses (no. JQN2020027) and the doctoral research initiation fund of North China University of Science and Technology (no. 28418599).

References

[1] A. T. C. Goh, R. H. Zhang, W. Wang, L. Wang, H. L. Liu, and W. G. Zhang, "Numerical study of the effects of groundwater drawdown on ground settlement for excavation in residual soils," Acta Geotechnica, vol. 15, no. 5, pp. 1259–1272, 2020.
[2] J. H. Liu and X. Y. Hou, Handbook of Foundation Pit Engineering, vol. 4, China Construction Industry Press, Beijing, 1997.

[3] L. J. Li, Numerical Analysis and Field Test Research of Double-Row Pile Supporting Structure, Taiyuan University of Technology, Taiyuan, 2013.

[4] J. Cao, G. W. Qian, Y. Gao, and H. X. Zuo, "Research on equivalent calculation model of soil between piles supported by double-row piles in foundation pit," Chinese Journal of Underground Space and Engineering, vol. 16, no. 3, pp. 749–757, 2020.

[5] Y. Dong, "Analysis of deformation behavior of double-row pile supporting structure in deep foundation pit in soft soil," Building Structure, vol. 47, no. 23, pp. 86–91, 2017.

[6] Y. Gao, H. K. Sun, and R. C. Liu, "Double-row pile support design and deformation law of foundation pit," Journal of Shandong University (Engineering Science Edition), vol. 49, no. 3, pp. 86–94, 2019.

[7] X. F. Jin, Z. J. Xian, Y. Z. Yu, and L. D. Ma, "Application and analysis of double-row pile supporting structure in deep foundation pit of Tianyue Square in Hefei," Building Structure, vol. 47, no. 23, pp. 80–85, 2017.

[8] Y. J. Shen, "Optimal design of double-row anti-slide pile top connection mode," Chinese Journal of Rock Mechanics and Engineering, vol. 29, no. 5, pp. 34–38, 2010.

[9] L. Zhang and X. G. Zhu, "Analysis of force and deformation of double-row piles in layered soil considering the interaction of piles and piles," Journal of Human University (Natural Science Edition), vol. 47, no. 11, pp. 120–126, 2020.

[10] S. H. Mirmoradi and M. Ehrlich, "Effects of facing, reinforcement stiffness, toe resistance, and height on reinforced walls," Geotechnics and Geomembranes, vol. 45, no. 1, pp. 67–76, 2017.

[11] D. Huang and J. Liu, "Upper-bound limit analysis on seismic rotational stability of retaining wall," KSCE Journal of Civil Engineering, vol. 20, no. 7, pp. 2664–2669, 2016.

[12] A. Guha Ray and D. Baidya, "Reliability coupled sensitivity-based seismic analysis of gravity retaining wall using pseudostatic approach," Journal of Geotechnical and Geoenvironmental Engineering, vol. 142, no. 6, pp. 2151–2160, 2016.

[13] S. Han, J. X. Gong, and Y. Q. Zhang, "Earth pressure of layered soil on retaining structures," Soil Dynamics and Earthquake Engineering, vol. 83, pp. 33–52, 2016.

[14] P. J. Santos and P. L. A. Barros, "Active earth pressure due to soil mass partially subjected to water seepage," Canadian Geotechnical Journal, vol. 52, no. 11, pp. 1886–1891, 2015.

[15] E. Ellis and R. Aslam, "Arching in piled embankments: comparison of centrifuge tests and predictive methods," Ground Engineering, vol. 42, no. 6, pp. 34–38, 2009.

[16] V. J. Pons, "Finite-element study of arching behaviour in reinforced fills," Ground Improvement, vol. 163, no. 4, pp. 217–229, 2010.

[17] R. P. Chen, Z. Z. Xu, Y. M. Chen, D. S. Ling, and B. Zhu, "Field tests on pile-supported embankments over soft ground," Journal of Geotechnical and Geoenvironmental Engineering, vol. 136, no. 6, pp. 777–785, 2010.

[18] J. A. Sloan, G. M. Filz, and J. G. Collin, "A generalized formulation of the adapted Terzaghi method of arching in column-supported embankments," in Proc. of Geo Frontiers, GSP 211 Advanced in Geotechnical Engineering, pp. 798–805, Dallas, Texas, USA, 2011.

[19] S. J. M. Eekenlen, A. Bezuijen, and H. J. Lodder, "Model experiments on piled embankments. Part I," Geotextiles and Geomembranes, vol. 32, pp. 69–81, 2012.

[20] K. Abe, S. Nakamura, H. Nakamura, and K. Shiomi, "Numerical study on dynamic behavior of slope models including weak layers from deformation to failure using material point method," Soils and Foundations, vol. 57, no. 2, pp. 155–175, 2017.

[21] P. Huang, S. Li, H. Guo, and Z. M. Hao, "Large deformation failure analysis of the soil slope based on the material point method," Computational Geosciences, vol. 19, no. 4, pp. 951–963, 2015.

[22] B. Wang, P. J. Vardon, and M. A. Hicks, "Investigation of retrogressive and progressive slope failure mechanisms using the material point method," Computers and Geotechnics, vol. 78, Supplement C, pp. 88–98, 2016.

[23] Y. Dong, D. Wang, and M. F. Randolph, "Runout of submarine landslide simulated with material point method," Journal of Hydrodynamics, vol. 29, no. 3, pp. 438–444, 2017.

[24] K. Soga, E. Alonso, A. Yerro, K. Kumar, and S. Bandara, "Trends in large-deformation analysis of landslide mass movements with particular emphasis on the material point method," Géotechnique, vol. 66, no. 3, pp. 248–273, 2016.

[25] M. A. Llanos-Serna, M. M. Farias, and D. M. Pedrosa, "An assessment of the material point method for modelling large scale run-out processes in landslides," Landslides, vol. 13, no. 5, pp. 1057–1066, 2016.

[26] W. T. Solowski and S. W. Sloan, "Evaluation of material point method for use in geotechnics," International Journal for Numerical and Analytical Methods in Geomechanics, vol. 39, no. 7, pp. 685–701, 2015.

[27] H. Liu, D. Kong, W. S. Gan, and B. J. Wang, "Seismic active earth pressure of limited backfill with curved slip surface considering intermediate principal stress," Applied Sciences, vol. 12, no. 1, p. 169, 2022.

[28] H. B. Zhang, G. X. Zhang, G. F. An, and T. J. Liu, "Study on the deformation and force characteristics of the supporting structure with sliced crown beam and double-row piles," Industrial Construction, vol. 44, no. 3, 2014.

[29] Technical Specification for Building Foundation Pit Support JGJ 120-2012 People’s Republic of China Industry Standard, 2012.

[30] B. Yang and L. Q. Hu, "Experimental study on the relationship between lateral earth pressure and horizontal displacement of retaining structure," Building Science, vol. 16, no. 2, pp. 14–20, 2000.

[31] G. B. Liu and H. Xueyuan, "Unloading stress-strain characteristics of soft soil," Underground Engineering and Tunnels, vol. 2, pp. 16–23, 1997.

[32] J. H. Chen, H. B. Zhao, F. L. He, J. W. Zhang, and T. K. Ming, "Studying the performance of fully encapsulated rock bolts with modified structural elements," International Journal of Coal Science & Technology, vol. 8, no. 3, pp. 64–76, 2021.

[33] Y. L. Chen, J. P. Zuo, D. J. Liu, and Y. J. Liu, "Experimental and numerical study of coal-rock bimaterial composite bodies under triaxial compression," International Journal of Coal Science & Technology, vol. 8, no. 5, pp. 908–924, 2021.

[34] G. L. Feng, B. R. Chen, Y. X. Xiao et al., "Microseismic characteristics of rockburst development in deep TBM tunnels with alternating soft-hard strata and application to rockburst warning: A case study of the Neelum-Jhelum hydropower project," Tunnelling and Underground Space Technology, vol. 122, p. 104398, 2022.
[35] Y. Yu, G. L. Feng, C. Xu, B. R. Chen, D. X. Geng, and B. T. Zhu, “Quantitative threshold of energy fractal dimension for immediate Rock Burst warning in deep tunnel: a case study,” Lithosphere, vol. 2021, no. Special 4, p. 1699273, 2022.

[36] G. L. Feng, X. T. Feng, B. R. Chen, Y. X. Xiao, and Y. Yu, “A microseismic method for dynamic warning of rockburst development processes in tunnels,” Rock Mechanics and Rock Engineering, vol. 48, no. 5, pp. 2061–2076, 2015.

[37] X. L. Li, Z. Y. Cao, and Y. L. Xu, “Characteristics and trends of coal mine safety development,” Energy Sources, Part A: Recovery, Utilization, and Environmental Effects, pp. 1–14, 2020.

[38] S. M. Liu, X. L. Li, D. K. Wang, and D. Zhang, “Experimental study on temperature response of different ranks of coal to liquid nitrogen soaking,” Natural Resources Research, vol. 32, no. 2, pp. 1467–1480, 2021.

[39] S. M. Liu, X. L. Li, and W. D. K. Wang, “Investigations on the mechanism of the microstructural evolution of different coal ranks under liquid nitrogen cold soaking,” Energy Sources, Part A: Recovery, Utilization, and Environmental Effects, pp. 1–17, 2020.

[40] X. L. Li, S. J. Chen, Q. M. Zhang, X. Gao, and F. Feng, “Research on theory, simulation and measurement of stress behavior under regenerated roof condition,” Geomechanics and Engineering, vol. 26, no. 1, pp. 49–61, 2021.

[41] X. L. Li, S. J. Chen, S. M. Liu, and Z. H. Li, “AE waveform characteristics of rock mass under uniaxial loading based on Hilbert-Huang transform,” Journal of Central South University, vol. 28, no. 6, pp. 1843–1856, 2021.